COMMUNITY ASSISTANCE
PLANNING REPORT NO. 190

A STORMWATER MANAGEMENT AND FLOOD CONTROL PLAN FOR THE LILLY CREEK SUBWATERSHED

VILLAGE OF MENOMONEE FALLS
WAUKESHA COUNTY
WISCONSIN

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WAUKESHA COUNTY, WISCONSIN

Prepared by the

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February 22, 1993

Village President, Village Board,
and Village Stormwater Management
and Flood Control Advisory Committee
Village of Menomonee Falls
W156 N8480 Pilgrim Road
Menomonee Falls, Wisconsin 53051

Ladies and Gentlemen:

In April of 1987, the Village of Menomonee Falls entered into an agreement with the Southeastern Wisconsin Regional Planning Commission governing the preparation of a stormwater management and flood control plan for the Lilly Creek subwatershed. The Regional Planning Commission staff, working in cooperation with Village and Wisconsin Department of Natural Resources staffs and the firm of BRW, Inc., environmental consultants retained by the Village, has now completed a recommended stormwater management and flood control plan for the Lilly Creek subwatershed. The plan is herewith transmitted for consideration and adoption by the Village Stormwater Management and Flood Control Advisory Committee, the Village Plan Commission, and the Village Board.

The stormwater management and flood control plan presented herein is consistent with regional as well as local land use development, water quality management, and flood control objectives, and is intended to serve as a guide to Village officials in the making of sound decisions over time concerning the development of stormwater management and flood control facilities in the Lilly Creek subwatershed. The Lilly Creek plan is also intended to refine and detail the previously adopted comprehensive plan for the Menomonee River.

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

Sincerely,

Kurt W. Bauer
Executive Director

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Chapter I

INTRODUCTION

The Lilly Creek subwatershed is located within the Menomonee River watershed in the Village of Menomonee Falls in northeastern Waukesha County. In 1985 the resident population of the Lilly Creek subwatershed was approximately 5,900 persons. The projected ultimate population of the subwatershed is approximately 17,800 persons, an increase of about 11,900 persons, or about three times the 1985 level. To accommodate this projected increase in population, urban land use within the subwatershed may be expected to increase from a total of about 2.84 square miles in 1985 to about 5.38 square miles—an increase of about 2.54 square miles, or about 89 percent over the 1985 level. In the absence of adequate planning, this conversion of land from rural to urban use may be expected to aggravate existing stormwater management and flood control problems and to create new problems. Recognizing the need for a systematic plan to address existing stormwater management and flooding problems and to avoid the creation of new problems as urban development proceeds in the area, the Village in April 1987 entered into an agreement with the Southeastern Wisconsin Regional Planning Commission to assist in the preparation of such a plan. The planning work was funded by a Local Assistance Grant from the State of Wisconsin, which was awarded on June 10, 1987.

The purpose of this report is to present the resulting stormwater management and flood control plan.[1] The plan seeks to promote the development of an effective stormwater management and flood control system, adequate to serve that portion of the Village within the Lilly Creek subwatershed under planned ultimate development conditions. To the extent practicable, the plan is intended to ameliorate existing stormwater drainage and flood control problems; to avoid the creation of new stormwater drainage and flood control problems as the area continues to develop; and to mitigate the effects of nonpoint source pollution on surface water quality. More specifically, this report:

1. Describes the existing stormwater management system and the existing stormwater management and flood control problems in the subwatershed and identifies the causes of those problems;
2. Describes existing and planned land use conditions and identifies related stormwater management requirements;
3. Includes an assessment of existing water quality conditions and biological conditions in the Lilly Creek stream system;
4. Provides a set of objectives and supporting standards to guide the development of an effective stormwater management and flood control system;
5. Presents alternative stormwater management and flood control plans;
6. Provides a comparative evaluation of the technical, economic, and environmental features of the alternative plans;
7. Recommends a cost-effective stormwater management and flood control plan for the Lilly Creek subwatershed consisting of various structural and nonstructural measures; and
8. Identifies the responsibilities of, and actions required by, the various governmental units and agencies that will implement the recommended plan.

This report was prepared by the staff of the Southeastern Wisconsin Regional Planning Commission in cooperation with the staffs of the

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[1] *The distinction between stormwater drainage, stormwater management, and flood control is not always clear. For the purposes of this report, drainage is defined as the control of excess stormwater on the land surface before such water has entered stream channels. Flood control is defined as the prevention of damage from the overflow of natural streams and watercourses. The term stormwater management encompasses both stormwater drainage and nonpoint source pollution control measures. Nonpoint source pollution is defined as the degradation of water quality in streams, lakes, ponds, or groundwater aquifers owing to the introduction of pollutants which are washed off impervious and pervious land surfaces during periods of rainfall and snowmelt.*

Village of Menomonee Falls and the Wisconsin Department of Natural Resources (DNR). The recommended stormwater management and flood control plan for the Lilly Creek subwatershed as presented herein is properly set within the context of broader flood control and water quality management plans for the Menomonee River watershed.[2] The findings and recommendations of urban nonpoint source pollution control studies being conducted by the DNR for the Lilly Creek subwatershed as part of the Menomonee River Priority Watershed Program are also reflected in the alternative stormwater management plans and the recommended plan presented in this report.

NEED FOR AND IMPORTANCE OF STORMWATER MANAGEMENT AND FLOOD CONTROL PLANNING

Stormwater management and flood control are among the most important and costly requirements of sound urban development. Good stormwater and floodland management are essential to the provision of an attractive and efficient, as well as safe and healthful, environment for urban life.

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

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

Occupancy of natural floodlands by flood-vulnerable land uses, together with development-induced changes in the flow characteristics of streams, can produce serious flood problems in a watershed. To ensure that future flood damage will be held to a minimum, plans for the proper utilization of the riverine areas of a watershed must be developed so that control of land uses in flood hazard areas, public acquisition of floodlands, and river engineering can be used to properly direct new development into a pattern compatible with the demands of the river system on its natural floodlands and to achieve an adjustment or balance between land use development and floodwater flow and storage needs.

Because of their important social, economic, and environmental impacts, stormwater management and flood control problems require sound

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[2] *See SEWRPC Planning Report No. 26, A Comprehensive Plan for the Menomonee River Watershed, October 1976; and SEWRPC Planning Report No. 30, A Regional Water Quality Management Plan for Southeastern Wisconsin: 2000, Volume One, Inventory Findings, September 1978, Volume Two, Alternative Plans, February 1979, and Volume Three, Recommended Plan, June 1979. The Menomonee River watershed plan has been formally adopted by the Wisconsin Department of Natural Resources and Waukesha County, as well as by the Regional Planning Commission. The regional water quality management plan has been adopted by the Wisconsin Department of Natural Resources, as well as the Commission. Also see the Milwaukee River Priority Watersheds Program Prospectus, Wisconsin Department of Natural Resources and SEWRPC, March 1985; and SEWRPC Community Assistance Planning Report No. 152, Stormwater Drainage and Flood Control System Plan for the Milwaukee Metropolitan Sewerage District, December 1990.*

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

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

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

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

BASIC CONCEPTS INVOLVED IN STORMWATER MANAGEMENT

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

The new concept emphasizes storage as well as conveyance, with the objectives of reducing the peak rate of runoff, and in some cases the total volume of runoff; reducing the transport of sediment and other water pollutants to downstream surface waters; and protecting against increased downstream flooding. The new concept also looks to controlling the quality, as well as the quantity, of runoff.

Regardless of the concept, urban stormwater management systems are generally designed to fulfill four basic objectives: 1) to prevent significant damage to buildings, other structures, and other forms of real property from relatively infrequent major rainfall events; 2) to maintain reasonably convenient access to and egress from the various land uses of an urban area during relatively frequent minor rainfall events; 3) to avoid undue hazards to public safety and health;

and 4) to mitigate the effects of nonpoint source pollutants on receiving watercourses. Thus, the total stormwater management system of an urban area may be conceived of as consisting of a major element operating infrequently and a minor element operating frequently. Both of these elements can, under certain conditions, utilize stormwater retention or detention, as well as conveyance, as a design solution. The benefits of stormwater storage may include a reduction in the high kinetic energy of surface runoff; a reduction in both the total volume and peak rate of discharge; the provision of multiple-use opportunities for recreational and aesthetic purposes; the provision of groundwater recharge; the entrapment of some pollutants; and a reduction in the adverse impacts of the remaining pollutants by controlled release.

For predominantly developed parts of urban communities—such as the established areas of the Village of Menomonee Falls—the development of stormwater storage and nonpoint source pollution control measures may be difficult, such development being constrained by the availability of open land on, or adjacent to, the drainage system. Some storage potential may exist within the developed areas such as on parking lots in commercial and industrial areas, in grass drainage swales, and on site in residential and recreational areas. Successful efforts have been made to integrate stormwater facilities into existing urban environments; however, such efforts have been costly and difficult to implement because of the existing development patterns and public concerns. Nevertheless, the practice of detaining or retaining stormwater runoff within the confines of an urban area as well as in developing areas to mitigate flooding, soil erosion, sedimentation, and surface water pollution deserves careful consideration as a part of any sound stormwater management planning effort. In outlying developing areas, the incorporation of stormwater storage facilities and nonpoint source pollution control measures may be more feasible because of the availability of land and the opportunity to plan for such facilities as an integral part of the urban development process.

Facilities designed solely for the control of stormwater quantity, including storm sewers, concrete-lined drainage channels, and dry detention basins which drain completely between storms, provide little or no reduction in nonpoint source pollutant loadings to receiving watercourses. However, when such facilities are integrated with nonpoint source pollution control measures such as wet detention basins, infiltration trenches, percolation basins, grass swales and waterways, and regular street sweeping and catch basin cleaning, a significant reduction in pollutant loadings may be achieved.

SCOPE OF THE STORMWATER MANAGEMENT AND FLOOD CONTROL PLAN

The recommended stormwater management and flood control plan for the 5.65-square-mile Lilly Creek subwatershed, as set forth in this report, incorporates compatible multiple-use planning concepts and recognizes the constraints imposed by other community needs, such as park and open space, transportation, sanitary sewerage, and water supply. Drainage, flood control, and nonpoint source pollution control requirements under existing and planned ultimate land use conditions are evaluated. The planning effort considered both the stormwater management facilities needed to serve areas that are planned to be converted from rural to urban land uses and the degree of rehabilitation needed to properly maintain, improve, or extend the existing stormwater management system serving that portion of the Village within the Lilly Creek subwatershed.

HISTORY OF SIGNIFICANT RECENT EVENTS RELATED TO STORMWATER MANAGEMENT AND FLOOD CONTROL ISSUES WITHIN THE LILLY CREEK SUBWATERSHED

One of the first steps in the preparation of this plan was a careful review of the findings and recommendations of previous stormwater management and flood control studies affecting the Lilly Creek subwatershed. Information on past studies related to stormwater management and flood control issues within the subwatershed was assembled from the files of the Village of Menomonee Falls; the Wisconsin Department of Natural Resources; Ruekert & Mielke, Inc., consulting engineers; and the Regional Planning Commission. That information, supplemented by interviews with the staffs of the above organizations, was used to compile a history of the significant events related to past stormwater

Table 1

100-YEAR RECURRENCE INTERVAL FLOOD DISCHARGES DEVELOPED FOR LILLY CREEK: 2000

		100-Year Recurrence Interval Flood Instantaneous Peak Discharge (cfs)					
		Menomonee River Watershed Study[a,e]			Village of Menomonee Falls Study[b,e]	Ruekert & Mielke 1984 Study[c,g]	SEWRPC 1984 Study[d,f,g]
Location	River Mile	Existing Land Use and Existing Channel	Planned Land Use and Existing Channel	Planned Land Use and Planned Channel	Planned Land Use and Planned Channel	Planned Land Use and Planned Channel	Planned Land Use and Planned Channel
Appleton Avenue	0.40	1,230	2,160	2,600	3,440	3,160	2,440
Good Hope Road	0.84	1,230	2,160	2,600	3,230	2,560	1,780
Brentwood Drive	1.06	1,230	2,160	2,600	3,230	2,350	1,780
Mill Road	1.88	540	855	1,300	2,560	1,730	1,120
Chicago & North Western Railway . . .	2.59	540	855	1,300	1,820	1,220	1,120
Silver Spring Drive . . .	2.97	540	855	1,300	1,720	920	1,120

[a] SEWRPC Planning Report No. 26, A Comprehensive Plan for the Menomonee River Watershed, October 1976.

[b] "Lilly Creek Channelization Proposal," supporting calculations, Village of Menomonee Falls, May 12, 1976.

[c] Lilly Creek Stormwater Management Plan-Village of Menomonee Falls, Ruekert & Mielke, Inc., Consulting Engineers, October 1984.

[d] Appendix B, prepared by SEWRPC, July 12, 1984, of Lilly Creek Stormwater Management Plan-Village of Menomonee Falls.

[e] Without detention storage constructed within the subwatershed.

[f] Existing land use and channel conditions and planned land use and existing channel conditions (both without detention storage) are the same as for the Menomonee River watershed study.

[g] With five constructed detention basins in the subwatershed along with natural detention upstream of Silver Spring Drive.

Source: SEWRPC.

management and flood control planning efforts within the Lilly Creek subwatershed.

SEWRPC Planning Report No. 26, A Comprehensive Plan for the Menomonee River Watershed, October 1976, set forth a recommended flood control plan for the Menomonee River watershed, including Lilly Creek. Ten-, 50-, and 100-year recurrence interval flood discharges under then existing and planned land use and channel conditions were developed for the Menomonee River and Lilly Creek from a statistical analysis of simulated historic peak flood discharges. Historic flood hydrographs were developed using the Hydrocomp continuous simulation hydrologic model. Meteorological data for the 35-year period from 1940 through 1974, along with soil type, slope, and land cover data characteristic of each subbasin within the watershed, were input to the model. The 100-year recurrence interval flood discharges developed for Lilly Creek for the Menomonee River watershed study under existing land use and channel conditions, planned land use and existing channel conditions, and planned land use and planned channel conditions are summarized in Table 1.

The three flood control alternatives that were considered for that portion of the Menomonee River within the Village of Menomonee Falls and for Lilly Creek included floodproofing and removal of structures, locally proposed channel modifications, and bridge and culvert alteration or replacement. The locally proposed channel modifications were initially described in the "Menomonee River Channelization Proposal" and the "Lilly Creek Channelization Proposal," prepared in 1976 by Max A. Vogt, Menomonee Falls Village Engineer. The Lilly Creek proposal presented two alternative channel modification plans in the reach from the mouth to Silver

Spring Drive. The alternatives were designed to eliminate flooding of buildings during the 100-year recurrence interval flood under year 2000 land use conditions and to provide adequate outlets for anticipated storm sewers. Peak discharges at various locations along Lilly Creek were developed using the Rational Method with 100-year recurrence interval rainfall data. The channel improvements were designed to accommodate all flows up to and including the 100-year recurrence interval event. The Rational Method discharges calculated by the Village are given in Table 1.

The first alternative presented in the village proposal considered a trapezoidal concrete-lined channel with a 12-foot bottom width, side slopes of one vertical on three horizontal, and an average depth of 10 feet. The concrete lining was designed to extend up to the 100-year recurrence interval flood depth of about seven feet. Under this alternative, the channel invert would be lowered an average of five feet and streambed slopes would range from 0.00075 foot per foot to 0.0025 foot per foot. In areas where the available right-of-way is limited, such as along Manor Hills Boulevard, retaining walls were called for above the 100-year recurrence interval flood level. The second alternative considered a turf-lined channel with the same streambed and side slopes and flow depths as for the first alternative, but with a 20-foot bottom width. The report also recommended the replacement of 10 bridges along Lilly Creek under either alternative. It was proposed that stormwater drainage needs be met through the construction of trunk storm sewers which would discharge to Lilly Creek. Cost estimates were presented for the two channel modification alternatives and the alternatives were compared to the floodproofing and structure removal alternative presented in the Menomonee River watershed plan.

The Menomonee River watershed plan incorporated the main features of the village concrete- and turf-lined channelization proposals into a single alternative which retained the cross-sectional shapes, sizes, and slopes proposed by the Village. The alternative called for a turf-lined channel upstream of Jerry Lane extended (River Mile 1.55) to W. Silver Spring Drive (River Mile 2.97), and a concrete-lined channel downstream of Jerry Lane extended to the confluence with the Menomonee River. Other modifications to the village proposals which were made in the watershed plan included providing concrete lining only up to the 10-year recurrence interval flood elevation rather than up to the 100-year elevation, and removing, but not replacing, four little-used private bridges which were not considered necessary for future development of local areas.

It was concluded in the watershed plan that both the floodproofing and channel modification alternatives were technically feasible means of resolving existing and forecast flood problems along Lilly Creek and along those reaches of the Menomonee River located within the Village. The benefit-cost ratio of the floodproofing alternative for Lilly Creek was estimated to be 1.38, as compared to a benefit-cost ratio of 0.69 for the channel modification alternative. The benefit-cost ratio for channel modification along the Menomonee River was estimated to be 0.27, while ratios for structure floodproofing and removal along the three reaches of the Menomonee River within the Village ranged from 1.87 to 13.60. After consideration of the technical and economic aspects, as well as certain nontechnical and noneconomic issues, Commission staff recommended the adoption of the structure floodproofing and removal alternatives for both Lilly Creek and those reaches of the Menomonee River located within the Village.

In light of the village commitment to channelization as reflected by the location and size and grades of existing and proposed storm sewers and storm sewer outfalls, the Menomonee River Watershed Committee, which included representatives of the Wisconsin Department of Natural Resources, did not accept the recommendation of Commission staff, and instead recommended that the channelization alternative be used to resolve flooding problems along Lilly Creek and the Menomonee River within the Village of Menomonee Falls.

A flood insurance study for the Lilly Creek subwatershed was issued by the Federal Emergency Management Agency in July 1978. The study is documented in the Flood Insurance Study for the Village of Menomonee Falls, Waukesha County, Wisconsin. The study used estimated peak 10-, 50-, 100-, and 500-year recurrence interval flood discharges at selected locations along Lilly Creek as determined by the Regional Planning Commission for the Menomonee River watershed plan. The discharges for then-existing land use and channel conditions

Table 2

FLOOD DISCHARGES FOR LILLY CREEK

Discharge Location	U. S. Public Land Survey Section	Peak Discharges[a] (cubic feet per second)		
		10-Year Recurrence Interval Flood Event	50-Year Recurrence Interval Flood Event	500-Year Recurrence Interval Flood Event
Appleton Avenue	NW 1/4, SW 1/4, Section 13, T8N, R20E	510	970	2,100
Mill Road	NE 1/4, NE 1/4, Section 26, T8N, R20E	190	400	1,000

[a]*Discharges based on 1975 land use and channel conditions.*

Source: Federal Emergency Management Agency 1978 flood insurance study and SEWRPC.

were used to compute water surface profiles for the study. The 100-year recurrence interval discharges for existing land use and channel conditions that were used for the study are the same as those listed for the Menomonee River watershed study in Table 1. Peak 10-, 50-, and 500-year discharges used for the flood insurance study are given in Table 2.

The Village subsequently contracted with Ruekert & Mielke, Inc., consulting engineers, to prepare a second level stormwater drainage and flood control plan for the Lilly Creek subwatershed to implement the recommendations of the Menomonee River watershed study. The resulting plan was entitled Lilly Creek Stormwater Management Plan-Village of Menomonee Falls, and was issued in October 1984.

The plan called for 22 new trunk storm sewer outfalls discharging to Lilly Creek, ranging in size from an 18-inch-diameter circular pipe to a 10-foot-wide by 5-foot-high box culvert. The trunk storm sewers were designed to replace portions of the existing swale and open channel drainage system in the subwatershed. To accommodate the recommended trunk storm sewers and to provide flood control benefits, channel modifications to Lilly Creek were recommended for the 2.97-mile-long reach from its mouth to Silver Spring Drive. The modified channel was trapezoidal with a concrete lining, a 12-foot bottom width, side slopes of one vertical on three horizontal, and depths ranging from 7.5 to 11.5 feet. The channel invert was proposed to be lowered an average of approximately four feet and streambed slopes would range from 0.001 foot per foot to 0.0026 foot per foot. Culvert replacements were recommended at eight locations along Lilly Creek, and the construction of one new crossing of the creek was proposed.

Five detention basins were proposed in order to reduce planned condition 100-year recurrence interval flows to levels which would not exceed the hydraulic capacities of the relatively new structures at Good Hope Road and Appleton Avenue. The basins were proposed to be located in the southeast and southwest one-quarters of U.S. Public Land Survey Section 26, Town 8 North, Range 20 East and in the southeast, southwest, and northeast one-quarters of Section 23, Town 8 North, Range 20 East.

The flood hydrographs used to size the trunk storm sewers and the modified Lilly Creek channel were developed using U. S. Soil Conservation Service procedures. A 10-year recurrence interval storm with a 24-hour duration was used to design the stormwater drainage systems for areas of existing and planned development that are tributary to the proposed trunk storm sewers; a 25-year, 24-hour storm was used to design the trunk storm sewer system; and a 100-year, 24-hour storm was used to design the Lilly Creek channel modifications and hydraulic structure replacements. The DRAINCALC computer program was used to develop, combine, and route flood hydrographs. Future land use conditions assumed ultimate development based on the village zoning for lands within the subwatershed. Table 1 lists the 100-year, 24-hour storm flood discharges under planned land use and planned channel and detention storage conditions.

The report included as an appendix a study prepared by the Regional Planning Commission addressing the effects of the recommended Lilly Creek drainage and flood control measures under existing and planned land use conditions on 100-year recurrence interval flood stages on the Menomonee River. For the Menomonee River watershed study, 100-year recurrence interval flood flows in the Menomonee River were determined using the Hydrocomp simulation model. In order to make a consistent determination of the effects of the proposed Lilly Creek channel modifications and bridge replacements on flood stages in the Menomonee River, it was also necessary to use the Hydrocomp model to develop flows for Lilly Creek with the proposed modifications in place. Table 1 presents flows determined using the Hydrocomp model for planned land use, channel, and storage conditions. The study concluded that in order to limit the increase in the 100-year recurrence interval flood stage on the Menomonee River under existing land use conditions to less than 0.1 foot, as was required by the village floodplain zoning ordinance and the version of Chapter NR 116 of the Wisconsin Administrative Code in force at that time, it would be necessary to construct one of the proposed detention basins at the same time as the recommended channel modifications. That detention basin was located in Section 26 upstream of the Chicago & North Western railway crossing. The Ruekert & Mielke report stated that the Village planned to implement the proposed channel modifications with the single detention basin. The report called for the remaining four detention basins to be constructed in the future as additional development occurred.

At the request of the Village, an additional report entitled The Lilly Creek Drainage System-A Stormwater Management Plan for the Subwatershed South of Silver Spring Drive was prepared by Ruekert & Mielke in January 1985. That report presented a stormwater management plan for the 411-acre area tributary to Lilly Creek in the southernmost portion of the subwatershed. A stormwater management system for that area was not specifically addressed in the 1984 Ruekert & Mielke study. Hydrologic analyses performed for the 1984 study were used to prepare a plan for accommodating stormwater runoff within the southernmost 411 acres of the subwatershed. The main features of the 1985 plan were the conveyance of most of the runoff from the area in a 4,650-foot-long, trapezoidal, turf-lined channel located along the south side of Silver Spring Drive, and the utilization of the large wetland area located south of Silver Spring Drive as a natural detention basin. The flows resulting from a 100-year recurrence interval storm that were used in the design of the channel are given in Table 3.

Facilities planning and design work for construction of a trunk sewer to extend sanitary sewer service to the Lilly Creek subwatershed were conducted simultaneously with the village stormwater management planning effort, with the intention of constructing the trunk sewer and the recommended drainage and flood control measures in a coordinated manner in order to minimize disruption to the affected property owners and to achieve an overall reduction in cost.

In order to obtain approval for the channel modification project, the Village applied to the Department of Natural Resources for permits under Sections 30.12, 30.19, and 30.195 of the Wisconsin Statutes. During the permitting process, the DNR raised concerns regarding the impacts of the proposed channel modification project on the water quality of Lilly Creek and on fish and wildlife habitats. As a result of these concerns, several alternatives to the concrete-lined channel modification plan presented in the 1984 Ruekert & Mielke report were developed. The first of these alternatives was proposed by Ruekert & Mielke, on behalf of the Village, in a memorandum to DNR Southeast District staff dated November 28, 1984. That memorandum proposed that a modified concrete-lined channel be provided only along the two most flood-prone reaches of Lilly Creek, those being the 1,200-foot-long reach in the Bowling Green Industrial Park between the Chicago & North Western railway tracks and Kaul Avenue and the 2,600-foot-long reach in the North Hills Manor Subdivision between Good Hope Road and Oakwood Drive extended. The remaining 7,400 feet of stream channel between Appleton Avenue and Silver Spring Drive would be widened and deepened, but lined with rock riprap rather than concrete. The originally proposed detention basin between the Chicago & North Western Railway and

Table 3

100-YEAR RECURRENCE INTERVAL FLOOD DISCHARGES DEVELOPED FOR THE SOUTHERNMOST TRIBUTARY TO LILLY CREEK ALONG SILVER SPRING DRIVE: 2000

Location	River Mile from Mouth of Tributary	100-Year Storm Peak Discharge (cfs)
Mouth	0.00	840
Badger Drive	0.35	780
Butternut Drive	0.55	720
Pilgrim Road	0.64	690

Source: Ruekert & Mielke, Inc., and SEWRPC.

Silver Spring Drive would be retained under this alternative.

In a December 1984 environmental assessment of the channel modification project proposed by the Village, the DNR set forth two additional alternatives. One alternative included a detention basin proposed by the Village, along with a riprap-lined channel having natural vegetation along both stream banks wherever possible. The second alternative proposed that the Village undertake "a stormwater management study and implementation program. . .to determine what the real causes of the flooding are and pinpoint alternative means of handling stormwater."

Agreement was not reached between the Village and the DNR on an alternative. The DNR formally objected to the Village's proposed project and a contested case hearing was held in March and April of 1985. The State of Wisconsin hearing examiner's decision issued in September 1985 denied the permit applications of the Village. The hearing examiner's decision was appealed by the Village and subsequently upheld in Waukesha County Circuit Court. The Village appealed the Circuit Court decision to the State Court of Appeals, which decided in favor of the DNR position in July 1987. In December 1987, the Wisconsin Supreme Court denied review of the previous lower court rulings, effectively ending the legal process within the State.

In 1986, concurrent with the legal actions set forth above, the Village and the DNR began preliminary discussions regarding preparation of a stormwater management plan for Lilly Creek. The need for stormwater management and flood control planning within the Lilly Creek subwatershed was emphasized by the relatively extensive flooding and drainage problems experienced as the result of several rainstorms in August and September of 1986. The Regional Planning Commission was first directly involved in those discussions in August 1986. In April 1987, the Village entered into an agreement with the Commission to assist in the preparation of a stormwater management and flood control plan which would address all practicable alternatives for alleviating existing and anticipated future stormwater management and flood control problems in the Lilly Creek subwatershed, and would result in a recommended plan which met multiple objectives for water quantity, water quality, and land use to the greatest degree possible. The plan presented herein is the result of that agreement.

The Commission began preliminary data inventory work for this stormwater management and flood control plan in June 1987. Waukesha County and the Commission were proceeding with the preparation of large-scale topographic mapping of the Village of Menomonee Falls at the same time that the initial inventory work for this system plan was being conducted. In addition, in August 1987 the Commission, at the request of the Village of Menomonee Falls, undertook the task of updating the land use and transportation elements of the 1973 village master plan. Because the hydrologic, hydraulic, and nonpoint source pollutant loading modeling which form the basis for this planning effort would be greatly enhanced by using the new topographic mapping and the updated land use plan, it was decided to wait until substantial portions of the new topographic mapping and the land use plan were available before proceeding with significant additional work on this plan. Following the completion in 1988 of most of the topographic maps covering the area within the Lilly Creek subwatershed and the issuance of the second preliminary draft land use plan in August 1988, the Commission resumed work on the stormwater management and flood control plan in September 1988.

ADDITIONAL STUDIES

Additional studies which were considered during preparation of this plan were the preliminary drafts of the Milwaukee River Basin Integrated

Resource Management Plan, Volume 4, Menomonee River Watershed Integrated Resource Management Plan: 2000, and A Nonpoint Source Control Plan for the Menomonee River Priority Watershed Project, both of which were prepared by the Wisconsin Department of Natural Resources. The Menomonee River Watershed Integrated Resource Management Plan: 2000 addresses both environmental protection and resource management activities in the watershed. The plan identifies environmental issues; assesses water quality potential; establishes watershed objectives related to surface water and groundwater quality, aquatic and terrestrial habitat, and recreation; and sets forth implementation procedures to accomplish broadly defined management strategies and recommendations. The portion of the nonpoint source plan dealing with Lilly Creek identifies water resources objectives, quantifies existing and anticipated future nonpoint source pollutant loadings, and recommends general programs for the control of nonpoint source pollution within the subwatershed.

SUMMARY

The Lilly Creek subwatershed is located within the Menomonee River watershed in the Village of Menomonee Falls in northeastern Waukesha County. The anticipated rapid conversion of land from rural to urban use in this subwatershed may be expected to aggravate existing stormwater management and flood control problems and, in the absence of sound planning, to create costly new problems. The need to resolve existing problems and to avoid the occurrence of new problems dictates the need to prepare a long-range stormwater management and flood control plan for the Lilly Creek subwatershed.

This report represents such a stormwater management and flood control plan. The plan seeks to promote the development of an effective stormwater management and flood control system for planned ultimate development conditions in the study area, a system that will minimize damages attendant to poor drainage while reducing downstream flooding, and that will protect and enhance surface water quality.

More specifically, this report describes the existing stormwater drainage system and the existing stormwater management and flood control problems of the Lilly Creek subwatershed; identifies the causes of these problems; describes existing and planned future land use conditions and identifies related stormwater management and flood control requirements; provides a set of objectives and supporting standards to guide the development of an effective stormwater management and flood control system for the subwatershed; presents alternative stormwater management and flood control system plans for the subwatershed; provides a comparative evaluation of the technical, economic, and environmental features of these plans; recommends a cost-effective stormwater management and flood control plan; and sets forth a plan implementation program.

The plan recognizes that good stormwater and floodland management is essential to the provision of an attractive and efficient, as well as safe and healthful, environment for urban life; and that inadequate stormwater and floodland management can be costly and disruptive, can create hazards to public health and safety, and can have adverse ecological and environmental impacts. Because of the technical complexity of the problems and the important social, economic, and environmental impacts involved, the plan recognizes that stormwater management and flood control planning must be based upon knowledge of the art of urban engineering and of the sciences of hydrology and hydraulics; an understanding of the social, economic, and environmental impacts involved; and information on the public attitudes toward stormwater management and flood control.

The recommended stormwater management and flood control plan presented herein also recognizes that the basic concept underlying urban stormwater management is undergoing reexamination. The old concept sought to eliminate excess surface water during and after a rainfall as quickly as possible through the provision of an efficient drainage system, a system consisting of enclosed conduits and improved open channels. The new concept emphasizes storage, as well as conveyance of runoff, with the objectives of reducing the peak rate of runoff, and, in some cases, the total volume of runoff; reducing the transport of sediment and other water pollutants to downstream surface waters; and protecting against increased downstream flooding. The new concept also looks to controlling the quality, as well as the quantity, of runoff.

The plan presented herein regards the stormwater runoff system of the area as consisting of a major element operating infrequently and a minor element operating frequently, with both of these elements incorporating, to the extent practicable, the storage as well as conveyance of excess runoff. The recommended stormwater management and flood control plan set forth herein thus incorporates compatible multi-use planning concepts, and recognizes the opportunities provided and the constraints imposed by other community needs, such as park and open space, transportation, and water supply. Stormwater management and flood control requirements are evaluated under both existing and planned future land use conditions.

SEWRPC Planning Report No. 26, A Comprehensive Plan for the Menomonee River Watershed, October 1976, recommended channel modifications and bridge replacements to solve existing and anticipated flooding problems along Lilly Creek. The recommended flood control plan was refined and integrated with a proposed stormwater management system in the Lilly Creek Stormwater Management Plan-Village of Menomonee Falls, October 1984, prepared for the Village by Ruekert & Mielke, Inc., consulting engineers. Modifications to that plan were proposed to meet concerns regarding water quality and fish and wildlife habitat raised by the Wisconsin Department of Natural Resources in considering the Village's permit applications for the work necessary to implement the plan. The Village and the DNR were unable to reach agreement on an alternative plan, and the issue was pursued further in a contested case hearing, in Waukesha County Circuit Court, in the State Court of Appeals, and finally, in the State Supreme Court. The position of the DNR was upheld in each step of the legal process, and the proposed channel modification and bridge replacement project was not constructed. The plan presented herein is developed for the purpose of resolving the differences between the Village and the DNR through the consideration of all practical alternatives for alleviating existing and anticipated future stormwater management and flood control problems in the Lilly Creek subwatershed.

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Chapter II

INVENTORY AND ANALYSIS

INTRODUCTION

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

Accordingly, this chapter presents data on the hydrologic phenomena governing the magnitude and frequency of stormwater flows, on existing stormwater drainage and flood control problems. on the anticipated type, density, and spatial distribution of land uses in the study area, and on the impact of the anticipated changes in land use on the stormwater management needs of the study area. Because water quality impacts are becoming increasingly of concern in stormwater management, this chapter also presents data on surface water quality conditions in the Lilly Creek subwatershed and identifies those sources of pollution related to stormwater management.

STORMWATER MANAGEMENT STUDY AREA

The Lilly Creek subwatershed constitutes the study area for stormwater management planning as shown on Map 1. The total areal extent of the subwatershed is approximately 5.65 square miles, all of which is contained within the Village of Menomonee Falls.

Selected characteristics of the surface water drainage system of the Lilly Creek subwatershed, including subwatershed and floodplain boundaries, are also shown on Map 1.

LAND USE

The Lilly Creek stormwater management and flood control plan is intended to identify the stormwater management and flood control needs of the Lilly Creek subwatershed under existing and planned land use conditions and to propose the best means of meeting those needs. Accordingly, a design year 2010 land use pattern was developed for the subwatershed, based upon a design year 2010 land use plan for the Village. Such a plan has been prepared for the Village by the Regional Planning Commission under a separate but coordinated planning effort.[1]

The land use plan identifies certain areas within the Lilly Creek subwatershed which are planned for industrial development after the year 2010, as well as a recommended land use pattern for the design year 2010 which can accommodate a resident population of about 18,000 persons in the Lilly Creek subwatershed. This stormwater management and flood control plan is based upon the land use plan, but also incorporates the areas planned for industrial development after the year 2010. Therefore, the stormwater management and flood control plan is essentially based on planned ultimate land use conditions within the subwatershed. The use of planned ultimate land use conditions is appropriate for stormwater management and flood control planning because it helps to ensure that components of the system are adequately sized for any increased hydraulic and pollutant loadings which would occur as upstream tributary areas are developed.

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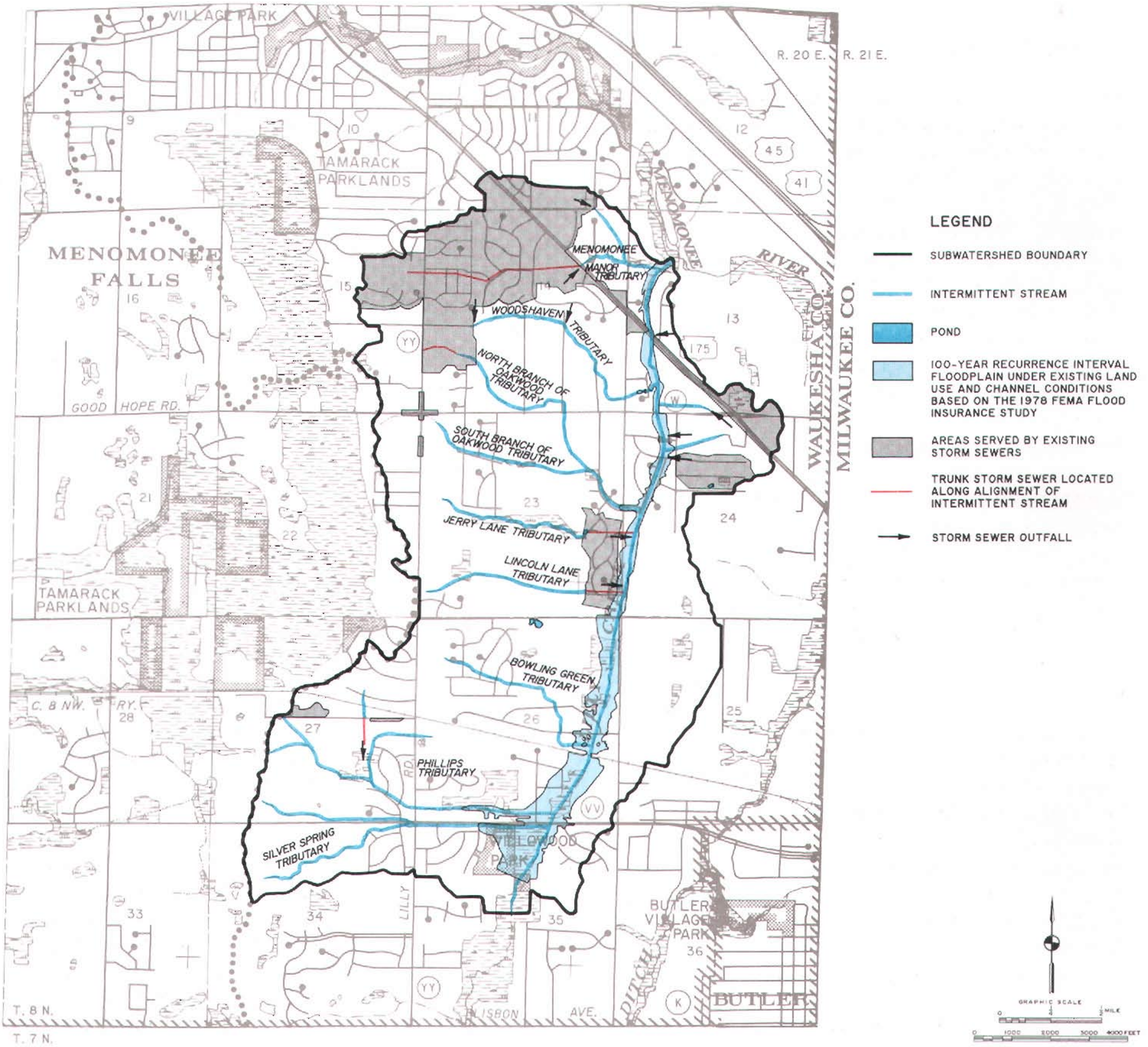
[1] *SEWRPC Community Assistance Planning Report No. 162, A Land Use and Transportation System Plan for the Village of Menomonee Falls: 2010, April 1990.*

Map 1

SELECTED CHARACTERISTICS OF THE LILLY CREEK SUBWATERSHED SURFACE WATER DRAINAGE SYSTEM: 1988

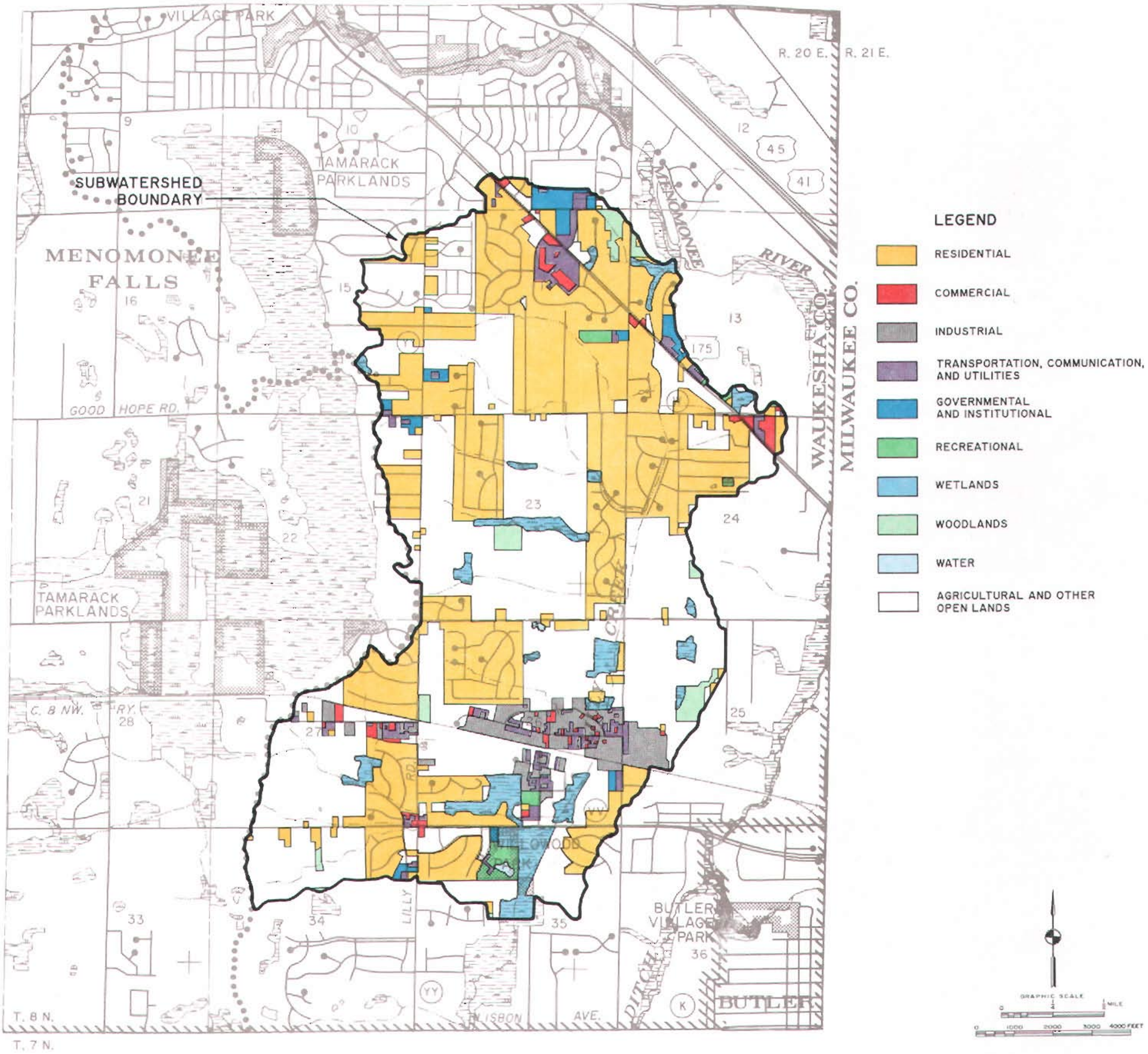


Source: Village of Menomonee Falls and SEWRPC.

The land use changes expected to occur within the subwatershed are, in part, the anticipated result of an aggressive Village development program. This program includes the establishment of a tax incremental finance district to fund and support, through public infrastructure development, desired land use development and redevelopment, the aggressive pursuit of state economic grants and loans from the Wisconsin Development Fund, the issuance of private activity bonds, and the provision by the Village of services encouraging development. This village development program gives impetus to the need to develop a stormwater management and flood control system plan for the Lilly Creek subwatershed.

Map 2

EXISTING LAND USE IN THE LILLY CREEK SUBWATERSHED: 1985



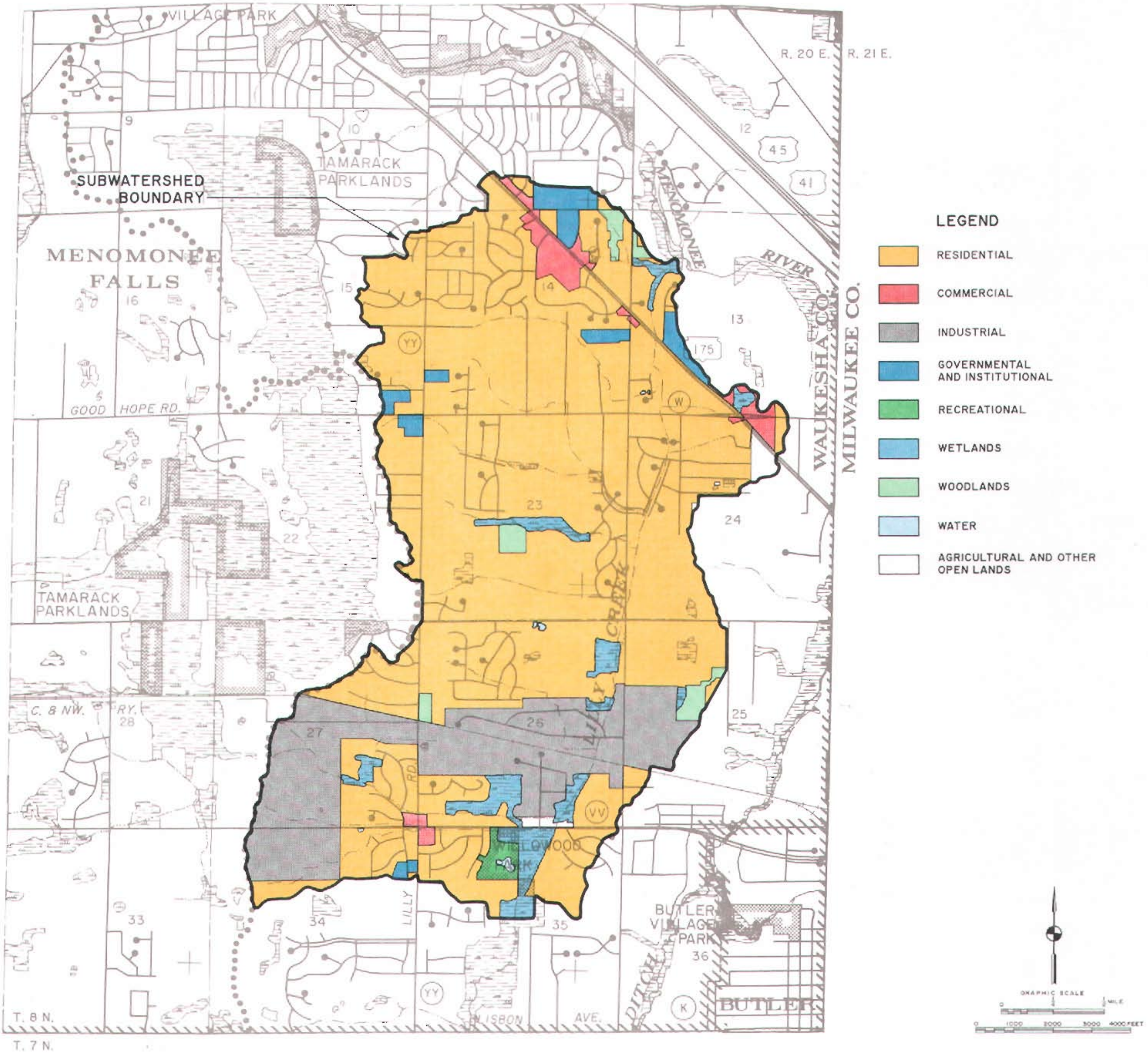
Source: SEWRPC.

Significant development is also anticipated because of the recent extension of public water and sanitary sewer service to the Lilly Creek subwatershed. The extension of sanitary sewer service will enable development to occur in areas of the subwatershed which have soils that are limited for the construction of private sewage treatment systems, a heretofore constraint on development within the subwatershed.

The subwatershed encompasses a total area of about 3,615 acres, or about 5.65 square miles. The existing year 1985 land use pattern is shown on Map 2. The planned ultimate land use pattern

Map 3

PLANNED ULTIMATE LAND USE IN THE LILLY CREEK SUBWATERSHED



Source: SEWRPC.

is shown on Map 3. The areal extent of the various existing and planned land uses within the subwatershed are set forth in Table 4. As indicated in that table, about 1,620 acres of rural land, or about 45 percent of the total subwatershed area, may be expected to be converted from rural to urban uses over the plan design period. This conversion would increase the amount of land in urban use within the subwatershed by about 89 percent. Of the total area to be converted, about 885 acres, or 55 percent, would be converted to residential use; about 319 acres, or 20 percent, to industrial use; and about 408 acres, or 25 percent, to other urban uses.

Table 4

EXISTING AND PROBABLE FUTURE LAND USE IN THE LILLY CREEK SUBWATERSHED: 1985 AND ULTIMATE

Land Use Category	Existing 1985		Planned Increment		Ultimate Total	
	Acres	Percent of Total	Acres	Percent Change	Acres	Percent of Total
Urban						
Residential	1,212	33.4	885	73.0	2,097	57.9
Commercial	35	1.0	40[a]	114.3[a]	75[a]	2.2
Industrial	109	3.0	319	292.7	428	11.8
Governmental and Institutional	44	1.2	61	138.6	105	2.9
Transportation, Communication and Utilities	391	10.8	284	72.6	675	18.6
Recreational	29	0.8	23	79.3	52	1.5
Subtotal	1,820	50.2	1,612[a]	88.8	3,432[a]	94.9
Rural						
Woodlands	49	1.4	-8	-16.3	41	1.1
Wetlands	156	4.3	-29[a]	-19.0[a]	127[a]	3.6
Surface Water	4	0.1	- -	- -	4	0.1
Agricultural and Other Open Lands	1,595	44.0	-1,584	-99.3	11	0.3
Subtotal	1,804	49.8	-1,621[a]	-89.9[a]	183[a]	5.1
Total	3,624	100.0	- -	- -	3,615[a]	100.0

[a] *Due to grading for a commercial development which was constructed in 1987, nine acres of land located in catchment area LCM04 (see Map 8 in Chapter V of this report) northeast of the intersection of W. Appleton Avenue and W. Good Hope Road were removed from the subwatershed and transferred into the Menomonee River direct drainage area. Those nine acres, apportioned between wetland and commercial land use categories, are subtracted from the planned increments and ultimate totals in this table. As a result of that subtraction, the incremental rural planned loss is nine acres larger than the incremental urban planned gain since all of the area removed from the subwatershed was in rural uses in 1985.*

Source: SEWRPC.

As indicated in Table 4, under planned ultimate land use conditions urban land uses would occupy about 3,432 acres, or about 95 percent of the total area of the subwatershed area. Residential uses would occupy about 2,097 acres, or about 58 percent of the subwatershed area; the remaining urban land uses, such as commercial, industrial, transportation, communication and utilities, governmental and institutional, and recreational, would occupy 37 percent. Under planned ultimate land use conditions, rural land uses would still be expected to account for about 183 acres, or about 5 percent of the total area of the subwatershed area. Woodlands would occupy about 41 acres of that total, or about 1 percent; agricultural and other open lands about 11 acres, or less than 1 percent; and other rural land uses, including wetlands and open water, about 131 acres, or about 4 percent.

Because of the direct relationships which exist between resident population levels and land use patterns, an evaluation of the historic and probable future resident population levels in the Lilly Creek subwatershed was made as a part of the stormwater management and flood control planning effort. As indicated in Table 5, from 1963 to 1970 the resident population of the subwatershed increased by about 16 percent, from about 6,300 to about 7,300 persons. From 1970 to 1985, the resident population of the subwatershed decreased by about 19 percent, to

Table 5

HISTORIC AND PROBABLE FUTURE RESIDENT POPULATION LEVELS FOR THE SOUTHEASTERN WISCONSIN REGION, WAUKESHA COUNTY, AND THE LILLY CREEK SUBWATERSHED

	Southeastern Wisconsin Region		Waukesha County		Village of Menomonee Falls		Lilly Creek Subwatershed	
Year	Population	Percent Change	Population	Percent Change	Population	Percent Change	Population	Percent
1900	501,808	- -	35,229	- -	678[a]	- -	- -	- -
1910	631,161	25.8	37,100	5.3	919	33.8	- -	- -
1920	783,681	24.2	42,612	14.9	1,019	10.9	- -	- -
1930	1,006,118	28.4	52,358	22.9	1,291	26.7	- -	- -
1940	1,067,699	6.1	62,744	19.8	1,469	13.8	- -	- -
1950	1,240,618	16.2	85,901	36.9	2,469	68.1	- -	- -
1960	1,573,614	26.8	158,249	84.2	18,276[b]	640.2	6,300[c]	- -
1970	1,756,083	11.6	231,335	46.2	31,697	73.4	7,290	15.7
1980	1,764,919	0.5	280,326	21.2	27,845	-12.2	6,170	-15.4
1985	1,742,742	-1.3	285,904	2.0	27,039	-2.9	5,900	-4.4
2010	1,872,200[d]	7.4	364,300	27.4	47,800	76.8	17,800	201.7

[a] *The Village of Menomonee Falls was incorporated in 1892.*

[b] *In 1958 the remaining territory of the Town of Menomonee Falls was annexed by the Village of Menomonee Falls and the Town of Menomonee Falls ceased to exist. Population totals represent the entire incorporated area of the Village of Menomonee Falls.*

[c] *Represents 1963 population levels as determined by the 1963 SEWRPC origin-destination travel survey.*

[d] *Intermediate population growth scenario.*

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

5,900 persons. Optimistic forecasts of population growth to the year 2010 indicate that the population of the subwatershed may be expected to again increase to about 17,800 persons, an increase of about 11,900 persons, or about 200 percent, over the 1985 population level. This large population increase is anticipated due to the recent extension of public water and sanitary sewer service to the subwatershed. A graphic comparison of historical, existing, and forecast population levels for the subwatershed, Waukesha County, and the Southeastern Wisconsin Region is provided in Figure 1. The anticipated increase in population within the subwatershed can readily be accommodated by the increase in residential land use anticipated in the land use plan for the subwatershed.

Within the subwatershed, the planned year 2010 resident population level of about 17,800 persons, assuming a household size of 2.7 persons per housing unit, would result in the need for approximately 6,590 housing units. Such housing units, if uniformly distributed over the 2,097 acres of residential land anticipated to be within the subwatershed by the design year 2010, would result in a density of approximately 3.1 housing units per net residential acre.

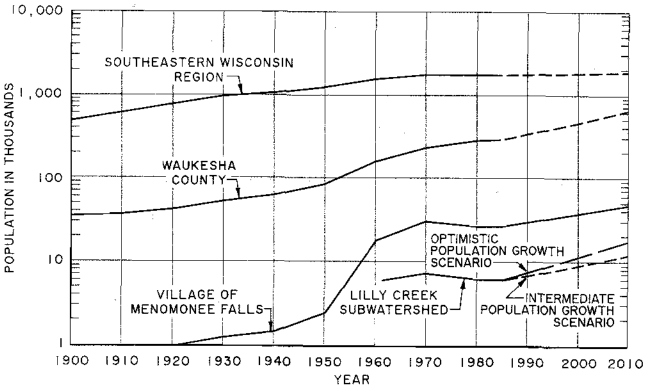
LAND USE REGULATIONS

Pertinent land use regulations in the subwatershed include zoning and land subdivision control ordinances. Comprehensive zoning represents one of the most important tools available to local units of government for controlling the use of land in the public interest, and such zoning has important implications for stormwater management.

The current Village of Menomonee Falls zoning ordinance provides for eight residential districts,

Figure 1

COMPARISON OF HISTORICAL, EXISTING, AND FORECAST POPULATION TRENDS FOR LILLY CREEK SUBWATERSHED, WAUKESHA COUNTY, AND THE SOUTHEASTERN WISCONSIN REGION



Source: SEWRPC.

two business districts, one commercial district, two industrial districts, two public districts, one agricultural district, two floodplain districts, one conservancy-wetlands district, one park and open space district, one institutional district, and one planned unit development district. Each district includes adjoining streets, and all zoning districts except the agricultural district may be expected to be applied in the subwatershed to attain the land use pattern envisioned in the adopted land use plan.

The subdivision and development for urban use of land within the Village of Menomonee Falls is regulated by the village land subdivision control ordinance. The ordinance requires that preliminary and final subdivision plats be filed for all divisions of land which would create five or more parcels of land 1.5 acres or less in area, or would do so by successive division within a period of five years. It also requires the subdivider to make improvements, including street pavements and surface water drainage facilities, to village specifications prior to final plat approval. The ordinance encourages installation of urban street cross-sections with curb and gutter and storm sewers, but permits installation of alternative rural street cross-sections with road ditches for drainage if approved by the Village Engineer.

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

IMPACT OF CHANGING LAND USE ON SUBWATERSHED STORMWATER MANAGEMENT SYSTEMS

Land use and cover in the study area markedly influence the stormwater runoff process. Land cover differs from land use in that it describes a type of surface: roofed, paved, grassed, or wooded, for example; land use describes the function or activity served: residential, commercial, or recreational, for example. The conversion of land from rural to urban use and the associated increase in impervious areas increases both the rate and volume of stormwater runoff for a given rainfall event and decrease the time of runoff. Such increases in rates and volumes of runoff can increase bank erosion and bed scour in receiving streams. In addition, increased imperviousness in areas of groundwater recharge may cause a reduction in stream base flow. Stormwater runoff from urban lands also carries different types and increased amounts of pollutants compared to runoff from rural lands.

The stormwater management and flood control system of a watershed should serve to support the existing, and promote the planned, land use pattern of the watershed. Therefore, consideration of both the existing and probable land use pattern of the watershed is necessary for the development of effective alternative stormwater management and flood control plans and for the selection of a recommended plan.

Table 6

RANGE OF SURFACE IMPERVIOUSNESS FOR LAND USE AND LAND COVER CONDITIONS

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

Source: SEWRPC.

As already noted, the conversion of rural land to urban uses in the Lilly Creek subwatershed may be expected to result in about 3,430 acres, or about 95 percent of the subwatershed, being devoted to urban land uses under planned ultimate land use conditions. This compares to the 1,820 acres, or 50 percent of the subwatershed, in urban land uses under existing 1985 conditions. This is an increase of approximately 88 percent in the amount of land in urban use over the plan design period. This change in land use may be expected to have a direct impact upon the quality, amount, and rate of stormwater runoff.

The percent of impervious surfaces in a watershed is an important factor in determining both the amount of stormwater runoff and the rate at which stormwater runoff is generated. Table 6 lists the ranges of surface imperviousness for various land use and land cover conditions. As indicated in that table, more than 65 percent of the total area of industrial and commercial areas may consist of impervious surfaces, while from 10 to 65 percent of the total area of residential areas may consist of impervious surfaces, depending on the density of the development. Generally, less than 10 percent of the total area of rural areas consists of impervious surfaces. The impact of the planned changes in land use on the volume and rate of stormwater runoff from each of the drainage subbasins within the Lilly Creek subwatershed is estimated in Chapters V and VI of this report.

CLIMATE

Air temperatures and the type, intensity, and duration of precipitation affect the extent of areas subject to inundation and the type and magnitude of stormwater and flood control problems within the subwatershed. The subwatershed has the typical continental-type climate, characterized primarily by a continuous progression of markedly different seasons and a wide range in monthly temperatures. The subwatershed lies in the path of both low pressure storm centers moving from the west and southwest and high pressure fair weather centers moving in a generally southeasterly direction. The confluence of these air masses results in frequent weather changes, particularly during spring and winter. These temporal weather changes consist of marked variations in temperature, precipitation, relative humidity, wind

Table 7

AVERAGE MONTHLY AIR TEMPERATURE AT MILWAUKEE: 1951 THROUGH 1985

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

Source: National Weather Service and SEWRPC.

Table 8

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

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

Source: National Weather Service and SEWRPC.

speed and direction, and cloud cover. The meteorologic events influence the rate and amount of stormwater runoff, the severity of storm drainage problems, and the required capacities of stormwater conveyance and storage facilities. Definitive, long-term meteorologic data are available for the Milwaukee National Weather Service station, located at Mitchell International Airport, in reasonable proximity to the Lilly Creek subwatershed.

Temperature and Seasonal Considerations

Air temperatures, which exhibit a wide monthly range, determine whether precipitation occurs as rainfall or snowfall; whether or not the ground is frozen and therefore essentially impervious; and the rate of snowmelt and attendant runoff. Table 7 presents average monthly air temperature variations at the Milwaukee National Weather Service Station for the 35-year period from 1951 through 1985. Summer temperatures, as measured by the monthly means for June, July, and August, average from 65°F to 70°F. Winter temperatures, as measured by the monthly means for December, January, and February, average from 19°F to 25°F. For the period 1871 through 1988 at Milwaukee, the maximum recorded temperature was 105°F in July 1934, and the lowest recorded temperature was -26°F in January 1982. The growing season, which is defined as the number of days between the last 32°F temperature reading in spring and the first in fall, averages about 180 days for the subwatershed. The last frost in spring normally occurs near the end of April, whereas the first freeze in fall usually occurs during the latter half of October. Streams and lakes begin to freeze over in late November; ice breakup usually occurs in late March or early April. Ice jams at bridges in spring can be a cause of localized flooding, which can be severe when combined with spring rainfall.

Precipitation

Precipitation within the subwatershed takes the form of rain, sleet, hail, and snow, ranging from gentle showers of trace quantities to brief, but intense and potentially destructive, thunderstorms or major rainfall-snowmelt events. These may cause property damage, inundation of poorly drained areas, stream flooding, street and basement flooding, and severe soil erosion and sedimentation. Average monthly and annual total precipitation and snowfall data from the Milwaukee National Weather Service station at Mitchell International Airport for the period 1951 through 1985 are presented in Table 8. The average annual total precipitation in the Lilly

Table 9

EXTREME PRECIPITATION PERIODS IN SOUTHEASTERN WISCONSIN: SELECTED YEARS, 1870 THROUGH 1985

Observation Station		Period of Precipitation Records, Except Where Indicated Otherwise	Total Precipitation						
			Maximum Annual		Minimum Annual		Maximum Monthly		
Name	County		Amount	Year	Amount	Year	Amount	Month	Year
Mitchell Field	Milwaukee	1870-1986	50.36[a]	1876	18.69[a]	1901	10.03	June	1917
Racine	Racine	1895-1986	48.33	1954	17.75	1910	10.98	May	1933
Waukesha	Waukesha	1982-1986	43.57	1938	17.30	1901	11.41	July	1952
West Bend	Washington	1922-1986	41.43	1984	19.72	1901	13.14[b]	August	1924
West Allis	Milwaukee	1954-1986	42.85	1960	17.49	1963	9.63	June	1954
Mt. Mary College	Milwaukee	1954-1986	41.25	1965	18.50	1963	10.17	June	1968

[a] *Based on the period 1941 through 1986.*

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

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

Creek subwatershed based on the Milwaukee National Weather Service station data is 31.81 inches, expressed as water equivalent, while the average annual snowfall and sleetfall measured as snow and sleet is 50.2 inches. Assuming that 10 inches of measured snowfall and sleetfall are equivalent to one inch of water, the average annual snowfall of 50.2 inches is equivalent to 5.02 inches of water and, therefore, only about 16 percent of the average annual total precipitation occurs as snowfall and sleet. Average total monthly precipitation ranges from 1.39 inches in February to 3.49 inches in April. The principal snowfall months are December, January, February, and March, during which 89 percent of the average annual snowfall may be expected to occur.

An important consideration in stormwater drainage is the seasonal nature of precipitation patterns. Based on historical observations, flooding in the Lilly Creek subwatershed is likely to occur at any time throughout the year except during winter. This is because the drainage area is relatively small and flood peaks are influenced by the effects of poorly drained soils and urban development. The relatively large proportions of poorly to very poorly drained soils, along with impervious surfaces in urban areas, inhibit infiltration. This significantly increases surface runoff during even minor rainfall events. Because the dampening effects of infiltration, including leaf interception during summer months, are diminished in urban areas, the annual distribution of flood events in urbanized watersheds is similar to the annual distribution of significant rainfall events, and significant flood events may be expected to occur during spring, summer, and fall.

Extreme precipitation data for southeastern Wisconsin, based on observations for stations located throughout the Region that have relatively long periods of record, are presented in Table 9. The minimum annual precipitation within southeastern Wisconsin, as determined from the tabulated data for the indicated observation period, occurred at Waukesha in 1901, when only 17.30 inches of precipitation occurred, or 55 percent of the average annual precipitation of 31.30 inches for southeastern Wisconsin. The maximum annual precipitation within southeastern Wisconsin occurred at Milwaukee in 1876, when 50.36 inches of precipitation was recorded, equivalent to 161 percent of the average annual precipitation.

Based on a period of record from 1870 through 1986 at General Mitchell Field, the minimum annual precipitation was 18.69 inches, reported in 1901; the maximum annual precipitation was 50.36 inches, reported in 1876. The maximum monthly precipitation was 10.03 inches, recorded in June 1917; the maximum 24-hour precipitation was 6.84 inches, recorded on August 6, 1986. Based on a period of record from 1940 through 1980, the maximum and minimum annual snowfall amounts were 90.8 inches in 1951 to 1952 and 12.1 inches in 1967 to 1968.

Stormwater management and flood control system design must also consider the characteristics of rainfall events for periods of time substantially shorter than 24 hours. The characteristics of rainfall events over these shorter peak precipitation periods are discussed in Chapter IV of this report.

Snow Cover and Frost Depth

The likelihood of snow cover and the depth of snow on the ground are important precipitation-related factors that influence the planning, design, construction, and maintenance of stormwater management and flood control facilities. Snow cover in the Lilly Creek subwatershed is most likely during the months of December, January, and February, when at least a 0.5 probability exists of having one inch or more of snow cover. The amount of snow cover influences the severity of spring snowmelt-rainfall flood events, which usually occur during March.

The depth and duration of ground frost, or frozen ground, influences hydrologic processes, particularly such factors as the proportion of rainfall or snowmelt that will run off the land directly into storm sewerage systems and surface water courses. The amount of snow cover is an important determinant of frost depth. Since the thermal conductivity of snow cover is less than one-fifth that of moist soil, heat loss from the soil to the colder atmosphere is greatly inhibited by the insulating snow cover. Frozen ground is likely to exist throughout the study area for approximately four months each winter season, from late November through March, with frost penetration to a depth ranging from six inches to more than four feet occurring in January, February, and the first half of March.

SOILS

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

In order to assess the significance of the diverse soils found in southeastern Wisconsin, the Southeastern Wisconsin Regional Planning Commission negotiated a cooperative agreement with the U. S. Soil Conservation Service in 1963 under which detailed operational soil surveys were completed for the entire Region. The results of the soil surveys have been published in SEWRPC Planning Report No. 8, Soils of Southeastern Wisconsin. The regional soil surveys have resulted in the mapping the Region's soils in great detail. At the same time, the surveys have provided data on the physical, chemical, and biological properties of the soils, and, more importantly, have provided interpretations of the soil properties for planning, engineering, agricultural, and resource conservation purposes, and for underlying stormwater management purposes. Detailed soils maps of the study area are available for use in stormwater management planning.

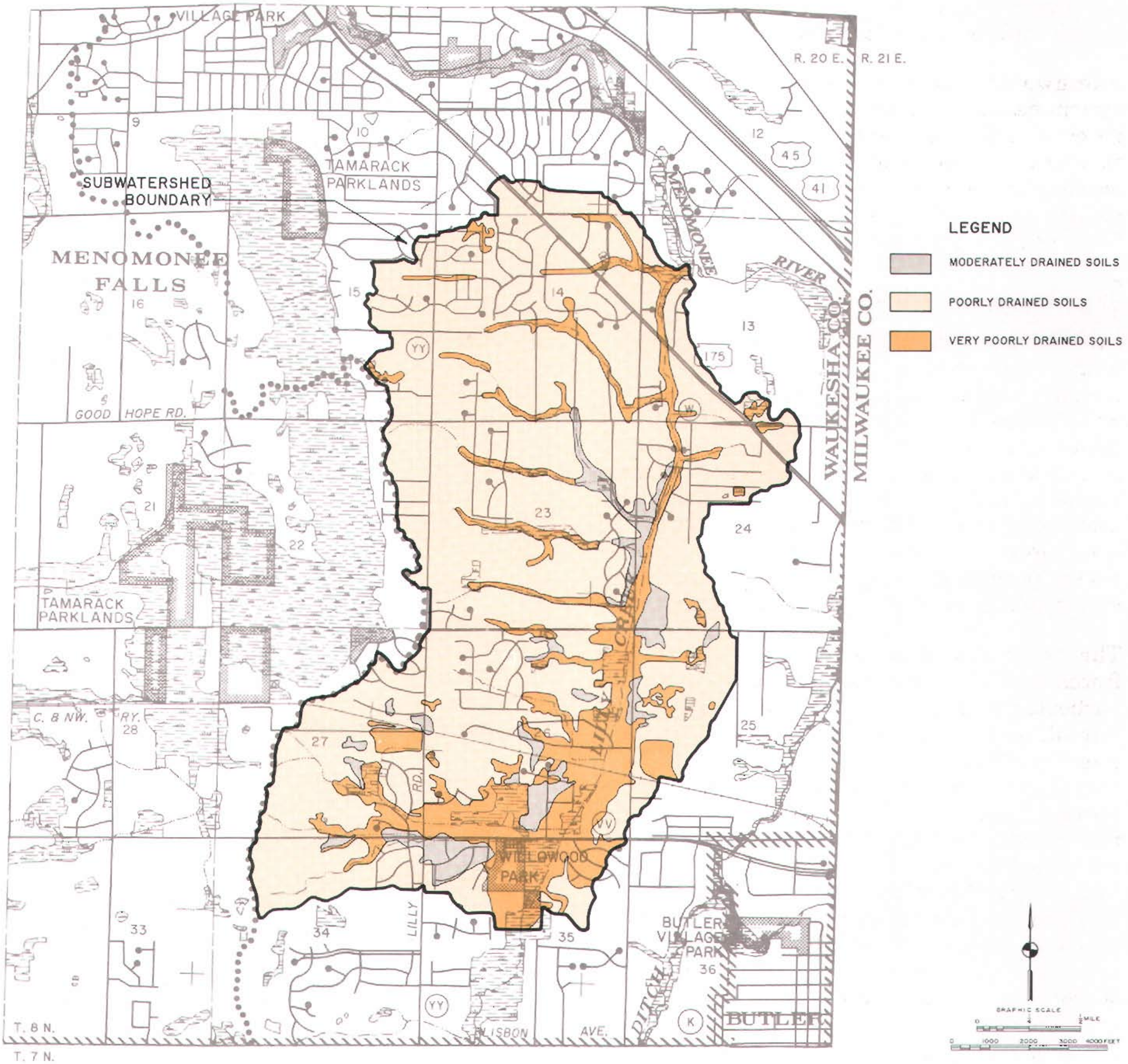
With respect to watershed hydrology, the most significant soil interpretation for stormwater management is the categorization of soils into hydrologic soil groups A, B, C, and D. In terms of runoff characteristics, these four hydrologic soil groups are defined as follows:

- Hydrologic Soil Group A: Very little runoff because of high infiltration capacity, high permeability, and good drainage.
- Hydrologic Soil Group B: Moderate amounts of runoff because of moderate infiltration capacity, moderate permeability, and good drainage.
- Hydrologic Soil Group C: Large amounts of runoff because of low infiltration capacity, low permeability, and poor drainage.
- Hydrologic Soil Group D: Very large amounts of runoff because of very low infiltration capacity, low permeability, and extremely poor drainage.

The spatial distribution of the four hydrologic soil groups within the Lilly Creek subwatershed is shown on Map 4. Hydrologic soil groups B, C, and D comprise 5 percent, 75 percent, and 20 percent, respectively, of the study area. Some 95 percent

Map 4

HYDROLOGIC SOIL GROUPS WITHIN THE LILLY CREEK SUBWATERSHED



Source: SEWRPC.

of the study area is covered by soils having poor or very poor drainage characteristics, which, therefore, may be expected to generate relatively large amounts of stormwater runoff.

BEDROCK

Bedrock formations underlying the study area generally lie at a depth of 40 to 180 feet below the surface of the Lilly Creek subwatershed, with overlying unconsolidated glacial deposits. In a localized area near the southern boundary of the subwatershed, bedrock is generally located within a few feet of the ground surface. It is not anticipated that bedrock would be encountered during construction of stormwater management facilities.

STORMWATER MANAGEMENT AND FLOOD CONTROL SYSTEM

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

Topography

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

Large-scale topographic maps of the entire Village of Menomonee Falls were prepared in 1987 and 1988 by Waukesha County and the Regional Planning Commission to Commission specifications at a scale of one inch equals 100 feet with contours at two-foot intervals. Those maps were utilized in the system engineering for the plan documented in this report. The large-scale topographic maps and monumented control survey network which resulted from that mapping program will also have permanent utility for the administration of the federal flood insurance program at the local level and for all types of municipal planning and engineering work.

The elevation of the Lilly Creek subwatershed ranges from a low of about 744 feet above National Geodetic Vertical Datum (NGVD) in the northwest one-quarter of U. S. Public Land Survey Section 13, Township 8 North, Range 20 East, at the confluence of Lilly Creek and the Menomonee River, to a high of about 911 feet NGVD in the northwest one-quarter of U. S. Public Land Survey Section 34, Township 8 North, Range 20 East. Land surface slopes range from a low of less than 1 percent for a portion of a drainage area located in the northeast one-quarter of U. S. Public Land Survey Section 23, Township 8 North, Range 20 East, to a high of about 15 percent for a portion of a drainage area located in the northeast one-quarter of U. S. Public Land Survey Section 27, Township 8 North, Range 20 East. In general, areas with slopes greater than 12 percent have severe limitations for urban residential development and, if developed, present serious potential drainage and erosion problems.

Catchment Areas and Subbasins

For stormwater management planning purposes, the Lilly Creek subwatershed was divided into smaller basic hydrologic units called catchment areas. The catchment areas were aggregated into subbasins, with each subbasin generally encompassing the area draining to one of the streams tributary to Lilly Creek, or the area draining to a storm sewer outfall to the Creek. The catchment area and subbasin boundaries are shown on Map 8 of Chapter V. The delineation of these catchment areas and subbasins permits a more accurate representation of the watershed hydrology in the computer models used to simulate stormwater runoff.

A number of considerations entered into the delineation of the catchment areas. Using the available large-scale topographic maps prepared to Commission standards in 1987 and 1988, the catchment areas were delineated so as to provide desired areas above discharge points at confluences of drainage channels, tributaries, and the main stem; at, or near, bridges and culverts; and at selected storm sewer inlets and outlets.

Within the total study area, there are 199 catchment areas, which range in size from about one to 64 acres, with an average size of 18 acres.

Streams, Drainage Channels, Storm Sewers, and Ponds

Perennial streams are watercourses which maintain a continuous flow throughout the year. Intermittent streams are those watercourses which do not sustain continuous flow during dry periods.

The Lilly Creek subwatershed contains no perennial streams. The intermittent streams in the subwatershed serve as the major drainage outlets for the storm sewers and drainage ditches. The intermittent streams are important components of the drainage system and they must be characterized in order to properly plan a stormwater management and flood control system. All known intermittent streams and ponds in the study area are shown on Map 1.

Table 10

CHARACTERISTICS OF STORM SEWER SYSTEMS WITHIN THE LILLY CREEK SUBWATERSHED

Subwatershed	Tributary Area (acres)	Length of Storm Sewer (feet)	Range of Storm Sewer Sizes (inches)	Range of Storm Sewer Slopes (ft/ft)
Lilly Creek	510	45,330	10 to 66 reinforced concrete pipe and corrugated metal pipe	0.0010-0.0540

Source: SEWRPC.

The network of intermittent streams serves a vital function by providing natural drainage for those areas not drained by engineered stormwater drainage facilities, and by receiving the discharge of the engineered stormwater drainage facilities. The Lilly Creek subwatershed contains 15.54 miles of intermittent streams, of which 12.04 miles are streams that are tributary to Lilly Creek. Lilly Creek accounts for the remaining 3.50 miles, or 23 percent of the total. There are three ponds, but no lakes, in the subwatershed.

Engineered stormwater drainage facilities within the subwatershed as of 1990, defined as constructed channels or roadside swales, storm sewers, and appurtenances, as opposed to natural watercourses, had a combined service area of about 1,655 acres, or 45 percent of the total subwatershed. About 525 acres, or 32 percent of the total area served by engineered stormwater drainage facilities, were tributary to drainage systems relying primarily on storm sewers for conveyance. The remaining 1,130 acres, or 68 percent, were tributary to drainage systems relying primarily on open drainage channels and associated culverts.

The portions of the study area served by storm sewers comprise 15 percent of the subwatershed area. The existing storm sewer system serves areas ranging in size from about 13 to 160 acres. As shown in Table 10, the total length of existing storm sewers in the study area is about 45,330 feet, or 8.6 miles. The slopes of the sewers range from 0.001 foot per foot to 0.054 foot per foot. The storm sewer systems are maintained by the Public Works Department of the Village of Menomonee Falls. Maintenance activities include sewer inspection; sewer, culvert, catch basin, and channel cleaning; and minor repair work on sewers, manholes, catch basins, and inlets.

Since May 1987, the Village has enforced a set of stormwater management guidelines requiring onsite detention and runoff control for certain new urban developments. Following adoption of this stormwater management and flood control plan, the Village intends to discontinue enforcement of the guidelines within the Lilly Creek subwatershed and to apply the systems plan recommended herein.

The guidelines are applied to four development categories: 1) residential developments with a gross aggregate area of five acres or more; 2) residential developments of from three to five acres with 50 percent or more impervious area; 3) developments other than residential with a gross aggregate area of three acres or more; and 4) developments which, in the opinion of the Village, would create flows which would cause downstream flooding damages, erosion, water pollution, or would otherwise endanger downstream property owners or property. Developments within the above categories are required to provide onsite detention storage to limit peak post-development runoff from the site for a 100-year recurrence interval rainfall event of any duration to the peak flow resulting from a five-year recurrence interval rainfall with the site in its undeveloped condition.

Estimates of the peak flows discharged from the existing engineered drainage system to receiving streams are set forth in Chapter V of this report. A description of the design rainfall recurrence interval used to estimate those flows is presented in Chapter IV.

Wetlands

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

A sound stormwater management plan should, to the extent practicable, utilize the stormwater storage capacity of any existing natural wetlands, while preserving the quality of the wetlands. Thus, wetland preservation should be an integral part of a stormwater management plan. Wetlands in the study area were identified in a special inventory conducted by the Commission using aerial photographic interpretation and field inspection supplemented by analysis of mapped soil data. The location and extent of wetlands in the subwatershed are shown on Map 2 and quantified in Table 4. In 1985, there were approximately 156 acres of wetlands in the subwatershed, comprising about 4.3 percent of the area.

Bridges, Culverts, and Other Structures

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

Map 5 shows the location of bridges and culverts in the subwatershed. Table 11 provides information on the size and types of bridges and culverts along Lilly Creek and its tributaries.

Flood Discharges and Natural Floodlands

As stated in Chapter I of this report, a flood insurance study was prepared for the Lilly Creek subwatershed by the Federal Emergency Management Agency as documented in the Flood Insurance Study for the Village of Menomonee Falls, Waukesha County, Wisconsin, July 1978.

The flood flows developed for that study and presented in Chapter I of this report were reviewed in conjunction with the preparation of estimated flows in the existing stormwater drainage system under this study. Chapters V and VI of this report present refined estimates of the flood flows under existing and planned land use and channel conditions.

The federal flood insurance study report includes flood insurance rate maps which show the expected surface elevations of the base 100-year flood and the attendant flood hazard areas under 1975 land use and channel conditions. Map 1 shows the flood hazard areas as delineated in the federal flood study. About 198 acres, or about 5.5 percent of the total study area, are located within the 100-year recurrence interval flood hazard areas.

STORMWATER DRAINAGE AND FLOODING PROBLEMS

Stormwater Drainage Problems

Areas with known existing drainage problems as identified by the Village of Menomonee Falls Public Works Department are shown on Map 6. Existing stormwater drainage problems include street, yard, and basement flooding due to ice and snow obstructions at culverts and to the inadequate hydraulic capacity of certain roadside swales and associated culverts. There are also problems with erosion along the tributaries to Lilly Creek. Additional areas of potential drainage and flooding problems which were identified through the analyses conducted for the system plan are discussed in Chapter V.

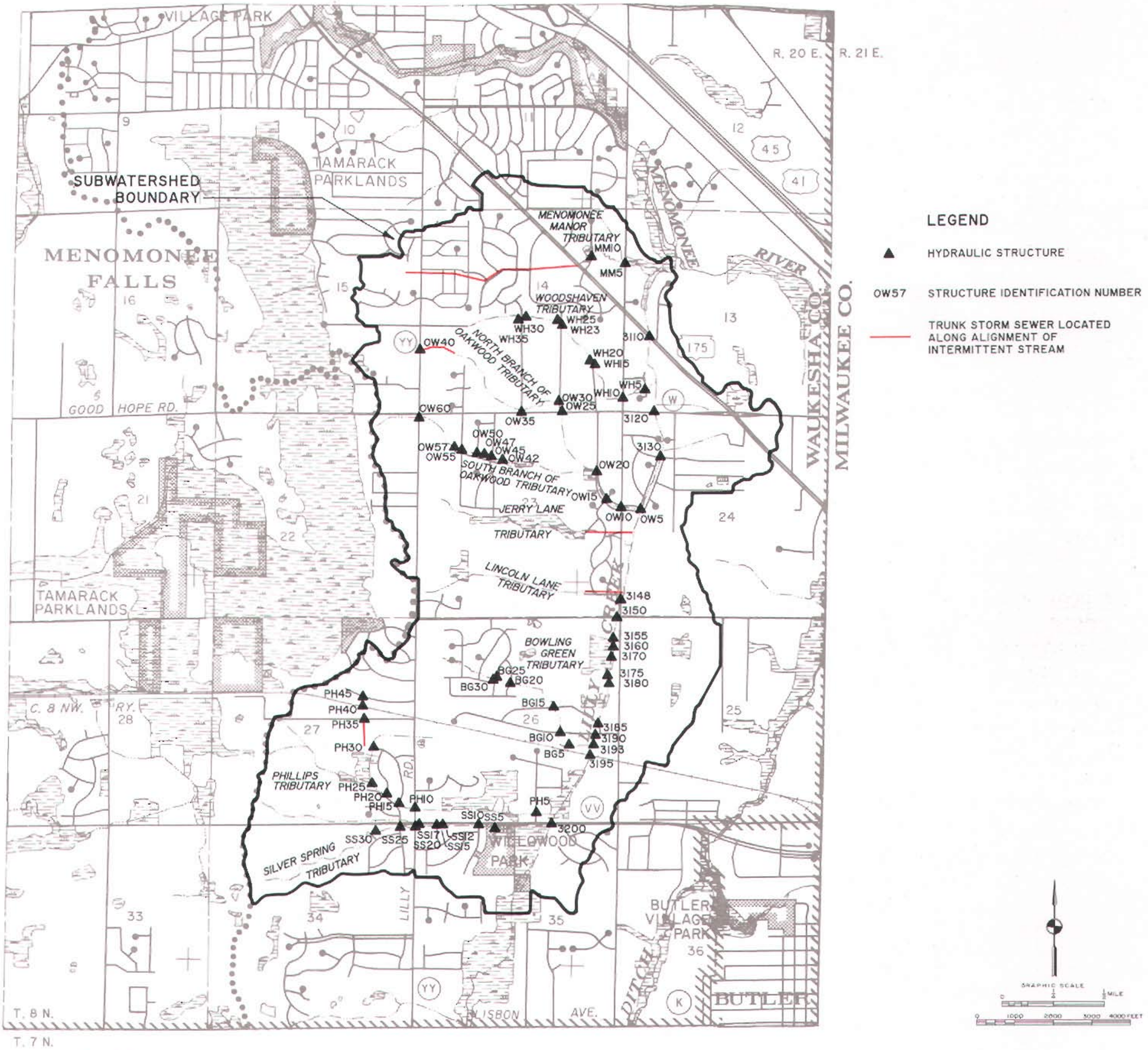
The identified existing and potential drainage problems were considered in the evaluation of the existing stormwater drainage system and in the design of alternative stormwater drainage and flood control system plans. Those plans are thus intended to abate the identified problems.

Infiltration of groundwater and inflow of stormwater into sanitary sewers is a problem related to stormwater drainage. Infiltration may be defined as water that leaks into a sanitary sewerage system through defective pipes, pipe joints, connections, or manhole walls. Inflow may be defined as water discharged into a sanitary sewerage system from such sources as roof leaders, cellar, yard, and area drains, foundation drains, cooling water discharges, drains from springs and swampy areas, manhole covers, cross connections from storm sewers and combined sewers, catch basins, storm waters, surface runoff, street wash waters, or drainage.

Infiltration/inflow studies of the Village's sanitary sewer system, including the northernmost part of the Lilly Creek subwatershed, were conducted by the Village in 1975 and by the

Map 5

LOCATION OF HYDRAULIC STRUCTURES IN THE LILLY CREEK SUBWATERSHED: 1988



Source: SEWRPC.

Milwaukee Metropolitan Sewerage District in 1976 and 1978. Those studies found excessive infiltration and inflow and were, therefore, followed by a sewer system evaluation survey by the District in 1981. That survey recommended a sanitary sewer rehabilitation program to reduce infiltration and inflow.

The Lilly Creek trunk sewer project, which extended sanitary sewer service to the Lilly Creek subwatershed, was completed in 1988. That local trunk sewer is connected to the metropolitan trunk sewer system of the Milwaukee Metropolitan Sewerage District. Through 1988, approximately two-thirds of the onsite

Table 11

STRUCTURE INFORMATION FOR LILLY CREEK AND TRIBUTARIES

Number on Map 5	Structure Identification	U. S. Public Land Survey Section[a]	Structure Type and Size	Structure Length (feet)	Upstream Invert Elevation (feet NGVD)	Downstream Invert Elevation (feet NGVD)
Lilly Creek						
3110	W. Appleton Avenue	NW 1/4, SW 1/4 Section 13	Triple 12.7-foot by 8.0-foot box culvert	183	747.8	747.7
3120	W. Good Hope Road (CTH W)	SW 1/4, SW 1/4 Section 13	25.4-foot by 16.8-foot corrugated metal elliptical pipe	80	753.3	753.1
3130	Brentwood Drive	NW 1/4, NW 1/4 Section 24	15.2-foot by 7.6-foot corrugated metal arch	28.4	758.0	758.0
3140	Lilly Road	SW 1/4, SW 1/4 Section 24	14.0-foot by 4.5-foot box culvert	31	767.7	767.7
3150	W. Mill Road	NE 1/4, NE 1/4 Section 26	16.6-foot by 5.0-foot corrugated metal arch	38.4	767.7	767.7
3155[b]	Private bridge	NE 1/4, NE 1/4 Section 26	Wooden footbridge	- -	- -	- -
3160[b]	Private bridge	NE 1/4, NE 1/4 Section 26	Wooden footbridge	- -	- -	- -
3170	Private bridge	NE 1/4, NE 1/4 Section 26	6.0-foot reinforced concrete pipe	19.6	768.2	768.2
3175	Private bridge	SE 1/4, NE 1/4 Section 26	7.3-foot circular steel pipe	12	767.4	767.1
3180	Private bridge	SE 1/4, NE 1/4 Section 26	10.2-foot circular steel pipe	24	767.4	767.4
3185	W. Kaul Avenue	NE 1/4, SE 1/4 Section 26	12.0-foot by 3.0-foot box culvert	49	768.5	768.5
3190	Bobolink Avenue	NE 1/4, SE 1/4 Section 26	10.4-foot by 3.9-foot box culvert	40	769.1	769.0
3193	Private bridge	NE 1/4, SE 1/4 Section 26	8.0-foot reinforced concrete pipe	21	770.0	769.9
3195	Chicago & North Western Railway	NE 1/4, SE 1/4 Section 26	12.0-foot by 13.0-foot box culvert	45	771.2	771.1
3200	Silver Spring Road, CTH VV	SW 1/4, SE 1/4 Section 26	Four 6.0-foot by 4.0-foot corrugated metal arch	154.8	773.5	772.5
Menomonee Manor Tributary						
MM5	Lilly Road	NE 1/4, NE 1/4 Section 14	78-inch corrugated metal pipe	58.3	752.5	752.3
MM10	Thorndell Drive	NE 1/4, NE 1/4 Section 14	7.8-foot by 5.2-foot corrugated metal pipe arch	34	765.5	765.0

Table 11 (continued)

Number on Map 5	Structure Identification	U. S. Public Land Survey Section[a]	Structure Type and Size	Structure Length (feet)	Upstream Invert Elevation (feet NGVD)	Downstream Invert Elevation (feet NGVD)
Woodshaven Tributary						
WH5	Melville Drive	SW 1/4, SW 1/4 Section 13	6.0-foot by 3.7-foot corrugated metal pipe arch	40.5	757.2	756.8
WH10	Lilly Road	SW 1/4, SW 1/4 Section 13	7.0-foot by 5.2-foot corrugated metal pipe arch	35.5	762.2	762.2
WH15	Driveway	SE 1/4, SE 1/4 Section 14	4.8-foot by 3.2-foot corrugated metal pipe arch	46.7	773.5	773.9
WH20	Driveway	NE 1/4, SE 1/4 Section 14	36-inch corrugated metal pipe	20	774.9	774.5
WH23	Driveway	NW 1/4, SE 1/4 Section 14	3.5-foot by 2.4-foot corrugated metal pipe arch	18	783.4	783.7
WH25	Northwood Drive	NW 1/4, SE 1/4 Section 14	3.6-foot by 2.4-foot corrugated metal pipe arch	36.3	786.2	785.7
WH30	Woodland Drive	NE 1/4, SW 1/4 Section 14	3.5-foot by 2.5-foot corrugated metal pipe arch	40.5	797.3	796.8
WH35	Driveway	NE 1/4, SW 1/4 Section 14	14-foot-wide concrete bridge	19	747.3	746.7
Oakwood Tributary						
OW5	Manor Hills Boulevard	NW 1/4, NW 1/4 Section 24	Double 5.3-foot by 3.3-foot corrugated metal pipe arch	50.5 51.0	764.2 764.1	763.3 763.6
OW10	Lilly Road	SE 1/4, NE 1/4 Section 23	Double 5.4-foot by 3.5-foot corrugated metal pipe arch	40.4 40.4	766.8 766.3	766.3 766.4
OW15	Memory Road	SE 1/4, NE 1/4 Section 23	Double 4.9-foot by 3.0-foot corrugated metal pipe arch	30 30	769.6 769.8	769.4 769.5
OW20	Oakwood Drive	SE 1/4, NE 1/4 Section 23	Double 5.0-foot by 3.0-foot corrugated metal pipe arch	97.2 97.0	773.2 772.9	772.4 772.4
OW25	W. Good Hope Road	SW 1/4, SE 1/4 Section 14	6.5-foot by 4.9-foot concrete and Lannon stone culvert	34.5	787.0	785.7
OW30	Northwood Drive	SW 1/4, SE 1/4 Section 14	5.5-foot by 4.3-foot corrugated metal pipe arch	31.0	788.9	788.6
OW35	Woodland Drive	SE 1/4, SW 1/4 Section 14	6.1-foot by 4.6-foot corrugated metal pipe arch	40.0	798.0	797.5
OW40	Pilgrim Road	NE 1/4, SE 1/4 Section 15	15-inch corrugated metal pipe	38.2	834.4	833.3
OW42	Driveway	NE 1/4, NW 1/4 Section 23	3.5-foot by 2.3-foot corrugated metal pipe arch	20.0	798.6	798.3
OW45	Country Lane	NE 1/4, NW 1/4 Section 23	3.5-foot by 2.3-foot corrugated metal pipe arch	40.5	800.7	800.1

Table 11 (continued)

Number on Map 5	Structure Identification	U. S. Public Land Survey Section[a]	Structure Type and Size	Structure Length (feet)	Upstream Invert Elevation (feet NGVD)	Downstream Invert Elevation (feet NGVD)
Oakwood Tributary (continued)						
OW47	Driveway	NE 1/4, NW 1/4 Section 23	3.5-foot by 2.3-foot corrugated metal pipe arch	20.0	804.0	803.6
OW50	Plainview	NE 1/4, NW 1/4 Section 23	3.0-foot by 2.3-foot corrugated metal pipe arch	40.0	805.4	805.0
OW55	Westwood Drive	NW 1/4, NW 1/4 Section 23	3.0-foot by 1.9-foot corrugated metal pipe arch	40.5	811.6	811.3
OW57	Driveway	NW 1/4, NW 1/4 Section 23	2.0-foot by 1.5-foot corrugated metal pipe arch	20.0	812.3	812.2
OW60	Pilgrim Road	NE 1/4, NE 1/4 Section 22	15-inch corrugated metal pipe	64.7	834.1	832.7
Bowling Green Tributary						
BG5	Driveway	NW 1/4, SE 1/4 Section 26	Double 3.5-foot by 2.5-foot corrugated metal pipe arch	60.0 60.0	774.5 774.6	774.2 774.1
BG10	Bobolink Avenue	NW 1/4, SE 1/4 Section 26	Double 3.5-foot by 2.5-foot corrugated metal pipe arch	61.0 61.0	775.1 775.1	774.6 774.7
BG15	Kaul Avenue	SW 1/4, NE 1/4 Section 26	Double 4.2-foot by 2.7-foot corrugated metal pipe arch	40.3 40.3	783.8 783.0	783.8 783.2
BG20	Wampum Drive	SE 1/2, NW 1/4 Section 26	36-inch corrugated metal pipe 24-inch corrugated metal pipe	40.6 42.5	805.1 805.2	804.6 804.9
BG25	Pochahontus Drive and How Avenue	SE 1/4, NW 1/4 Section 26	30-inch corrugated metal pipe	48.3	807.9	807.2
BG30	Pochahontas Drive	SE 1/4, NW 1/4 Section 26	24-inch corrugated metal pipe 30-inch corrugated metal pipe	40.6 40.5	807.1 807.4	806.4 806.7
Phillips Tributary						
PH5	Enterprise Avenue	SW 1/4, SE 1/4 Section 26	Four 6.0-foot by 3.9-foot corrugated metal pipe arches	43.0 43.0 43.0 43.0	772.9 772.8 772.9 773.0	772.9 772.8 772.8 772.8
PH10	Pilgrim Road	SE 1/4, SE 1/4 Section 27	7.5-foot by 4.5-foot corrugated metal pipe arch	40.4	782.2	782.0
PH15	Cheryln Drive	SE 1/4, SE 1/4 Section 27	Double 3.0-foot by 2.0-foot corrugated metal pipe arch	42.5 42.5	786.2 786.1	785.7 785.8
PH20	Elmway Drive	SE 1/4, SE 1/4 Section 27	Double 2.5-foot by 1.5-foot corrugated metal pipe arch	42.5 42.5	788.5 788.5	788.4 788.0
PH25	Driveway	SE 1/4, SE 1/4 Section 27	3.5-foot by 2.3-foot corrugated metal pipe arch	20	790.3	790.2
PH30	Cheryln Drive	NE 1/4, SE 1/4 Section 27	3.7-foot by 2.2-foot corrugated metal pipe arch	48.1	796.5	795.8

Table 11 (continued)

Number on Map 5	Structure Identification	U. S. Public Land Survey Section[a]	Structure Type and Size	Structure Length (feet)	Upstream Invert Elevation (feet NGVD)	Downstream Invert Elevation (feet NGVD)
Phillips Tributary (continued)						
PH35	Kohler Lane	SE 1/4, NE 1/4 Section 27	24-inch reinforced concrete pipe	85	808.1	807.2
PH40	Chicago & North Western Railway	SE 1/4, NE 1/4 Section 27	35-inch steel culvert	75.5	814.7	812.5
PH45	Hawthorne Drive	SW 1/4, NE 1/4 Section 27	30-inch corrugated metal pipe	44.5	825.5	824.3
Silver Spring Tributary						
SS5	Driveway	NE 1/4, NW 1/4 Section 35	Double 6.0-foot by 3.9-foot corrugated metal pipe arch	50 50	775.6 775.6	775.4 775.2
SS10	Badger Drive	NE 1/4, NW 1/4 Section 35	Double 6.0-foot by 4.0-foot corrugated metal pipe arch	36.0 36.3	776.8 776.9	776.5 776.7
SS12	Driveway	NE 1/4, NW 1/4 Section 35	Double 6.0-foot by 4.0-foot corrugated metal pipe arch	18 18	779.8 779.8	779.9 779.6
SS15	Butternut Drive	NW 1/4, NW 1/4 Section 35	Double 6.0-foot by 4.0-foot corrugated metal pipe arch	36.0 36.2	780.1 780.1	779.6 779.7
SS17	Driveway	NW 1/4, NW 1/4 Section 35	Double 6.0-foot by 4.0-foot corrugated metal pipe arch	42 42	783.8 783.8	783.5 783.6
SS20	Pilgrim Road	NE 1/4, NE 1/4 Section 34	Double 6.0-foot by 3.9-foot corrugated metal pipe arch	82.7 82.7	784.4 784.4	783.5 783.8
SS25	Bette Drive	NE 1/4, NE 1/4 Section 34	Double 6.0-foot by 3.9-foot corrugated metal pipe arch	36.5 36.3	786.6 786.7	786.5 786.4
SS30	Driveway	NE 1/4, NE 1/4 Section 34	Double 4.8-foot by 3.0-foot corrugated metal pipe arch	28.3 28.3	794.8 794.9	794.3 794.3

[a] *All structures are located in T8N, R20E.*

[b] *These structures were deemed to be hydraulically insignificant. That is, they do not represent a significant obstruction to flow under flood conditions. Consequently, no detailed survey information is provided.*

Source: SEWRPC.

sewage disposal systems in the subwatershed had been replaced by sanitary sewers connected to the new trunk sewer. By the end of 1992, it is anticipated that all urban land uses in the subwatershed will substantially be connected to sanitary sewers. Because the new collector and trunk sewers have only been in place for a short time, no data are available on the amount of infiltration and inflow to the new system.

The Village ordinance prohibiting the connection of clearwater drains to the sanitary sewer system, along with the provision of an efficient stormwater drainage system as recommended in this report, should help limit infiltration and inflow to the sanitary sewerage system as urban development proceeds in the subwatershed. In areas of intensive urban development, infiltration of stormwater to reduce pollutant loadings

Map 6

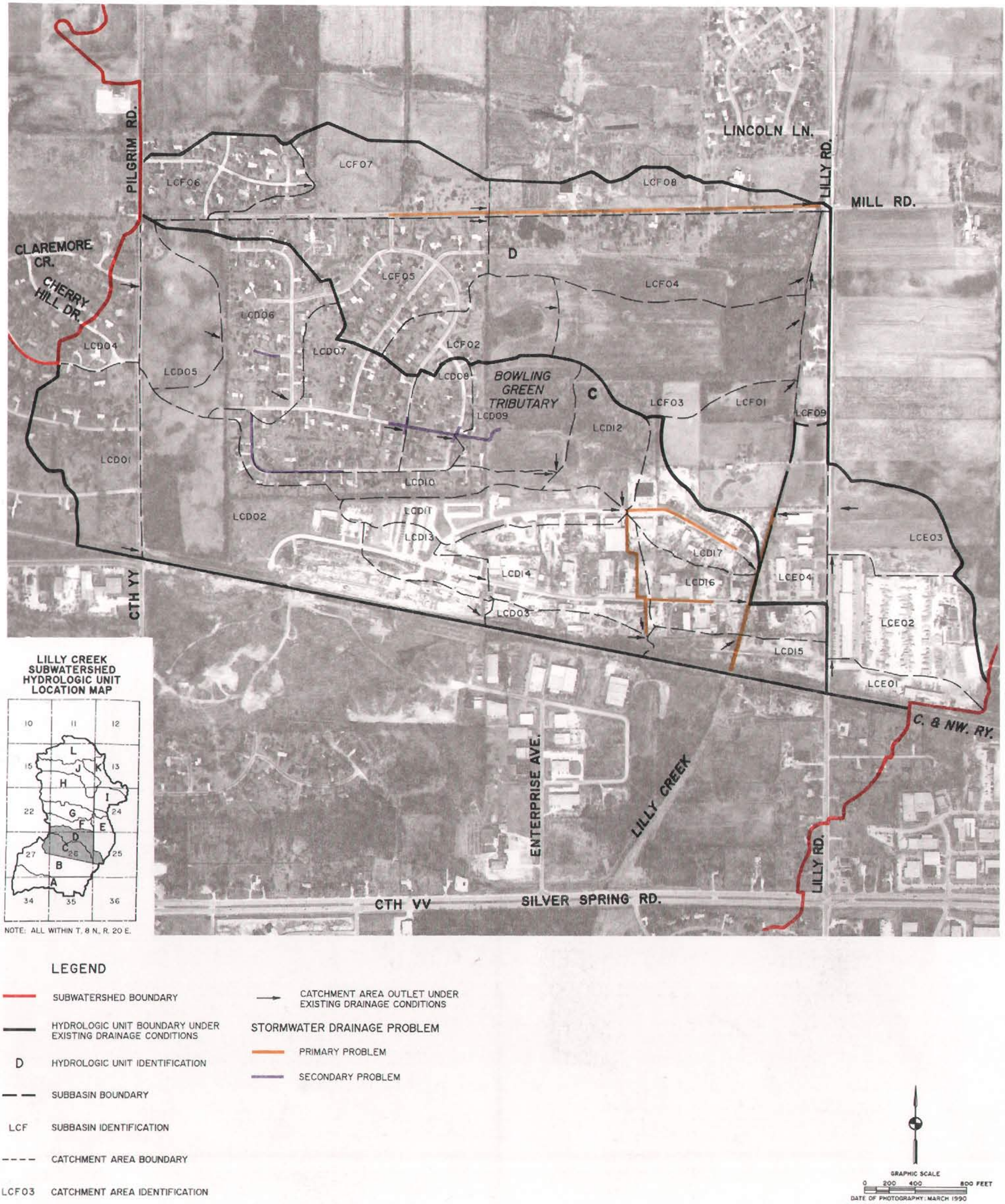
EXISTING STORMWATER DRAINAGE PROBLEM AREAS IN THE LILLY CREEK WATERSHED

HYDROLOGIC UNITS A AND B

Map 6 (continued)

EXISTING STORMWATER DRAINAGE PROBLEM AREAS IN THE LILLY CREEK WATERSHED

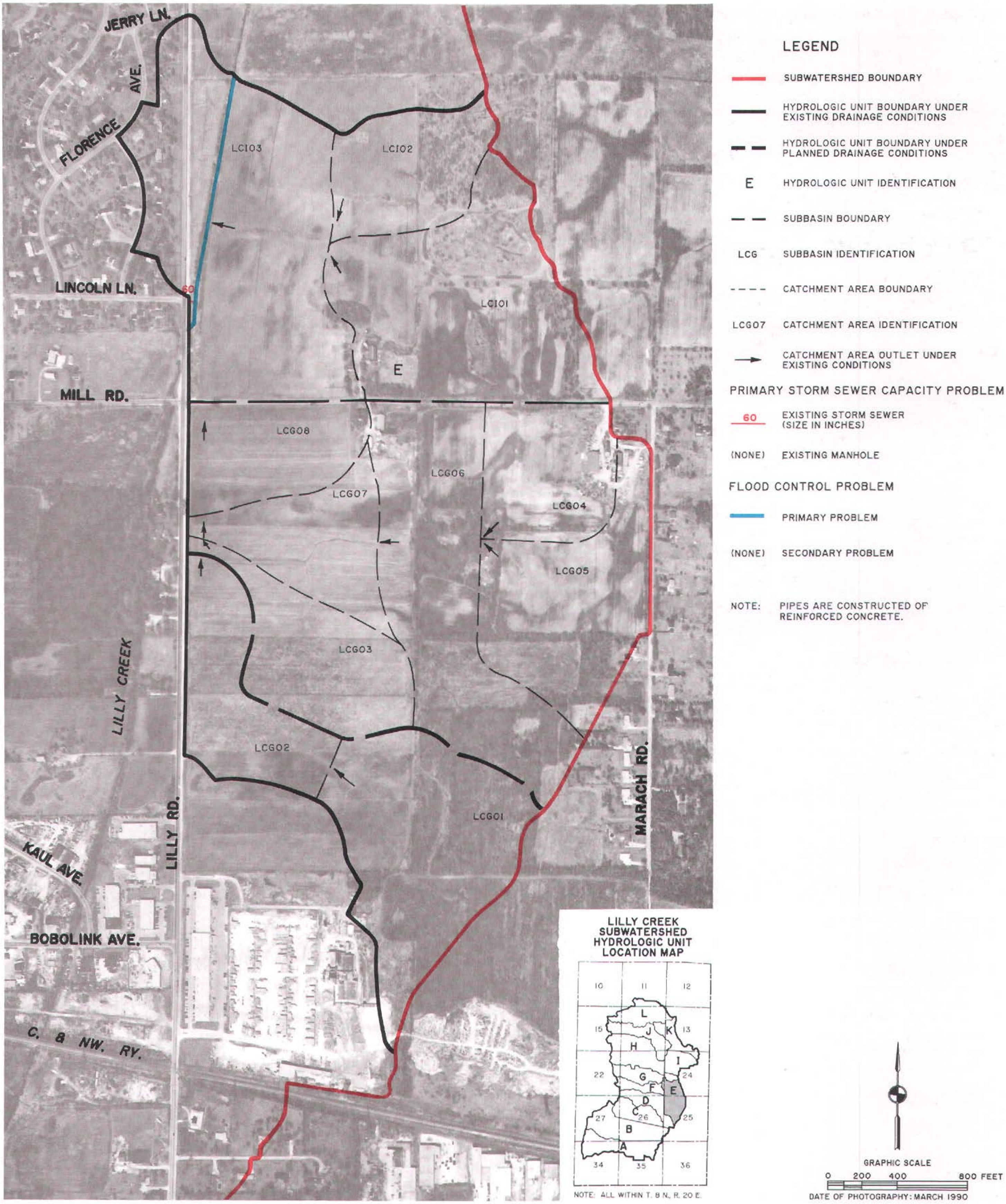
HYDROLOGIC UNITS C AND D



Map 6 (continued)

EXISTING STORMWATER DRAINAGE PROBLEM AREAS IN THE LILLY CREEK WATERSHED

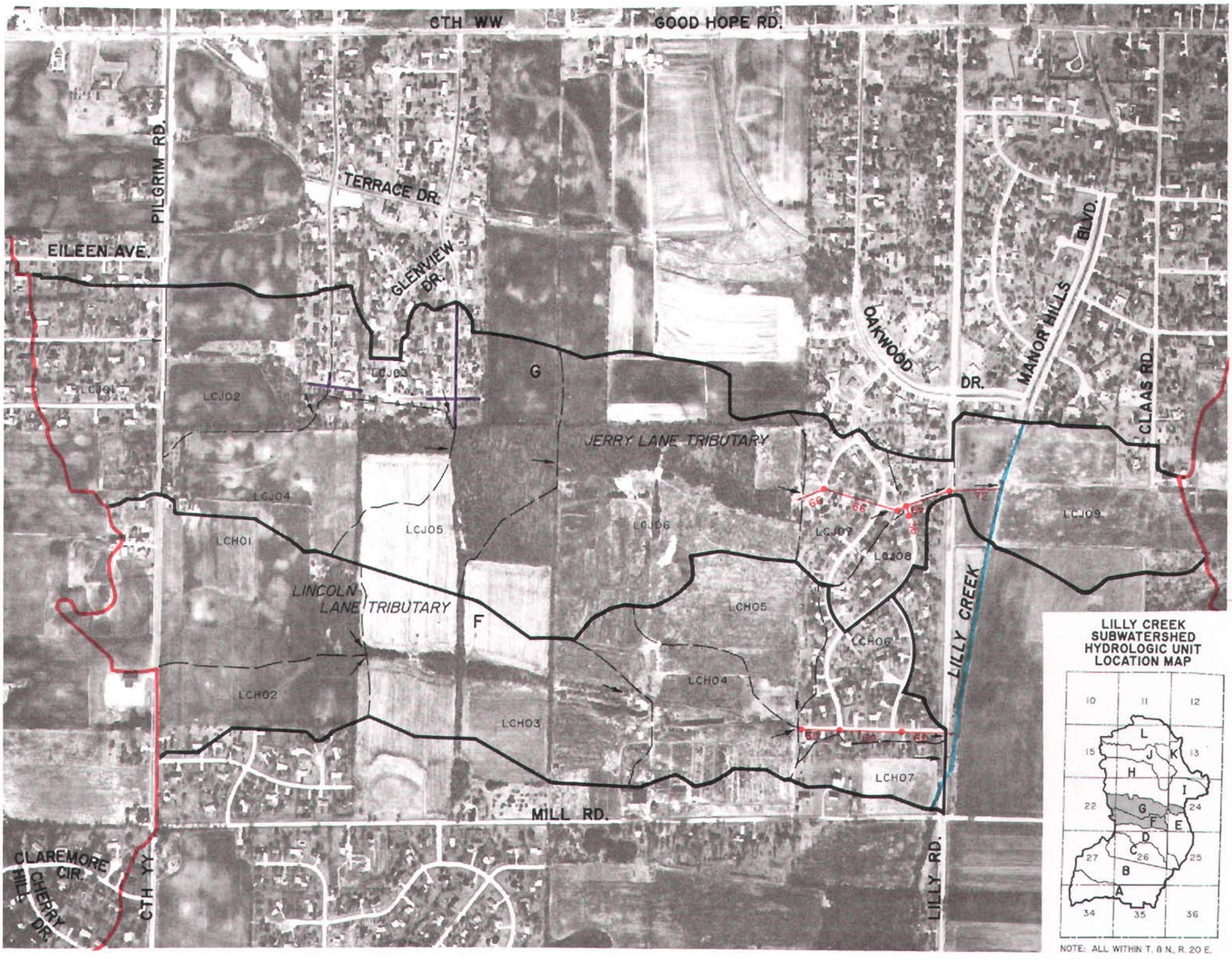
HYDROLOGIC UNIT E



Map 6 (continued)

EXISTING STORMWATER DRAINAGE PROBLEM AREAS IN THE LILLY CREEK WATERSHED

HYDROLOGIC UNITS F AND G



LEGEND

━	SUBWATERSHED BOUNDARY
━	HYDROLOGIC UNIT BOUNDARY UNDER EXISTING DRAINAGE CONDITIONS
G	HYDROLOGIC UNIT IDENTIFICATION
(NONE)	SUBBASIN BOUNDARY
LCF	SUBBASIN IDENTIFICATION
- - - -	CATCHMENT AREA BOUNDARY
LCF07	CATCHMENT AREA IDENTIFICATION
→	CATCHMENT AREA OUTLET UNDER EXISTING DRAINAGE CONDITIONS

PRIMARY STORM SEWER CAPACITY PROBLEM

60	EXISTING STORM SEWER (SIZE IN INCHES)
●	EXISTING MANHOLE

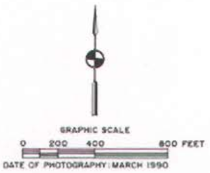
STORMWATER DRAINAGE PROBLEM

(NONE)	PRIMARY PROBLEM
━	SECONDARY PROBLEM

FLOOD CONTROL PROBLEM

━	PRIMARY PROBLEM
(NONE)	SECONDARY PROBLEM

NOTE: PIPES ARE CONSTRUCTED OF REINFORCED CONCRETE.

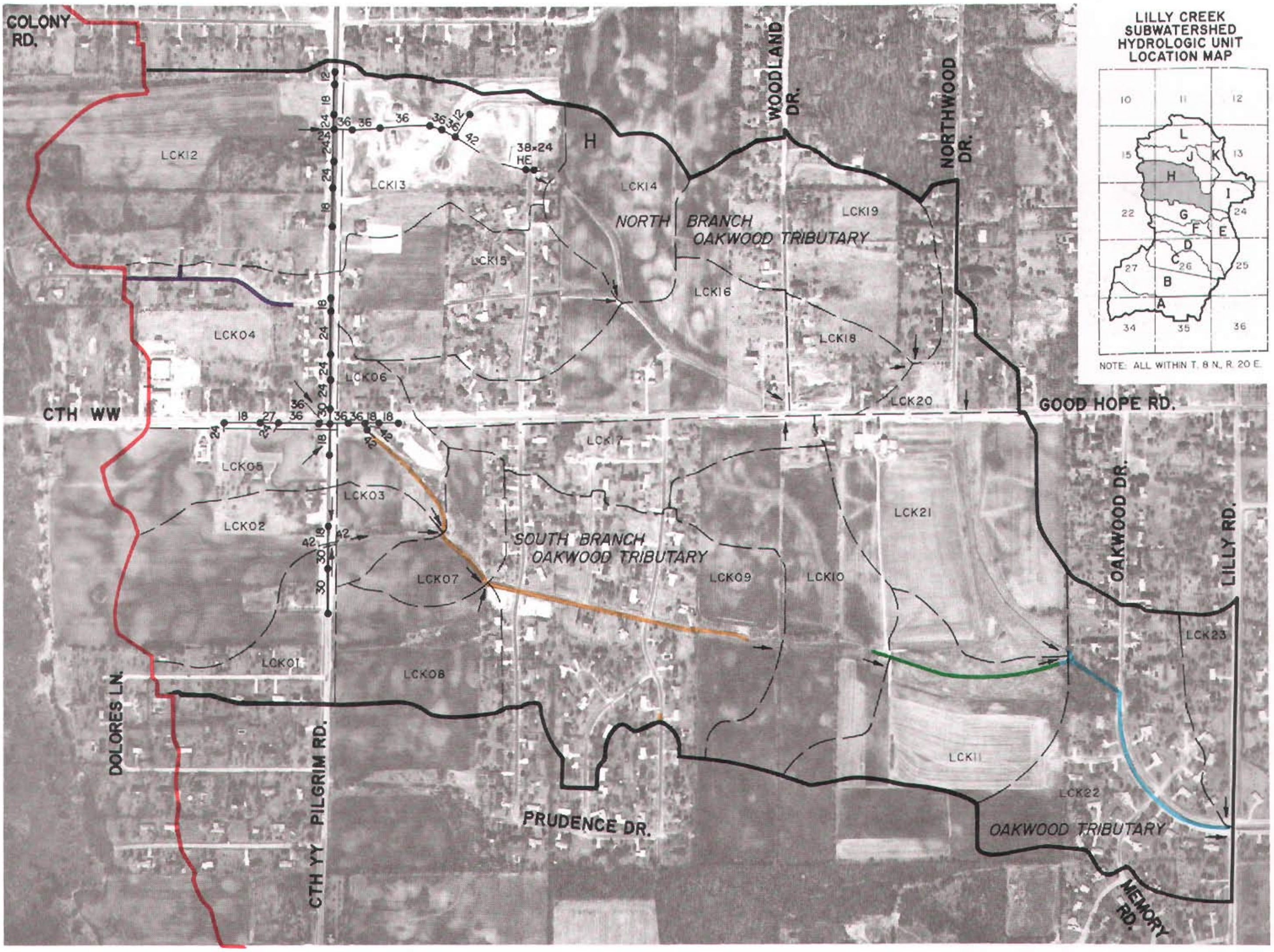


DATE OF PHOTOGRAPHY: MARCH 1990

Map 6 (continued)

EXISTING STORMWATER DRAINAGE PROBLEM AREAS IN THE LILLY CREEK WATERSHED

HYDROLOGIC UNIT H



LEGEND

—	SUBWATERSHED BOUNDARY
—	HYDROLOGIC UNIT BOUNDARY UNDER EXISTING DRAINAGE CONDITIONS
H	HYDROLOGIC UNIT IDENTIFICATION
(NONE)	SUBBASIN BOUNDARY
LCK	SUBBASIN IDENTIFICATION
----	CATCHMENT AREA BOUNDARY
LCK16	CATCHMENT AREA IDENTIFICATION
→	CATCHMENT AREA OUTLET UNDER EXISTING DRAINAGE CONDITIONS
42	EXISTING STORM SEWER (SIZE IN INCHES)
●	EXISTING MANHOLE

STORMWATER DRAINAGE PROBLEM

—	PRIMARY PROBLEM
—	SECONDARY PROBLEM

STREAMBANK EROSION PROBLEM

—	PRIMARY PROBLEM
(NONE)	SECONDARY PROBLEM

FLOOD CONTROL PROBLEM

—	PRIMARY PROBLEM
(NONE)	SECONDARY PROBLEM

TYPE OF STORM SEWER

HE	HORIZONTAL ELLIPTICAL REINFORCED CONCRETE PIPE

NOTE: PIPES ARE CONSTRUCTED OF REINFORCED CONCRETE.

GRAPHIC SCALE

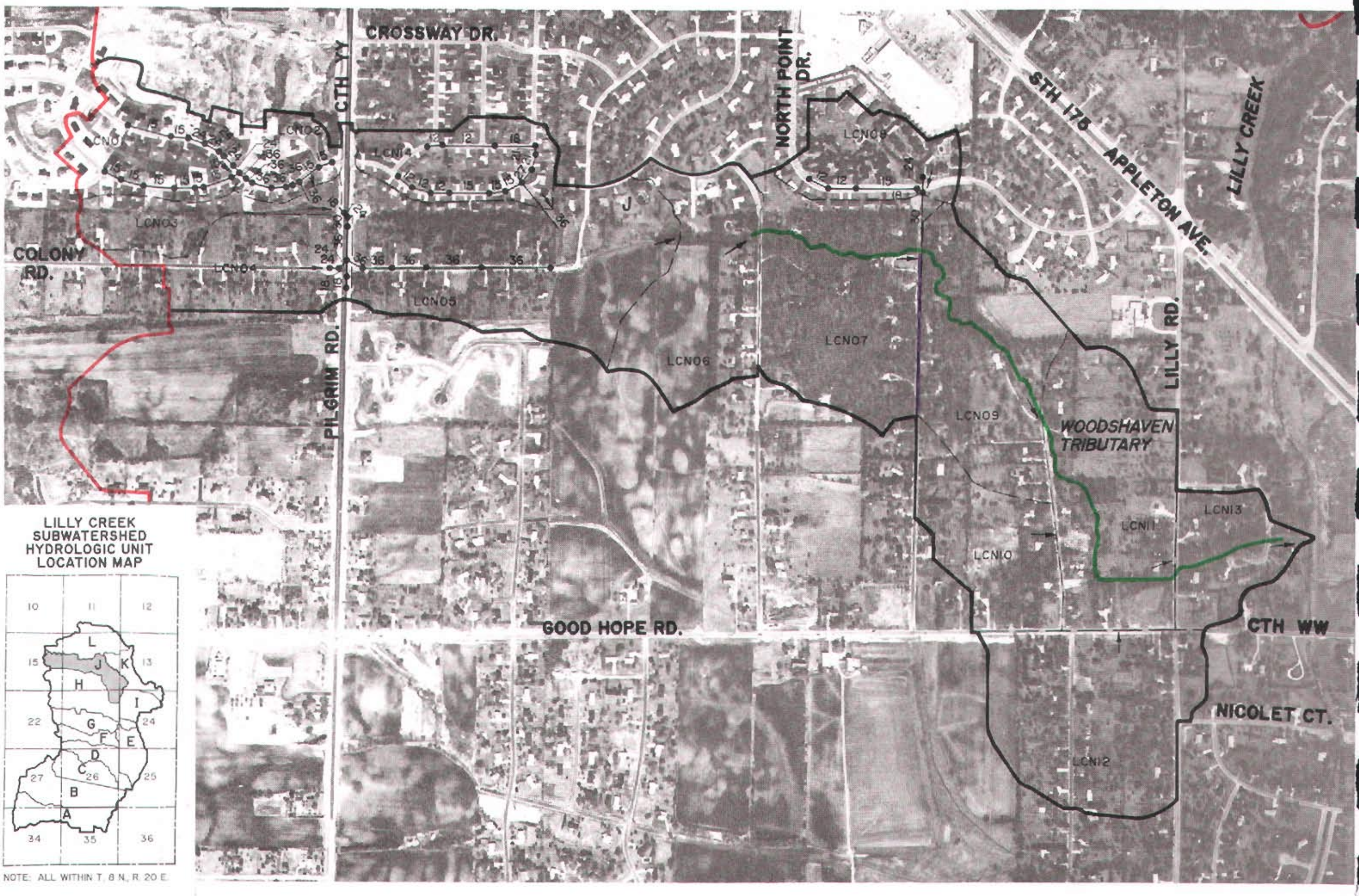
0 200 400 800 FEET

DATE OF PHOTOGRAPHY: MARCH 1990

Map 6 (continued)

EXISTING STORMWATER DRAINAGE PROBLEM AREAS IN THE LILLY CREEK WATERSHED

HYDROLOGIC UNIT J



NOTE: ALL WITHIN T. 8 N., R. 20 E.

LEGEND

- SUBWATERSHED BOUNDARY
- HYDROLOGIC UNIT BOUNDARY UNDER EXISTING DRAINAGE CONDITIONS
- J HYDROLOGIC UNIT IDENTIFICATION
- (NONE) SUBBASIN BOUNDARY
- LCN SUBBASIN IDENTIFICATION
- ---- CATCHMENT AREA BOUNDARY
- LCN09 CATCHMENT AREA IDENTIFICATION
- → CATCHMENT AREA OUTLET UNDER EXISTING DRAINAGE CONDITIONS
- 42 EXISTING STORM SEWER (SIZE IN INCHES)
- ● EXISTING MANHOLE

STORMWATER DRAINAGE PROBLEM

- (NONE) PRIMARY PROBLEM
- SECONDARY PROBLEM

STREAMBANK EROSION PROBLEM

- PRIMARY PROBLEM
- (NONE) SECONDARY PROBLEM

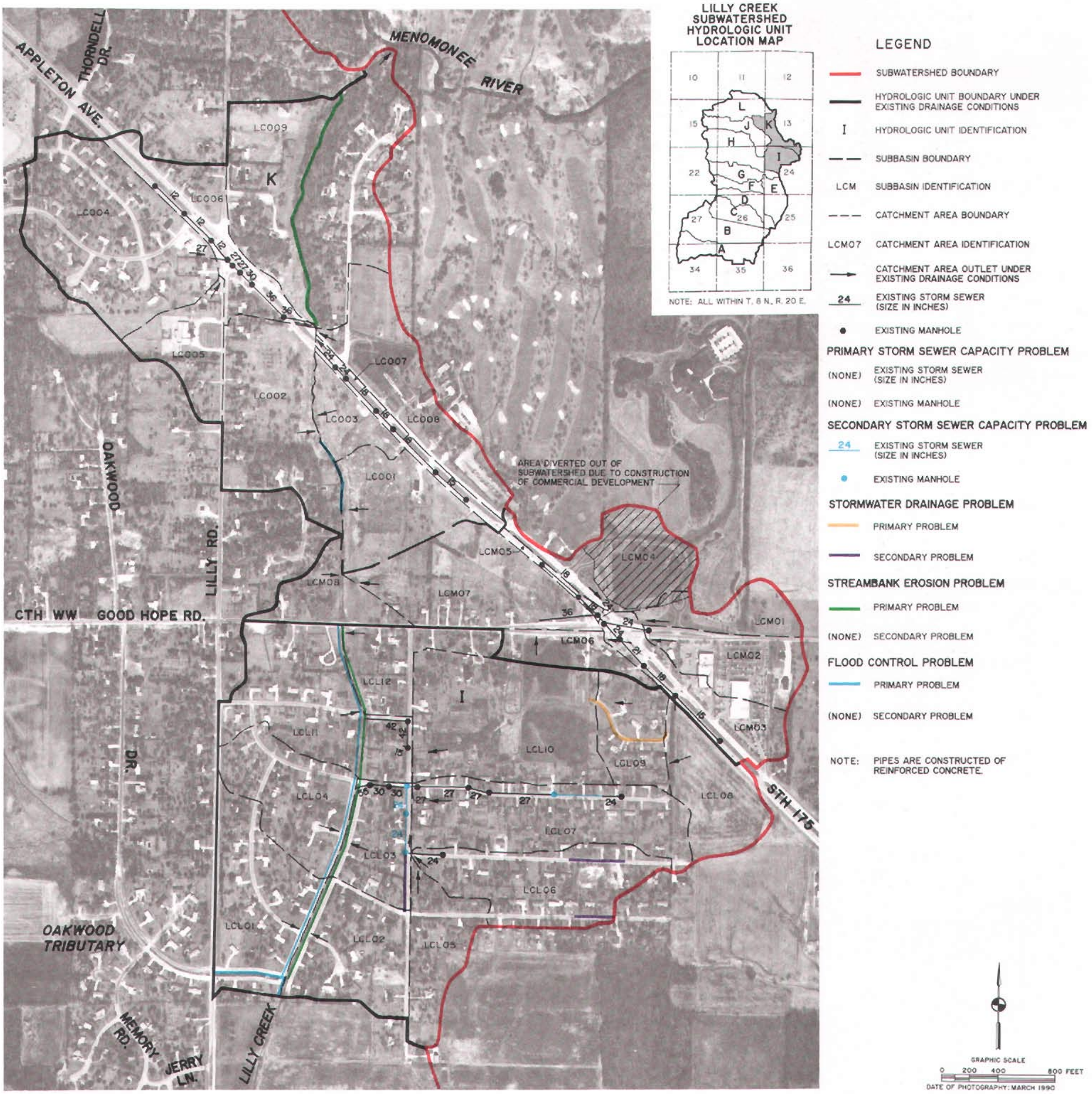
NOTE: PIPES ARE CONSTRUCTED OF REINFORCED CONCRETE.

GRAPHIC SCALE
0 200 400 800 FEET
DATE OF PHOTOGRAPHY: MARCH 1990

Map 6 (continued)

EXISTING STORMWATER DRAINAGE PROBLEM AREAS IN THE LILLY CREEK WATERSHED

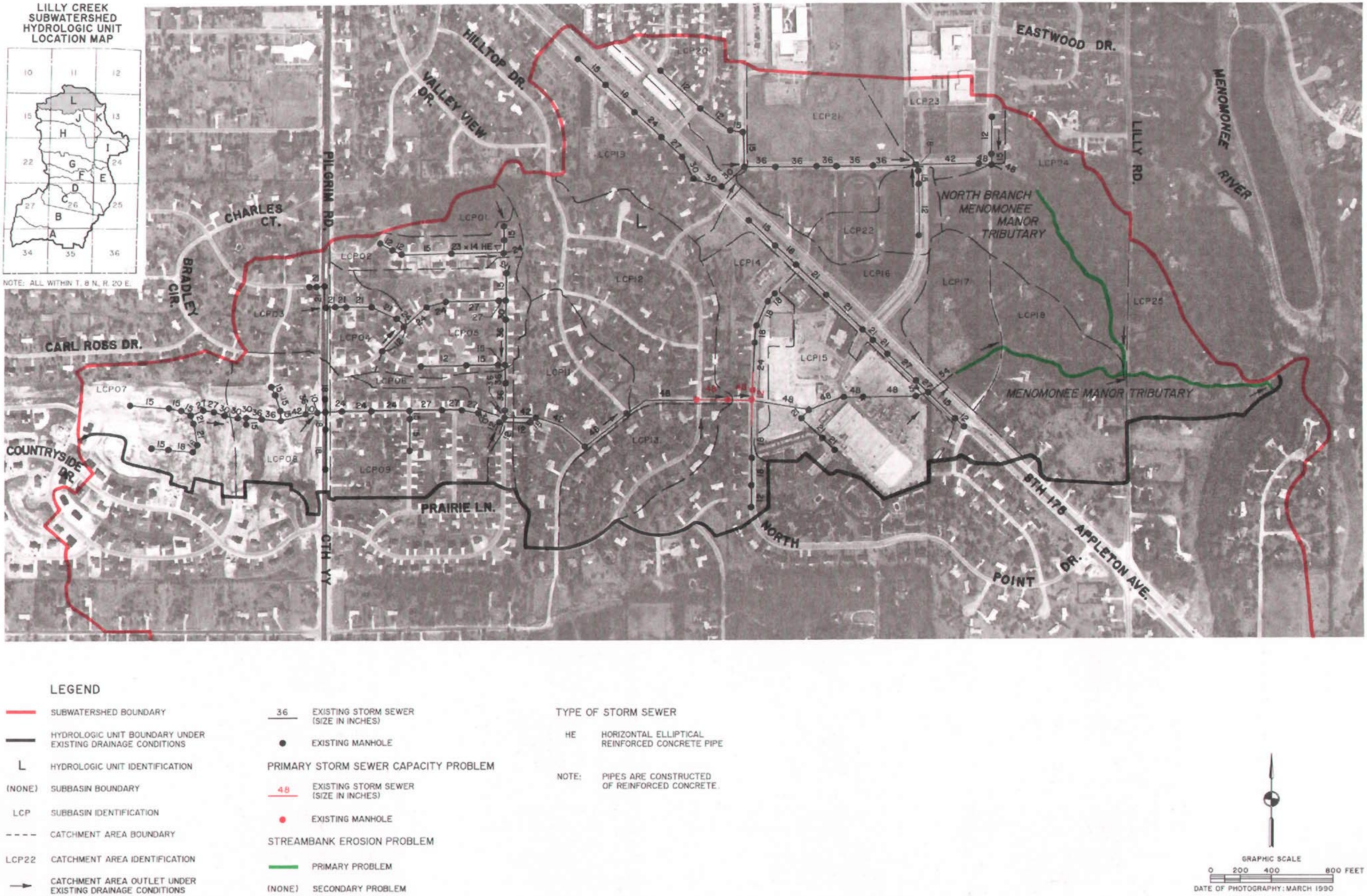
HYDROLOGIC UNITS I AND K



Map 6 (continued)

EXISTING STORMWATER DRAINAGE PROBLEM AREAS IN THE LILLY CREEK WATERSHED

HYDROLOGIC UNIT L



Source: SEWRPC.

and runoff volumes delivered to receiving streams may conflict with the goal of reducing infiltration and inflow to sanitary sewers. Therefore, infiltration measures are best used in areas of low density development.

Flooding Problems

The severity of flooding problems along Lilly Creek has increased over time as urban development has proceeded in the subwatershed. Historic flooding problems through 1973 are documented in SEWRPC Planning Report No. 26, A Comprehensive Plan for the Menomonee River Watershed, Volume One, October 1976. The floods of March 30, 1960, and July 18, 1964, resulted in local street closings due to flooding, while the flood of April 21, 1973, resulted in secondary flooding of basements along the 1.34-mile reach of Lilly Creek from the Menomonee River to the intersection of Oakwood Drive and Manor Hills Boulevard. Observations made by local residents during the 1973 flood indicate that the extent of flooding along the Creek in the vicinity of Good Hope Road may have been affected by the accumulation of debris on the upstream side of the Good Hope Road culvert. That culvert has since been replaced with a larger structure.

The floods of August 6, 1986, and September 11, 1986, which were of similar magnitude, had the most severe consequences of any floods which had occurred through 1991. Those two floods caused structure flooding and damages. Flooding of an exposed basement was reported at one home located north of Good Hope Road, along Melville Drive. Flooding problems were most severe in the approximately 0.5-mile-long reach of Lilly Creek along Manor Hills Boulevard from the stream crossing at Brentwood Drive to Oakwood Drive, where direct overland first-floor flooding was reported at one home and overland basement flooding was reported at three homes. In that same area, the Village also reported secondary flooding of basements due to backups through septic systems. First-floor flooding also occurred at an office along Lilly Creek at Bobolink Avenue in the Bowling Green Industrial Park. In addition, high water levels in the Creek, combined with channel and culvert obstructions due to construction activity on the Lilly Creek trunk sewer, may have aggravated stormwater drainage and flooding problems near the mouths of tributaries to the Creek where there was basement flooding. Such flooding was experienced in the Lincoln Lane-Lilly Road area.

DESCRIPTION OF SOURCES OF WATER POLLUTION

The quality of the surface waters in the Lilly Creek subwatershed is an important concern of this study. Improper stormwater management may result in pollutant contributions from the watershed to the streams and also in high flow velocities and volumes, which can cause erosion of streambanks and scour of the streambed. Under these conditions, high pollutant loadings are contributed, some of which are deposited in downstream beds, thereby potentially influencing water quality conditions over a relatively long period of time. Erosion and the resulting sediment contributed to the stream systems can destroy important stream and riparian fish and aquatic life habitat and result in the discharge of pollutants, such as nutrients, pesticides, and metals, which are transported in the stream system attached to sediment particles. Stormwater runoff from urban lands, including lawns and pavements, can contain high concentrations of water pollutants, such as organic substances, nutrients, fecal coliform organisms, metals, and sediment. High pollutant concentrations and excessive erosion and sedimentation in the streams of the subwatershed reduce their suitability, and the suitability of downstream waters, for recreational uses such as swimming, fishing, and boating; limit the ability of the water body to support desirable forms of fish and other aquatic life; adversely affect the aesthetics of the water resource; reduce the hydraulic capacity of drainage channels and streams; and result in the loss of, or damage to, public and private shoreline property.

There are no Wisconsin Pollutant Discharge Elimination System (WPDES) permitted point sources of pollution which discharge to Lilly Creek or its tributaries. Thus, nonpoint sources of pollution account for essentially all of the pollutant loadings to Lilly Creek and its tributaries. These nonpoint sources include urban and rural land stormwater runoff, construction site erosion, streambank erosion, atmospheric contributions, industrial material leaks and spills, and malfunctioning septic tank systems.[2]

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[2] *Although industrial material leaks and spills are considered to be point pollution sources for regulatory purposes, the leaks and spills which have occurred in the subwatershed are more characteristic of nonpoint sources of pollution and are, therefore, categorized as such here.*

Pollutant loading estimates to Lilly Creek are presented in Chapter V, "Evaluation of Existing and Alternative Future Stormwater Management and Flood Control Systems," of this report.

Rural Land Runoff

In 1985, approximately 50 percent of the total Lilly Creek subwatershed area was in rural land use, with agricultural and open land accounting for about 88 percent of the rural total, and woodlands, wetlands, and open surface water accounting for the remaining 12 percent. By the year 2010, over 99 percent of the agricultural and open land is expected to be converted to urban land use. Thus, the importance of rural land runoff as a source of pollution to Lilly Creek is expected to decline substantially in the future.

The most significant rural nonpoint source of pollutants to Lilly Creek is cropland erosion, which contributes sediments, nutrients, organic matter, and pesticides to the stream. The severity and extent of water pollution from cropping practices varies considerably depending on the soils, slopes, and types of cropping practices used. There were no livestock operations located in the subwatershed in 1985. An inventory of the severity of cropland erosion is presented in the Menomonee River watershed nonpoint source control plan prepared by the Wisconsin Department of Natural Resources.

Natural, undisturbed woodlands and wetlands contribute few pollutants to surface waters. Only about 16 percent of the present woodland and wetland area is expected to be converted to urban use by the year 2010. These natural areas will thus remain as important natural buffers to help reduce pollutant loadings to the streams. Disturbances such as tree harvesting, road and trail construction, drainage, or filling would reduce the quality of the woodland and wetland areas and increase pollutant loadings.

Urban Land Runoff

Urban land uses covered about 50 percent of the Lilly Creek subwatershed in 1985. Residential use accounted for 67 percent of the total urban area. By the year 2010, urban land uses are expected to cover almost 95 percent of the subwatershed area. Stormwater runoff from lawns, rooftops, streets and driveways, parking lots, and storage areas contributes sediment, nutrients, organic matter, oil and grease, bacteria, metals, and toxic organic substances to streams. Urban development generally increases stormwater flow rates and runoff volumes and the loadings of some pollutants. Stormwater runoff impacts are most severe in areas having large amounts of impervious areas directly connected to storm sewers or receiving waters. Stormwater pollutant concentrations and loadings vary considerably depending on the land use and land management activities.

Of particular concern is the potential for increased loadings of some priority pollutants. The priority pollutants are 126 substances identified by the U. S. Environmental Protection Agency found in surface waters and which, in excessive concentrations, are toxic to humans or to fish and other aquatic life. Some of these priority pollutants may be deposited in the bottom sediments, potentially contaminating fish food supplies and having toxic effects on benthic organisms. Certain pollutants accumulate in the tissue of aquatic organisms. The Wisconsin Department of Natural Resources has issued fish consumption advisories for some urban streams because of accumulations of polychlorinated biphenyls (PCBs) in the tissue of fish. The U. S. Environmental Protection Agency, as part of the Nationwide Urban Runoff Program completed in 1983,[3] measured the concentration of priority pollutants in 121 urban runoff samples collected at 61 sites located throughout the United States. The Agency reported that 77 of the 126 priority pollutants were each detected in at least one of the urban runoff samples. Each of 17 of the priority pollutants listed in Table 12 were detected in more than 10 percent of the runoff samples. Five of the substances, all metals, were detected in more than 50 percent of the samples tested, with three of those metals, lead, zinc, and copper, detected in more than 90 percent of the samples. The metals lead, zinc, copper, and cadmium were also frequently detected at all of the sites monitored under a Nationwide Urban Runoff Program project conducted in Milwaukee County.[4]

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[3] *U. S. Environmental Protection Agency, Results of the Nationwide Urban Runoff Program, Volume I, Final Report, December 1983.*

[4] *R. Bannerman, K. Baun, M. Bohn, P. E. Hughes, and D. A. Graczyk, Evaluation of Urban Nonpoint Source Pollution Management in Milwaukee County, Wisconsin, Volume I, Urban Stormwater Characteristics, Sources, and Pollutant Management by Street Sweeping, U. S. Environmental Protection Agency, PB 84-113164, 1983.*

Table 12

PRIORITY POLLUTANTS DETECTED IN MORE THAN 10 PERCENT OF URBAN STORMWATER RUNOFF SAMPLES TESTED: 1983

Priority Pollutant	Detection Level (percent)
1. Lead	94
2. Zinc	94
3. Copper	91
4. Chrominum	58
5. Arsenic	52
6. Cadmium	48
7. Cyanide	23
8. α -Hexachlorocyclohexane	20
9. α -Endosulfan	19
10. Pentachlorophenol	19
11. Chlordane	17
12. Fluoranthene	16
13. γ -Hexachlorocyclohexane (Lindane)	15
14. Pyrene	15
15. Phenol	14
16. Phenanthrene	12
17. Dichloromethane (methylene chloride)	11

Source: U. S. Environmental Protection Agency.

Toxic organic substances were less prevalent than were metals in the runoff samples. All of the organic substances tested were identified in 20 percent or less of the samples tested.

The U. S. Environmental Protection Agency reported that acute and/or chronic water quality criteria recommended by the Agency for lead, zinc, copper and cadmium levels were exceeded in some of the urban runoff samples.[5] These excedents of the criteria do not necessarily indicate that an actual violation of the criteria would occur in receiving waters. However, once the Lilly Creek subwatershed is essentially fully developed in urban use, urban runoff will constitute the majority of the flow in the creek during storm events. Thus, criteria violations could indeed occur in the creek during storm events if nonpoint source controls are not provided.

Table 13 presents a list of selected toxic substances frequently detected in stormwater runoff from residential and industrial land. Pesticides were most frequently found in residential areas, while industrial land runoff more often contained other toxic organic substances. Metals were frequently found in both residential and industrial land runoff.

Table 13

SELECTED TOXIC SUBSTANCES FREQUENTLY DETECTED IN RESIDENTIAL AND INDUSTRIAL LAND STORMWATER RUNOFF

Toxic Substance	Residential Land Runoff	Industrial Land Runoff
Haloginated Aliphatics		
1,2,-dichlorethane	- -	X
Methylene chloride	- -	X
Tetrachlorethylene	- -	X
Phthalate Esters		
Bis (2-Ethylene) phthalate	X	- -
Butylbenzyl phthalate	X	X
Diethyl phthalate	- -	X
Di-N-Butyl phthalate	X	X
Polycyclic Aromatic Hydrocarbons		
Phenanthrene	- -	X
Pyrene	- -	X
Chrysene	X	X
Fluoranthene	- -	X
Other Volatile Compounds		
Benzene	X	X
Chloroform	- -	X
Ethylbenzene	- -	X
N-Nitro-sodimethylamine	- -	X
Toluene	- -	X
Metals		
Chromium	- -	X
Copper	X	X
Lead	X	X
Zinc	X	X
Pesticides and Phenols		
γ -Hexachlorocyclohexane (Lindane)	X	- -
Chlordane	X	- -
Dieldrin	X	- -
Endosulfan sulfate	X	- -
Endrin	X	- -
Isophorone	X	- -
Methoxychlor	X	- -
Polychlorinated biphenyls	- -	X
Pentachlorophenol	X	X
Phenol	X	X
α -Hexachlorocyohexane	X	- -

Source: Robert Pitt, Wisconsin Department of Natural Resources.

Potential sources of selected toxic substances in urban runoff are listed in Table 14. Studies have found that some substances, such as Lindane, dieldrin, polychlorinated biphenyls, and some metals, are contributed to urban waters during both wet weather and dry weather.[6] Automobile

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[5] *U. S. Environmental Protection Agency, Results of the Nationwide Urban Runoff Program, Volume I, Final Report, December 1983.*

[6] *R. Pitt and J. McLean, Toronto Area Watershed Management Strategy Study: Humber River Pilot Watershed Project, Ontario Ministry of the Environment, Toronto, Ontario, 1986.*

Table 14

POTENTIAL SOURCES OF SELECTED TOXIC SUBSTANCES FOUND IN URBAN RUNOFF

Toxic Substances	Automobile Use	Pesticide Use	Industrial Use
Melogenated Aliphatics			
Methylene chloride	- -	Fumigant	Plastics, paint remover, solvents
Methyl chloride	Leaded gas	Fumigant	Refrigerant, solvent
Phthalate Esters			
Bis(2-ethyhexyl) phthalate	- -	- -	Plasticizer
Butylbenzyl phthalate	- -	- -	Plasticizer, printing inks, paper, stain, adhesive
Di-N-butyl phthalate	- -	Insecticide	- -
Polycyclic Aromatic Hydrocarbons			
Chrysene	Gasoline oil/grease	- -	Solvent
Phenanthrene	Gasoline	- -	Wood and coal combustion
Pyrene	Gasoline, soil, asphalt	Wood preservative	Wood and coal combustion
Other Volatile Compounds			
Benzene	Gasoline	- -	Solvent
Chloroform	Formed from salt, gasoline, asphalt	Insecticide	Solvent, chlorination
Toluene	Gasoline, asphalt	- -	Solvent
Metals			
Chromium	Metal corrosion	- -	Paint, metal corrision, electroplating
Copper	Metal corrosion	Algicide	Paint, metal corrosion, electroplating
Lead	Gasoline, batteries	- -	Paint
Zinc	Metal corrosion, road salt, rubber	Wood preservative	Paint, metal corrosion
Pesticide and Phenols			
γ-Hexachlorocyclohexane (Lindane)	- -	Mosquito control, seed pretreatment	- -
Chlordane	- -	Termite control	- -
Dieldrin	- -	Insecticide	Wood processing
α-Endosulfan	- -	Insecticide	- -
α-Hexachlorocyclohexane	- -	Insecticide	- -
Pentachlorophenol	- -	Wood preservative	Paint
Polychlorinated biphenyls	- -	- -	Electrical, insulation, paper adhesives

Source: Robert Pitt, Wisconsin Department of Natural Resources.

use contributes to loadings of several priority pollutants. Substances contributed by coal and wood combustion, plastics, and preserved wood may be difficult to control at their source.

Construction Site Erosion

Construction site erosion is the most significant potential source of sediments to Lilly Creek. From 1985 to 2010, it is expected that 1,612 acres, or 45 percent of the total subwatershed area, will be converted from rural to urban use. Construction activities typically involve soil disturbance, the destruction of the vegetative cover, and changes in surface topography and drainage. In particular, the clearing and grading of construction sites subjects the soils to high erosion rates. Erosion rates from construction sites are typically 10 to 20 times higher than rates from agricultural land.[7] This excessive soil erosion frequently causes onsite construction problems, and the eroded sediment often causes sedimen-

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[7] *S. J. Goldman, K. Jackson, and T. A. Bursztynsky, Erosion and Sediment Control Handbook, McGraw-Hill Book Company, 1986.*

tation problems in downstream areas. The sediments are frequently deposited in storm sewers, culverts, drains, and waterways, decreasing their capacities and clogging them, sometimes causing flooding problems. Furthermore, erosion of the soil from the site is, in many cases, a loss of a valuable natural resource.

These high sediment contributions also contain nutrients which may increase algal growths, reduce water clarity, deplete oxygen supplies, lead to fish kills, and create odors. Ecological damages to nearby streams often include erosion of streambanks and destruction of streambank vegetation, the sediment covering of benthic fauna and fish spawning sites, the filling of stream pools, and increased turbidity, which reduces instream photosynthesis and overall stream productivity.

Streambank Erosion

The energy of flowing water in a stream channel is dissipated along the stream length by turbulence, streambank and bed erosion, and sediment resuspension. In general, increased urbanization may be expected to result in increased stream flow rates and volumes, with potential increases in streambank erosion and bottom scour. Streambank erosion destroys aquatic habitat, spawning, and feeding areas; contributes to downstream water quality degradation by releasing sediments to the water; and provides material for subsequent sedimentation downstream, which, in turn, covers valuable benthic habitats, impedes navigation, and fills downstream stormwater storage basins, wetlands, ponds, and lakes. These effects may be mitigated by utilization of proper stormwater management practices.

In 1985 and 1989, the Wisconsin Department of Natural Resources conducted surveys of streambank erosion in the Lilly Creek subwatershed. The stream surveys identified streambank erosion problems and estimated the following: the stream length affected, the height of the eroding streambank, the lateral recess, or erosion rate, of the bank, and the weight of sediment lost. About 14,000 linear feet of streambank were estimated to be eroding, resulting in the annual loss of about 5,800 cubic feet of sediment weighing approximately 352 tons. Reaches with eroding streambanks as identified by the Department surveys and by the Village of Menomonee Falls are shown on Map 6.

Atmospheric Contributions

Pollutants may also be contributed directly to surface waters through airborne emissions and subsequent dry fallout and washout. Atmospheric sources may be important contributors of sediment, nutrients, metals, and toxic organic substances. The total suspended particulate loading from the atmosphere in urban areas is up to 50 percent higher than in rural areas.[8] These particles also act as carriers for other pollutants.

Important nutrients contributed by the atmosphere are phosphorus and nitrogen. Windblown soil is the major source of phosphorus in dry fallout.[9] Particles containing phosphorus are also washed out by precipitation. Total phosphorus concentrations in rainwater are typically two to three times higher than the levels which can cause eutrophic conditions in lakes. Oxides of nitrogen may react with sodium, potassium, and other metals to form soluble nitrates which, when washed from the atmosphere, may contribute to the fertility of surface waters. Nutrient loadings from the atmosphere are usually highest in spring and summer, precisely when nutrient contributions may have the most significant impact on aquatic plant growth.

Atmospheric loadings are also important sources of metals, primarily lead, zinc, and cadmium.[10] A major source of lead is from the exhaust of automobiles burning leaded gasoline. However, the increasing use of unleaded gasoline has resulted in a corresponding decrease in dissolved

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[8] *International Joint Commission, The IJC Menomonee River Watershed Study, Volume 8, Atmospheric Chemistry of Lead and Phosphorus, December 1979.*

[9] *U. S. Environmental Protection Agency, Determination of Atmospheric Phosphorus Addition to Lake Michigan, EPA-600/3-80-063, July 1980.*

[10] *International Joint Commission, The IJC Menomonee River Watershed Study, Volume 6, Dispersibility of Soils and Elemental Composition of Soils, Sediments, and Dust and Dirt from the Menomonee River Watershed, December 1979.*

lead concentrations in surface waters.[11] Lead, like most metals, has an affinity for very small particles.

Atmospheric sources also contribute to loadings of toxic organic substances such as polychlorinated biphenyls and polycyclic aromatic hydrocarbons (PAHs). PCBs, which are insoluble, are usually associated with extremely small particles, from 0.002 to 0.1 micron in diameter.[12] PCB loadings from the atmosphere are highest near industrial areas in an order of magnitude higher than in rural areas. Although production of PCBs is now banned, much of the present input of PCBs results from the low-temperature incineration of solid wastes that contain PCBs.[13] PAHs are released to the atmosphere as a byproduct of man-made combustion processes.

Leaks and Spills of Industrial Materials

Leaks and spills of industrial materials may be directly discharged to waterways or the materials may be transported to the waterways via stormwater surface runoff and groundwater flow. These materials often contain toxic metals and organic substances which destroy streambank vegetation, contaminate bottom sediments, and harm fish and aquatic life. Contaminated bottom sediments may act as a residual source of the toxic substances, causing long-term effects which persist for years after the occurrence of the spill or leak.

Within the Lilly Creek subwatershed, the Bowling Green Industrial Park, located just north of the Chicago & North Western railway and west of Lilly Creek, contains a number of facilities which have had waste material handling and storage problems. After industrial spills were identified in 1984,[14] the Wisconsin Department of Natural Resources conducted a 1985 investigation of waste storage and disposal procedures within the industrial park. That investigation, completed in November 1985, identified several waste storage and disposal problems, including direct discharge of untreated industrial wastewater; spills of oil, solvents, and transmission fluid; sewage leaks; and discharge of concrete wash water.[15] Voluntary corrective measures, as well as enforcement actions, were taken to control these pollution sources. However, occasional problems with spills and leaks within industrial areas in the subwatershed remain.

Malfunctioning Onsite Sewage Disposal Systems

An onsite sewage disposal system may be a septic tank system or a holding tank. In 1992, there were approximately 66 sewage holding tanks in use in the subwatershed. By the end of 1992, it is anticipated that 47 of those holding tank users will be connected to sanitary sewers and the sewage treated by the Milwaukee Metropolitan Sewerage District sewage treatment plants.

At the request of the Village of Menomonee Falls, the Waukesha County Department of Health conducted an evaluation of onsite sewage disposal systems in the Lilly Creek area in 1983.[16] The Department of Health found that, of 334 septic tank systems surveyed, 40 percent were saturated, lying in soils with a high groundwater or slow permeability, and about 36 percent were failing. In addition, several

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[11] *R. B. Alexander and R. A. Smith, "Trends in Lead Concentrations in Major U. S. Rivers and Their Relation to Historical Changes in Gasoline-Lead Consumption," Water Resources Bulletin, Vol. 24, No. 3, pp. 557-568, June 1988.*

[12] *International Joint Commission, The IJC Menomonee River Watershed Study, Volume 9, Atmospheric Chemistry of PCBs and PAHs, March 1980.*

[13] *U. S. Environmental Protection Agency, Toxic Substances in the Great Lakes, EPA 905/9-80-005, June 1980.*

[14] *W. Wawrzyn, Investigation of Pollution Sources in Lilly Creek, Menomonee Falls, Waukesha County, Wisconsin Department of Natural Resources, Memorandum to File, August 27, 1984.*

[15] *R. Klett, Final Status Report-Bowling Green Industrial Park Environmental Evaluation, Wisconsin Department of Natural Resources, Memorandum to Gloria McCutcheon, Southeast District Director, November 13, 1985.*

[16] *G. A. Morris, Director, Environmental Health Services, Waukesha County Department of Health, letter to Max Vogt, Director of Public Works, Village of Menomonee Falls, April 12, 1983.*

residences were discharging laundry and floor drain wastes to the ground surface. Saturated and failing septic tank systems contribute bacteria, organic matter, and nutrients to the groundwater and to surface waterways. The ongoing program for providing sanitary sewer service throughout the subwatershed by 1992 will eliminate these malfunctioning septic tank systems.

EXISTING NONPOINT SOURCE POLLUTION CONTROL FACILITIES AND PROGRAMS WITHIN THE SUBWATERSHED

Under existing conditions, control of nonpoint source pollutants within the Lilly Creek subwatershed is accomplished through the filtering and infiltration effects of roadside drainage swales which serve about 31 percent of the subwatershed area; through weekly sweeping of arterial streets from March through November; through sweeping of collector streets in the spring of the year; through catch basin cleaning; through leaf collection in those areas of the subwatershed with urban street cross sections; and through an administrative procedure whereby construction erosion control measures are required on a project by project basis during land development. In addition, the adverse impacts of road salt on water quality are limited by the Village's ice and snow removal policy of applying salt only on arterial streets, including Lilly Road and W. Appleton Avenue, and applying a salt-sand mixture on other streets. The ongoing program of connecting private sewage systems to the Lilly Creek trunk sewer will essentially eliminate those systems as nonpoint sources of water pollution. Several dry detention basins have been constructed to collect runoff from developing areas and to reduce peak downstream flows, but those basins would not be expected to provide significant reductions in nonpoint source pollutants.

DESCRIPTION AND ASSESSMENT OF EXISTING WATER QUALITY AND BIOLOGICAL CONDITIONS

Stormwater management planning efforts require the evaluation of existing water quality conditions and of the relationship of those conditions to existing biological communities. This section discusses the existing water quality conditions in Lilly Creek based on the available data, which are quite limited. However, relatively extensive biological surveys have been conducted since 1984 by the Wisconsin Department of Natural Resources. Survey results summarized herein address fishery resources, bottom-dwelling organisms, and aquatic habitat conditions.

Water Quality Conditions

Few water quality samples have been taken from Lilly Creek. A single water quality sample taken during the spring of 1984 from Lilly Creek at W. Appleton Avenue indicated that the dissolved oxygen concentration was 13.6 milligrams per liter (mg/l) and the temperature was 45°F (7°C).[17] While both values are suitable for fish and aquatic life, temperature and dissolved oxygen problems, if they existed, would be more likely to occur during the summer.

Bacterial measurements made in Lilly Creek at W. Appleton Avenue in the summer and fall of 1985 indicated fecal contamination.[18] Fecal coliform levels ranged from 320 to 17,000 membrane filter fecal coliform counts per 100 milliliters (ml), fecal streptococcus levels ranged from 140 to 29,000 counts per 100 ml, and enterococcus levels ranged from 110 to 13,000 counts per 100 ml. In general, the bacteria levels were higher in fall than in summer. The ratio of the fecal coliform to fecal streptococcus levels indicated that both animal and human wastes were the source of the contamination. The bacteria levels measured were, in general, higher than the levels which can be considered safe for full or partial body contact in recreational uses.

A groundwater seep in the Bowling Green Industrial Park was sampled for bacterial contamination and several volatile organic substances.[19] Eight volatile organic substances

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[17] *Wisconsin Department of Natural Resources, Lilly Creek Stream Classification, Revised Draft, March 1985.*

[18] *Wisconsin Department of Natural Resources, Bacteria Report for the North Branch, East-West Branch, and Menomonee River Watershed, The Milwaukee River Priority Watershed Project, 1985.*

[19] *Wisconsin Department of Natural Resources, Lilly Creek Stream Classification, Revised Draft, March 1985.*

were measured in the sample. While the measured value did not exceed acute toxic standards for aquatic life, these substances may have chronic effects on some organisms. The fecal streptococcus level, 510,000 counts per 100 ml, was very high; the probable source of the contamination was animal waste.

Fishery Resources

The fish community in Lilly Creek is diverse and abundant. In 1984, the Creek had the greatest diversity, or number of species, of all of the Menomonee River tributaries. In general, due to increased flow and higher quality habitat, the abundance and diversity of fish was better in the downstream portions of the Creek than in the upstream portions. Water quality, flow, and habitat conditions are suitable to support the successful propagation of several fish species, including blacknose dace, stoneroller, creek chub, white sucker, and northern pike. Within the Menomonee River watershed and downstream of the Menomonee Falls dam, Lilly Creek represents one of only two streams, the other being the Little Menomonee River, where northern pike could spawn successfully.

Table 15 summarizes the fish species surveyed in Lilly Creek in 1985. Of the 14 species identified, five species were classified as sport fish; two species as intolerant of pollution; four species as tolerant of pollution; and three species as very tolerant of pollution. The fish community was dominated by the intolerant blacknose dace, the tolerant creek chub, and the tolerant common shiner.

Benthic Organisms

Benthic macroinvertebrates are bottom-dwelling organisms important as fish food and also serve as an indicator of overall water quality conditions. Benthic macroinvertebrates were collected in Lilly Creek just upstream of W. Appleton Avenue in May 1984. The bottom substrate at this location was 60 percent gravel, 30 percent sand, and 5 percent each silt and clay.

Midge larvae dominated the benthic community, accounting for 93 percent of the organisms sampled. The most dominant genera were Cricotopus, followed by Orthodadius and Thienemannimyia. The pollution-tolerant organism Asellus intermedius was the only nonmidge species identified.

A procedure known as the Hilsenhoff Biotic Index, based on the benthic invertebrates present, was used to classify overall water quality conditions. The Index calculations indicated that Lilly Creek at W. Appleton Avenue had poor water quality.

Aquatic Habitat

The aquatic habitat consists of those physical and biological characteristics of a surface water which determine its potential for supporting different communities of organisms. In 1984, the Wisconsin Department of Natural Resources surveyed the habitat of the main stem of Lilly Creek.

Upstream of Lilly Road, the fish and aquatic life habitat of Lilly Creek was limited by extremely low stream flows, and by less than desirable bottom substrates. Substrates were primarily clay and silt, with some coarse sand and gravel. Bank vegetative cover was dominated by trees and shrubs and lesser amounts of grasses. Extensive shading by trees and shrubs limited the establishment of bankside grasses. Stream channel widths ranged from five to eight feet, and water depths in this segment ranged from 0.25 foot in riffle runs to two feet in some pools. This stream segment was characterized by the Department as a shallow run in the lower reaches and marshy run in the upper reaches. Additional instream habitat is provided by woody debris and thalweg contours. Bank erosion was not severe, although there was some bank scouring. Overall habitat for fish and aquatic life in this segment was rated by the Department as fair to poor.

Downstream of Lilly Road to W. Appleton Avenue, the fish and aquatic life habitat improved. The substrate consisted of compact clay reaches, sand, coarse sand, gravel, and lesser amounts of rubble. The Creek was approximately three feet wide and meandered slightly within its channelized upper banks. This segment was primarily a series of riffles and shallow runs. Additional fish and aquatic life habitat was provided by pools and undercut banks, which provide important habitat during low-flow periods. The banks were well covered with tall grasses and shrubs, resulting in stable side slopes and good wildlife cover. The reach downstream of Brentwood Drive to Good Hope Road had extensive riprap along the banks and

Table 15

FISHERY RESOURCES IN LILLY CREEK: 1985

Species	Tolerance Classification[a]	Upstream of Menomonee River River Mile 0.2	Upstream of Appleton Avenue River Mile 0.4			Nicolet Court Extended River Mile 0.9		Manor Hill Boulevard and Bay Ridge Court River Mile 1.3	Downstream of Mill Road River Mile 1.9		Upstream of C&NW Railroad Crossing River Mile 2.6
		3-18-85	5-10-84	3-18-85	9-13-85	4-5-85	9-13-85	9-13-85	5-10-84	4-5-85	4-5-85
Bluegill (Lepomis macrochirus)	WS	- -	- -	- -	2	- -	- -	- -	- -	- -	- -
Green Sunfish (Lepomis cyanellus)	WS	3	2	1	23	2	27	8	4	3	- -
Sunfish (Lepomis sp.)	WS	1	- -	- -	2	- -	5	67	- -	- -	- -
Undetermined Centrarchidae Hybrid	WS	- -	- -	- -	2	- -	1	- -	- -	- -	- -
Black Bullhead (Ictaluras melas)	WS	- -	- -	1	3	- - - -	29	- -	- -	- -	
Blacknose Dace (Rhinichthys atratulus)	IT	18	57	18	21	78	16	6	7	25	5
Stoneroller (Campostoma sp.)	IT	- -	- -	- -	19	9	- -	- -	- -	4	1
Common Shiner (Notropis cornutus)	T	61	9	1	18	65	12	16	15	- -	4
Creek Chub (Semotilus atromaculatus)	T	12	24	8	37	11	27	82	16	8	2
Bluntnose Minnow (Pimephales notatus)	T	16	- -	- -	- -	1	- -	- -	- -	- -	- -
White Sucker (Catostomus commersoni)	T	14	4	4	60	3	53	43	1	9	2
Fathead Minnow (Pimephales promelas)	VT	6	1	- -	14	2	7	52	- -	8	- -
Central Mudminnow (Umbra limi)	VT	4	1	6	34	2	6	10	- -	19	1
Brook Stickleback (Culaea inconstans)	VT	3	2	- -	9	20	4	4	- -	14	23
Tolerance Class Summary											
WS Species/Number Fish	- -	1/4	1/2	2/2	3/32	1/2	1/33	1/104	1/4	1/3	- -
IT Species/Number Fish	- -	1/18	1/57	1/18	2/40	2/87	1/16	1/6	1/7	2/29	2/6
T Species/Number Fish	- -	4/103	3/37	3/13	3/115	4/80	3/92	3/141	3/32	2/17	3/8
VT Species/Number Fish	- -	3/13	3/4	1/6	3/57	3/24	3/17	3/66	- -	3/41	2/24
Total Species/Number Fish	- -	9/138	8/100	6/39	11/244	10/193	8/158	9/317	5/43	8/90	7/38

[a] *WS - Warmwater sport*
IT - Intolerant forage
T - Tolerant forage
VT - Very tolerant

Source: Wisconsin Department of Natural Resources.

deeper holes in the channel. Habitat in the segment downstream of Lilly Road to W. Appleton Avenue was rated by the Department as fair.

Downstream of W. Appleton Avenue to its confluence with the Menomonee River, the substrate was predominantly clay and silt with some sand and gravel. The channel banks had little ground cover, making them more susceptible to bank erosion than previous segments. The channel width was approximately six feet and the average water depth was 0.5 foot. Shallow pools had formed behind riffle areas and beneath debris, where holes had been scoured. Overall, fish and aquatic life habitat in this segment was rated by the Department as fair in 1984. However, the Department has subsequently undertaken a fish habitat improvement project which involved the creation of pools and riffles through the placement of riprap deflectors along the streambank. That project has enhanced the habitat conditions within this segment.

SUMMARY

The stormwater management and flood control plan presented in this report focuses on the 5.65-square-mile Lilly Creek subwatershed. An inventory of pertinent hydrologic and hydraulic characteristics of the subwatershed and related natural and man-made features is an essential step in the stormwater management and flood control planning process. Accordingly, data on land use, land use regulations, climate, soils, the existing stormwater management and flood control system and existing drainage and flooding problems; on existing point and nonpoint sources of water pollution; on existing programs to control nonpoint source water pollution; and on water quality conditions are presented in this chapter.

Land use characteristics, including impervious area, the type of storm drainage system, the level and characteristics of human activity, and the type and amount of pollutants deposited on the land surface, greatly influence the quantity and quality of stormwater runoff. Urban land uses within the Lilly Creek subwatershed are expected to increase about 89 percent, from a total of 1,820 acres, or 50 percent of the subwatershed area in 1985, to about 3,430 acres, or 95 percent of the subwatershed area, under planned ultimate land use conditions. The residential land use category is expected to experience the largest absolute increase, about 885 acres, to a total in the plan design year of about 2,100 acres. The large increase in urban land use is anticipated due to an aggressive Village development program and to the recent extension of public water and sanitary sewer service to the subwatershed.

Attendant to this increase in urban land use is an anticipated increase in the resident population of that portion of the Village of Menomonee Falls which is located in the Lilly Creek subwatershed. The resident population of the subwatershed area is expected to increase from about 5,900 persons in 1985 to about 17,800 persons under planned ultimate conditions. The anticipated increase in population can readily be accommodated by the increase in residential land anticipated within the subwatershed over the 1985 through 2010 time period.

The anticipated change in land use will directly impact the amount and quality of stormwater runoff. Increased rates and volumes of runoff result from the higher proportion of impervious areas, such as streets, parking lots, and rooftops. Thus, urban development can increase flood flows, stages, streambank erosion, and streambed scour in downstream watercourses. Such development can also increase the downstream surface-water pollutant loadings and may reduce stream base flows. Therefore, careful planning of urban stormwater management systems to meet sound water resource and related management objectives is essential.

Existing pertinent land use regulations include zoning and land division ordinances. These land use regulations, summarized in this chapter, represent important tools for the Village of Menomonee Falls in directing the use of land in the public interest. Such zoning has important implications for stormwater management.

Climatological factors affecting stormwater management include air temperature and the type and amount of precipitation. Air temperature affects whether precipitation occurs as rainfall or snowfall, whether the ground is frozen and, therefore, essentially impervious, and the rate of snowmelt and attendant runoff. The seasonal nature of precipitation patterns is an important consideration in stormwater drainage. Flooding along the streams in the study area is likely to occur at any time throughout the year except during winter because of the relatively small drainage areas and the impacts of urban development. The maximum monthly precipitation recorded at the National Weather Service station at Mitchell International Airport in Milwaukee was 10.03 inches in June 1917 and the maximum 24-hour precipitation was 6.84 inches, recorded on August 6, 1986. The amount of snow cover influences the severity of snowmelt flood events and the extent and depth of frozen soils.

Soil properties influence the rate and amount of stormwater runoff from land surfaces. About 95 percent of the study area is covered by soils which generate moderate relatively large amounts of runoff.

For planning purposes, the study area was divided into 199 catchment areas. These catchment areas range in size from about 1 to 64 acres, with an average size of 18 acres. These areas are drained by a total of 15.54 miles of intermittent streams.

The existing storm sewer system serves a combined drainage area of about 525 acres, or about 15 percent of the subwatershed. The existing system of open drainage channels and associated culverts serves about 1,130 acres, or 31 percent of the subwatershed area.

Existing stormwater drainage problems include street, yard, and basement flooding due to ice and snow obstructions at culverts and to the inadequate hydraulic capacity of certain roadside swales and associated culverts. There are also problems with erosion along the tributaries to Lilly Creek. Under existing land use and channel conditions, flooding problems include potential street flooding and basement and first floor flooding of structures along the Creek.

Sources of water pollutants to Lilly Creek include urban and rural land stormwater runoff, construction site erosion, streambank erosion, atmospheric contributions, industrial material leaks and spills, and malfunctioning septic tank systems. There are no known point sources of pollution which discharge to Lilly Creek or its tributaries.

Very few water quality samples have been taken from Lilly Creek. The very limited data available indicate that portions of the stream are contaminated with bacteria from both human and animal waste sources. Toxic organic substances were measured in a tributary to Lilly Creek which drains industrial land. At present, there is no evidence of inadequate dissolved oxygen levels, or of excessive temperature or nutrient levels.

When surveyed in 1984, the fishery resources in Lilly Creek were abundant and diverse; 14 species were identified. Several fish species successfully propagate in the Creek. The benthic, or bottom-dwelling, organisms present in the Creek were dominated by pollution-tolerant species and were representative of poor water quality conditions. In general, the aquatic habitat was rated as poor to fair in the reaches upstream of Lilly Road and as fair in the downstream reaches.

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Chapter III

STORMWATER MANAGEMENT AND FLOOD CONTROL SYSTEM COMPONENTS

INTRODUCTION

A stormwater management and flood control system plan seeks to combine drainage, water quality management, and flood control system components in a manner which will meet agreed-upon stormwater management and flood control objectives in a cost-effective manner. This chapter describes, to the extent required for system planning purposes, stormwater management and flood control system components and the function of these components within a stormwater management and flood control system. Each component or element is described, its function identified, and its relationship to the overall system discussed.

DRAINAGE SYSTEM COMPONENTS

There are two distinct drainage systems to be considered in the development of the stormwater management element of this system plan: the minor system and the major system. The minor stormwater drainage system is intended to minimize the inconveniences attendant to inundation from more frequent storms, generally up to the 10-year recurrence interval storm event. The minor drainage system consists of drainage swales in sideyards and backyards, street curbs and gutters, roadside swales, storm sewers and appurtenances, and some storage facilities. It is composed of the engineered paths provided for stormwater runoff to reach receiving streams and watercourses during these more frequent storm events.

The major stormwater drainage system is designed for conveyance and storage of stormwater runoff during major storm events, that is, generally, for storms exceeding the 10-year recurrence interval, when the capacity of the minor system is exceeded. The major stormwater drainage system consists of the entire street cross-section and interconnected drainage swales, watercourses, and stormwater storage facilities. Portions of the streets, therefore, serve as components of both the minor and major stormwater drainage systems. When providing transport of overland runoff to the piped storm sewer system, the streets function as a part of the minor drainage system; when utilized to transport overflow from surcharged pipe storm sewers and culverts and overflowing roadside swales, the streets function as a part of the major drainage system. Major drainage system components must be carefully studied to identify areas subject to inundation during major storm events.

The minor and major stormwater management systems are comprised of four basic types of facilities: overland flow, collection, conveyance, and storage. The storage component of the stormwater quantity management system may also perform a water quality management function. These five components are discussed below, followed by a discussion of various structural and nonstructural flood control system components.

Overland Flow

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

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

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

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

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

Because overland flow has a broad impact on the overall system objectives, it was considered to be an important and essential component of the stormwater management system for the Lilly Creek subwatershed. Specific arrangements for overland flow, however, cannot be addressed at the systems level of planning. The design of such arrangements must be done on a site-specific basis as urban development or redevelopment takes place, and especially during the land subdivision process attendant to urbanization. Overland flow was considered in the systems planning process, however, through the development of the general guidelines set forth in Chapter IV, which includes a description of practical techniques for minimizing the rate and volume of runoff. In evaluating alternative stormwater management systems it was assumed that these general guidelines will be followed to the extent practicable either as land is converted from rural to urban uses or as existing urban uses are redeveloped.

Collection

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

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

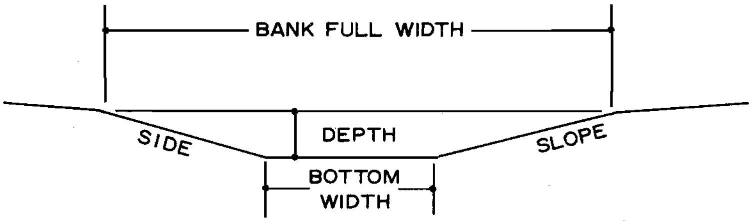
Drainage Swales: A stormwater drainage swale is a small depression, or valley, in the land surface. The purpose of a drainage swale is to collect overland flow from areas such as frontyards, sideyards, and backyards, and to transmit it to larger, open stormwater drainage channels or to subsurface conveyance facilities. Drainage swales are generally grass-lined, but may be paved to prevent erosion on steep slopes or to avoid standing water on flat slopes. A typical drainage swale is shown in Figure 2.

Drainage swales cannot be specifically addressed at the systems level of planning. The

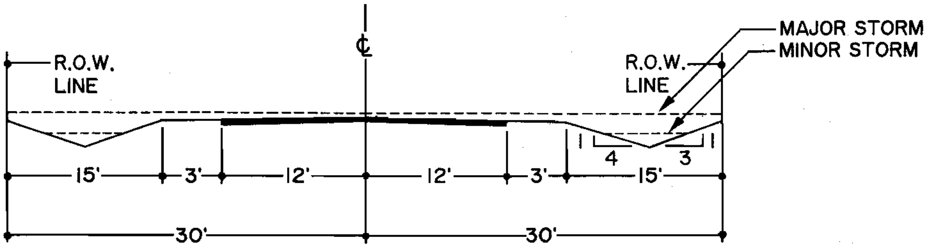
Figure 2

TYPICAL SWALE AND ROADWAY CROSS-SECTIONS SHOWING WATER COLLECTION AREAS

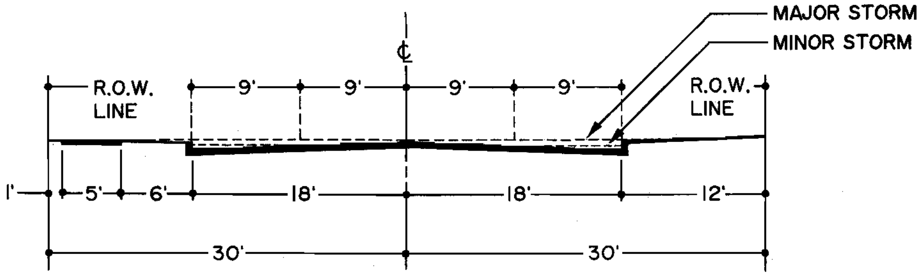
DRAINAGE SWALE



ROADWAY WITH ROADSIDE SWALE



ROADWAY WITH CURB AND GUTTER



Source: Village of Menomonee Falls and SEWRPC.

design of such components must be done on a site-specific basis as urban development or redevelopment takes place. The design of swales, then, like the design of overland flow, is considered in the systems planning process using the detailed design criteria provided in Chapter IV of this report.

Roadside Swales: A roadside swale is a long, narrow, shallow depression or valley running parallel and adjacent to a roadway providing longitudinal drainage. Roadside swales in urban areas are generally grass-lined, but may also be paved to prevent erosion on steep slopes or to avoid standing water on flat slopes. The roadside swale can serve as either a collection component or a conveyance component, or a combination of such components, of the stormwater management system. A typical residential roadway and swale combination is shown in Figure 2. The swale collects stormwater runoff from the roadway surface and the tributary overland flow areas of abutting lands. The collected stormwater is then transmitted to open channel or subsurface conveyance facilities. Roadside swales generally have lower capital costs but higher operation and maintenance costs than curb-and-gutter collection systems. They also provide lower runoff velocities and can provide for stormwater infiltration and add storage capacity. Nonpoint source water pollution loadings carried by stormwater are generally reduced as flows are collected in swales. Partial or full paving of swales may reduce or eliminates nonpoint source pollution control benefits via infiltration, and thus should be avoided where possible. Through the use of roadside swales, stormwater runoff can be managed entirely in a surface drainage system and the construction of storm sewers can be avoided. Such surface drainage systems are most practical in areas developed at relatively low densities, since each intersecting private driveway, as well as each public roadway, must be provided with a culvert pipe to carry the drainage. As densities increase, lot areas and widths decrease and front yard setbacks decrease, and a point is reached at which the provision of a storm sewer becomes more economical, desirable, and maintainable than the provision of roadside swales and culverts. The use of roadside swales provides a "rural," "suburban," or "estate" appearance and is desired by some communities for this reason. Roadway pavements with roadside swales are often called ribbon pavements.

Under some conditions, as, for example, very close driveway culvert spacing or minimum longitudinal gradient, culvert headwater elevations and entrance losses may dictate the design. In areas with limited right-of-way, a rectangular, concrete-lined channel may be required. In other reaches, the channel can more typically be triangular or trapezoidal in shape with grassed bottom and side slopes. In areas of minimum longitudinal gradient, a paved channel bottom may be necessary. The stormwater management plan assumes the use of roadside swales with a cross-section similar to that shown in Figure 2 in certain areas of the subwatershed. Systems level design criteria for roadside swales are provided in Chapter IV.

Roadway Curbs and Gutters: A roadway curb and gutter is a low vertical surface with attendant depression in the roadway cross-section adjacent to the curb line. A typical residential roadway configuration with curb and gutter is shown in Figure 2. The roadway gutter collects stormwater from the roadway surface and from the tributary overland flow areas of abutting lands. The collected stormwater is typically discharged from the roadway gutters into stormwater inlets or catch basins that transmit the stormwater to subsurface conveyance facilities. Curbs and gutters are required in higher density urban areas where the use of roadside swales and culverts becomes impractical. Curbs and gutters reduce the potential for stormwater infiltration, increase stormwater runoff flow velocity, and limit the removal of nonpoint source water pollution loadings.

This stormwater management plan assumes the use of a typical roadway cross-section with curb and gutter similar to that shown in Figure 2 in certain areas of the subwatershed.

It is important to note that curbs and gutters serve certain other improvement functions in addition to the drainage function. Curbs and gutters perform a structural function in supporting pavement edges and protecting those edges against the effects of traffic and moisture. Water seeping into the pavement and subbase along the unprotected pavement edges can shorten pavement life and increase maintenance costs. Curbs and gutters also perform a safety function, defining the pavement edge for drivers and pedestrians; help protect street lights, fire hydrants, and signs from damage by vehicles; and help to keep dirt and litter contained on pavement surfaces where mechanical sweepers can collect it. Such dirt and litter must be collected by hand on unpaved shoulders. Finally, roadway cross-sections with curbs and gutters require less right-of-way than such sections with roadway ditches.

Stormwater Inlets: The stormwater inlet is a device through which stormwater is transmitted from the surface collection facilities to subsurface conveyance facilities. Stormwater inlets are placed at strategic locations along drainage swales, roadside swales, and gutters to transmit collected stormwater into subsurface conveyance facilities. The inlet structure includes a stormwater grate, a drop structure, and a connection to the underground conveyance facility.

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

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

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

3. The combined curb inlet and gutter inlet, which is referred to as a combination inlet. That type is the standard inlet used in the Village of Menomonee Falls.

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

Catch Basins: A catch basin is defined as a stormwater inlet equipped with a small sedimentation basin, or grit chamber. The purpose of a catch basin is to remove sediment and debris from stormwater before it is transmitted to the subsurface conveyance facilities. Stormwater enters through the surface inlet and drops to the lower basin area. Heavy sediment particles and other debris are collected in the basin area. This debris is then removed during maintenance operations. The Village encourages catch basin installation for new storm sewers and has a program for catch basin cleaning and repair. The catch basin is designed to reduce the maintenance requirements for the underground conveyance system, particularly in areas where heavy sediment loads may otherwise be carried into the conveyance system. The use of catch basins fell into disfavor because of the cost of the periodic cleaning required. Nonpoint source pollution abatement, however, may warrant the reintroduction of the catch basin in urban areas. Typical catch basin installations are given in the Standard Specifications for Sewer and Water Construction in Wisconsin.

Properly maintained, the catch basin is an effective sediment trap. Improperly or inadequately cleaned catch basins may have a negative impact on receiving water quality. Decaying organic material trapped in the basin may produce noxious odors and the basin water may become rich in organic material and nutrients and low in dissolved oxygen content. This basin water becomes a part of the first flush of stormwater from subsequent storm events. Basin waters may also provide a place for mosquitoes to breed. Thus, improperly cleaned and maintained catch basins are not beneficial components of the overall stormwater management system.

Collection Elements Applicable to the Lilly Creek Subwatershed Stormwater Management System: The general policy of the Village of Menomonee Falls is to encourage the provision of full curb and gutter and storm sewers for the collection of stormwater in commercial areas and in areas of new medium- and high-density residential development with lot frontages of less than 120 feet. In the preparation of the stormwater management and flood control plan, consideration was given to the use of both an urban street cross-section with a curb and gutter collection system and a rural street cross-section with roadside swales and culverts.

Conveyance

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

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

Open Channel Conveyance: Open channel conveyance facilities generally follow the natural surface drainage pattern. In some instances, the natural channel configuration can be maintained with only minor modifications, such as removal of obstructions and reducing the overall channel roughness. In certain areas it may be necessary to modify the existing channel by widening, deepening, and realigning, or to construct an entirely new channel, in order to provide the required conveyance capacity. Manmade open channel conveyance facilities may be lined with grass, concrete, riprap, or composite material, depending on the need to prevent erosion or to avoid standing water. Typical open channel cross-sections are shown in Figure 19 in Chapter IV.

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

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

The locations and sizes of significant existing and proposed culverts in the Lilly Creek subwatershed are set forth in the stormwater management system plan. A discussion of the hydraulic conditions affecting culvert discharge and of culvert design criteria is given in Chapter IV.

Storm Sewer Conveyance: A storm sewer is defined as an underground conduit that transports stormwater runoff from collection facilities to an ultimate point of disposal. The purpose of a storm sewer is to receive stormwater runoff from stormwater inlets and catch basins and convey that runoff to surface water drainage facilities. The storm sewer provides a rapid conveyance route for stormwater to a point of disposal on a receiving on-surface watercourse. Subsurface storm sewer systems are generally more costly to construct than surface conveyance facilities; however, they are often required in order to meet stormwater management objectives.

Prefabricated Portland cement concrete pipe is the most commonly used material for the construction of storm sewers in the Village of Menomonee Falls and in the Region. Concrete pipe is commercially available in standard lengths ranging from four feet to eight feet and in circular, elliptical, and arch pipe sections, with circular sections ranging from six inches to 108 inches in diameter. Nonreinforced concrete pipe is commercially available in diameters ranging from six inches to 18 inches, while reinforced concrete pipe is commercially available in diameters ranging from 12 inches to 108 inches. Fittings for concrete pipe, such as wyes, tees, and manholes, are readily available. Concrete provides a high-strength, widely used and accepted storm sewer pipe. Prefabricated galvanized steel pipe, such as corrugated metal pipe and corrugated metal pipe arch, is also used in stormwater management systems. The most common application of these materials is in culvert installations but, in some cases, corrugated metal pipe is used for storm sewer construction. Corrugated metal is light weight, strong, and flexible, and is manufactured in generally longer lengths than is concrete pipe. It is more difficult to connect inlets to corrugated metal pipe. Polyvinyl chloride (PVC) pipe, also referred to as plastic pipe, is light in weight, manufactured in generally longer lengths than concrete pipe, and more hydraulically efficient than corrugated metal pipe. Although most readily available in diameters up to 12 inches, PVC pipe can be obtained in diameters up to 30 inches. There is only limited experience with PVC pipe for storm sewer applications and thus its long-term performance characteristics are not known. Criteria for the hydraulic design of storm sewers are provided in Chapter IV. Other pipe materials, such as asbestos-cement pipe, vitrified clay pipe, ductile iron pipe, and welded steel pipe, are also available. These materials are not commonly used for gravity-flow storm sewers in the Region. There are limited applications for asbestos-cement pipe and ductile iron pipe as pressure stormwater conveyance facilities.

Manholes: A manhole is a structure which provides access to the storm sewer system for observation and maintenance purposes. Manholes are typically placed at all junctions in the sewer system, at changes in horizontal or vertical alignment, and from 300 to 600 feet apart along the sewers. Smaller sewers are normally laid in straight lines between manholes; larger sewers may be laid on curves. Greater manhole spacing distances are allowable for sewers large enough to allow entrance by maintenance personnel to the sewer itself. Junctions for smaller storm sewers can be accommodated in ordinary manholes. Larger sewers, however, may require the provision of special junction chambers. Typical storm sewer manhole designs are given in the Standard Specifications for Sewer and Water Construction in Wisconsin.

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

Junction Chambers: A junction chamber is a structure which provides access to an underground sewer and accommodates major changes in the size, alignment, or number of storm sewers. Typically, they are unique, cast-in-place, reinforced concrete vaults.

The approximate locations of junction chambers are set forth in the stormwater management plan. The type of junction chamber is dependent on the sewer sizes and alignment conditions at each point in the system. Accordingly, the details of any proposed junction chamber must be determined in the detailed design phase preceding construction.

Conduit End Structures: A conduit end structure is a structure used to make the transition between a culvert or storm sewer and a swale, channel, or other surface watercourse. The primary purpose of an end structure is hydraulic control and efficiency. This includes preventing scour before the pipe inlet, preventing scour and undermining beyond the pipe outlet, and providing a hydraulically efficient pipe entrance. Conduit end structures also provide structural support for the pipe end and stabilization and protection of the embankment slope. The end structure provides protection from, and dissipation of, the excess energy cause by the velocity change and turbulence associated with these flow transitions. Typical end structures are given in the Wisconsin Department of Transportation Facilities Development Manual.

The details of any end structure must be determined on a site-specific basis in the detailed design phase preceding construction.

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

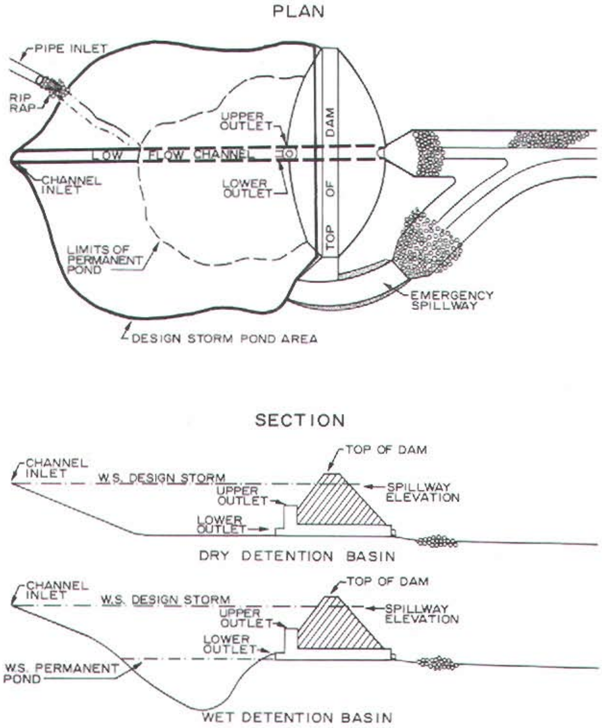
Pumping stormwater from storage areas is less dependable and more costly than gravity drainage. Electrical service can be interrupted, especially during thunderstorms. Maintenance of stormwater pumping facilities is a significant concern, since these facilities require periodic inspection and maintenance. Where deep storage is required, or where the grade is not sufficient to provide adequate gravity drainage, pumped discharge is necessary.

Storage

Stormwater storage can be defined as both the temporary detention and the long-term retention

Figure 3

TYPICAL STORMWATER
DETENTION STORAGE STRUCTURES



Source: SEWRPC.

of stormwater within the system. The primary purpose of stormwater storage is to reduce the peak stormwater discharge rates both within the stormwater management system itself and in the receiving waterways. Stormwater storage also allows greater infiltration of stormwater, recharging the groundwater reservoir; it reduces flow velocity and thus the potential for stream erosion; it enhances the removal of sediment and other particulates suspended in stormwater; and it usually reduces the cost of downstream stormwater conveyance and flood control facilities.

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

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

Detention storage is the temporary storage of stormwater accompanied by controlled release. The purpose of detention storage is to hold back, or delay, stormwater runoff temporarily to reduce the peak rate of stormwater runoff from the drainage area. A dry detention basin normally drains completely between spaced runoff events. A wet detention basis temporarily stores floodwaters on top of a permanent pool of water used for other purposes. Typical dry and wet detention basins are shown in Figure 3.

Retention storage is the long-term storage of stormwater without release to the surface water drainage system in order to remove stormwater from the surface drainage system and to allow it to infiltrate or evaporate, thus reducing the overall volume of stormwater that reaches the outfall of the drainage basin. Stormwater retention basins are often relatively shallow basins, either natural or man-made, with substantial bottom areas to allow infiltration into the groundwater reservoir.

Stormwater retention basins and wet detention basins with normal water levels at the water table elevation may serve as water supply and fire protection reservoirs and may capture stormwater for industrial or municipal uses. Retention basins and wet detention basins can also serve as recreational facilities for uses not involving body contact with the water and as aesthetic focal points in desirable "green" open spaces. Wet detention basins can be designed in series to include connecting open green areas that further enhance the overall stormwater management system effectiveness.

There are a wide variety of passive stormwater detention measures that can be used in an urban setting. They consist of grassed stormwater collection swales designed to flow at low velocities, thereby providing in-line storage; stormwater conveyance swales designed to include check dams to reduce flow velocities, thereby providing storage; and berms, also used to provide increased storage volume. Stormwater storage can also be provided on flat rooftops, in parking lots, and in specially designed and constructed stormwater storage facilities. These storage measures generally detain stormwater for short periods of time, in some cases allowing increased infiltration, evaporation, and transpiration, and can significantly reduce downstream peak stormwater discharges.

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

It is important to note that the indiscriminate location and/or phasing of construction of detention facilities within a watershed can actually increase the magnitude and duration of downstream peak flows. Such a situation occurs when prolongation of peak, or near-peak, outflows from a storage facility causes these flows to coincide with near-peak flows from upstream or downstream areas. Therefore, rather than requiring such facilities in ordinances based on broad "policy" plans, it is imperative that competent engineers experienced in this field design and evaluate such facilities on a watershedwide basis and within the context of a system plan.

It is not always desirable or feasible to provide storage in a stormwater management system. In most developed urban areas, suitable parcels of land are not readily available for the construction of stormwater retention or detention basins. Other methods of onsite storage and collection system storage may be feasible in such cases, but may cause objectionable disruption of urban activity.

URBAN NONPOINT SOURCE POLLUTION CONTROL MEASURES

Nonpoint source water pollution control is the management of urban and rural land uses to reduce the loadings of pollutants discharged to surface waters. For the purposes of this report, such control measures will be considered only with respect to urban nonpoint sources of pollution. A comprehensive discussion of the types and effects of both urban and rural nonpoint sources of water pollution is given in SEWRPC Technical Report No. 21, Sources of Water Pollution in Southeastern Wisconsin: 1975 (1978), and a more in-depth discussion of urban nonpoint sources of pollution is set forth in Evaluation of Urban Nonpoint Source Pollution Management in Milwaukee County, Wisconsin (1983) by the Wisconsin Department of Natural Resources, Southeastern Wisconsin Regional Planning Commission, and the U. S. Department of the Interior, Geological Survey. Many of the various nonpoint source pollution control measures discussed below are described in detail in the Wisconsin Construction Site Best Management Practice Handbook (April 1989) and Construction Site Erosion and Stormwater Management Plan and Model Ordinance (draft 1985) by the Wisconsin Department of Natural Resources. In recent years, increased attention has been focused on nonpoint source pollution control through the Wisconsin Priority Watersheds Program and through stormwater discharge regulations promulgated by the U. S. Environmental Protection Agency.

There are two major categories of urban nonpoint sources of pollution: the erosion of soil from disturbed land areas, especially construction sites; and pollutants transported in stormwater runoff from developed urban areas.

Construction Site Erosion Control

The primary pollutants transported through the erosion of soil from disturbed land areas are suspended sediments and sediment-attached pollutants such as phosphorus and lead. Residential, commercial, industrial, highway, and public utility construction sites all have the potential for producing large amounts of sediment which will reach receiving streams if not controlled. Because of the transitory nature of construction projects, measures to control construction site erosion and runoff are inherently of a short-term nature. Such control measures include mulching and seeding of disturbed areas, construction of filter fabric and straw bale fences to intercept eroding soil prior to discharge to a receiving stream, channel stabilization, construction of sediment traps and wet detention basins, stabilization of streambanks through provision of sod or riprap, and protection of stormwater inlets. It is feasible and desirable to deal with construction site erosion and sedimentation problems on a site-by-site basis through regulations. The proper control of erosion can be readily achieved under the provisions of ordinances which govern construction practices, allowable soil loss, and the application of certain erosion control measures. It would be difficult and potentially unsound to attempt to deal with construction site erosion through stormwater management planning conducted for a particular drainage basin. Thus, this stormwater management plan does not specifically address construction site erosion control, other than to recommend that appropriate ordinances be developed and implemented to sufficiently regulate construction activities and the attendant erosion control measures.

Control of Nonpoint Source Pollutants from Developed or Developing Areas

The second major category of urban nonpoint sources of pollution is the stormwater runoff and associated pollutants contributed from developed urban areas. As land is converted from rural to urban uses, the impervious areas is increased, different types of pollutants accumulate on the land surface, and the overall amount of pollutants is increased. During periods of rainfall or snowmelt, these pollutants are washed off the land surface and transported to receiving streams. The control of urban nonpoint source pollution requires long-term solutions which effectively reduce the loadings of those pollutants causing water quality problems and yet are flexible enough to be adapted to planned development patterns and densities. Due to restrictions on available land and the constraints imposed by existing land use patterns in developed urban areas, the range of nonpoint source pollution control measures which are applicable in developed urban areas is more limited than in developing areas, where the necessary nonpoint control measures can be anticipated and planned. The control of nonpoint sources of pollution in developed urban areas requires the preparation on a basin-by-basin basis of detailed stormwater management plans. Thus, the control of urban stormwater runoff and associated pollutants is an important element of this plan.

Nonpoint source pollution control measures appropriate for developed urban areas can be classified as either source controls or structural controls. Source controls are intended to keep pollutants out of runoff by eliminating the source of the pollutant. Structural controls are best management practices applied to remove pollutants carried by runoff. Source controls include restricted use of fertilizers and pesticides, improved pet waste and litter control, the reduced use of galvanized steel roof materials and gutters, proper disposal of motor vehicle fluids, street sweeping, leaf collection, catch basin cleaning, reduced use of street deicing salt, management of material storage areas to reduce pollutant contributions to runoff, spill control, and use of unleaded gasoline. Structural controls may include infiltration facilities, stormwater detention facilities, and physical or chemical treatment processes. Table 16 summarizes the reductions in pollutant loadings which can be achieved by various nonpoint pollution control measures. Control measures and the types of land use for which such measures are most effective are listed in Table 17.

Infiltration Devices: Infiltration systems can achieve a high level of loading reduction for both dissolved and particulate pollutants from the drainage area served, with the pollutant loading reductions being proportional to the resulting reduction in stormwater volume. Some systems, such as infiltration basins and trenches, grass filter strips, porous pavements, grass swales and waterways, and perforated drainage systems, also filter additional pollutants from the remaining runoff. Grass-lined infiltration basins and gravel-filled infiltration trenches often collect the stormwater runoff from frequent storm events from small impervious areas such as parking lots or roofs. Typical infiltration trench installations for parking lots are shown in Figure 4. Infiltration trenches are generally lined with filter cloth. Trenches may be entirely below grade, or they may be adapted to the existing topography with one side as a low berm constructed of pervious material and covered with filter cloth and small riprap. Such an installation would collect and store runoff, which would gradually be released by infiltration through the berm. Grass filter strips, which are generally placed between the pollution source and the collector system, remove pollutants in overland flow through both filtering and infiltration. Porous pavements are generally most applicable in parking areas which do not handle heavy traffic loads. Such pavements may

Table 16

EFFECTIVENESS OF URBAN NONPOINT SOURCE WATER POLLUTION ABATEMENT MEASURES

Abatement Measures	Approximate Percent Reduction of Released Pollutants					
	Suspended Solids	Phosphorus	Nitrogen	Biochemical Oxygen Demand	Metals	Bacteria
Wet Detention Basin	80-90	40-60	40-80	40-60	60-80	- -[a]
Percolation Basin	80-100	60-80	60-80	80-100	80-100	80-100
Infiltration Trench	80-100	60-80	60-80	80-100	80-100	80-100
Porous Pavement	80-100	60-80	60-80	80-100	80-100	80-100
Grass Swale	20-40	20-40	20-40	20-40	0-20	- -[a]
Grass Filter Strip	20-40	0-20	0-20	0-20	20-40	- -[a]
Stormwater Sedimentation-Flotation Basin	0-20	- -[a]	- -[a]	- -[a]	- -[a]	- -[a]

[a] *Insufficient data available.*

Source: Thomas R. Schueler, Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMPs, Metropolitan Washington Council of Governments, 1987.

consist of perforated asphalt with predominantly large aggregate, or specially constructed concrete or paving-block grids with openings for the establishment of grass cover. Grass waterways and perforated drainage systems can be effectively incorporated into the conveyance system for transport of runoff to receiving waters. Grass swales, usually placed along roadways, also reduce pollutant loadings through both filtering and infiltration.

While properly located and sized infiltration devices can substantially reduce the loadings of pollutants from nonpoint sources to receiving waters, care must be taken to avoid contamination of the groundwater. Studies have shown that particulates are effectively filtered out in the top layers of soil surrounding infiltration devices. However, dissolved pollutants may reach the groundwater when infiltration devices are improperly located in areas with unsuitable topography and soils or with a shallow depth to bedrock or to the groundwater table. Other potential adverse impacts of infiltration devices include wet basements, sump pump overloading, building and foundation failures, and excessive infiltration of clear water into sanitary sewers. Because of these potential problems, infiltration devices should be avoided in areas with a high potential for groundwater contamination and in areas of intensive urban development. These measures are best used in areas of low-density development, where problems with basements, foundations, and excessive sewer infiltration can be avoided.

Stormwater Sedimentation-Flotation Basins: Stormwater sedimentation-flotation basins, as shown in Figure 5, are designed to remove sediment and hydrocarbon loadings from parking lot runoff before they are conveyed to the storm drain network or to an infiltration device.[1] The effectiveness of such devices in removing pollutants has not been monitored in the field; however, due to their relatively small storage volumes and resultant brief retention times, they would not be expected to provide a high degree of pollutant removal. The basins require cleaning at least twice a year. Basins may be designed with or without weep holes in the sides and bottom. A basin with weep holes would theoretically provide greater pollutant removal than the standard three-chamber inlet because of exfiltration through the weep holes in the base of the sediment and oil chambers, but clogging of the weep holes with sediment may reduce the effectiveness of the basin.

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[1] *Thomas R. Schueler, Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMPs, Metropolitan Washington Council of Governments, 1987.*

Table 17

APPLICABILITY OF CONTROL MEASURES TO ABATE URBAN NONPOINT SOURCES OF WATER POLLUTION

Control Measure	Applicability for Land Use				
	Residential	Industrial	Commercial	Institutional	Open Lands
Roof Drains to Lawns	X	X[a]	X	X	--
Infiltration Basins and Trenches	X	X[a]	X	X	X
Porous Pavement	--	--	X	X	--
Perforated Drainage Systems	X	--	X	X	--
Grass Swales	X	X[a]	X	X	X
Grass Filter Strips	X	X[a]	X	X	X
Wet Detention Basins	X	X	X	X	--
Stormwater Sedimentation-Flotation Basins	--	X	X	X	--
Roof Storage	--	X	X	X	--
Street Cleaning	X	X	X	X	--
Litter and Pet Waste Control Ordinances	X	--	X	X	X
Leaf and Clippings Collection and Disposal	X	--	--	--	--
Reduced Use of Road Deicing Salt	X	X	X	X	X

NOTE: Control measures would be applicable to reduce pollutant loadings from those land uses marked with an "X".

[a] *These stormwater infiltration measures are not appropriate for manufacturing industrial areas, but may be considered on a site-specific basis for nonmanufacturing industrial areas.*

Source: SEWRPC.

Street Sweeping, Pet Waste Control Ordinances, and Leaf Collection: Street sweeping can be an effective method of urban nonpoint source pollution control under certain circumstances. A modest increase in the sweeping of residential streets throughout the sweeping season produces only marginally higher pollutant reductions.[2] Data collected during the Milwaukee Nationwide Urban Runoff Program indicated that street cleaning in residential areas typically achieved less than a 10 percent reduction in pollutant loadings.[3] Approximately 20 to 70 percent reductions in pollutant loadings from industrial areas can be achieved if parking and storage areas are

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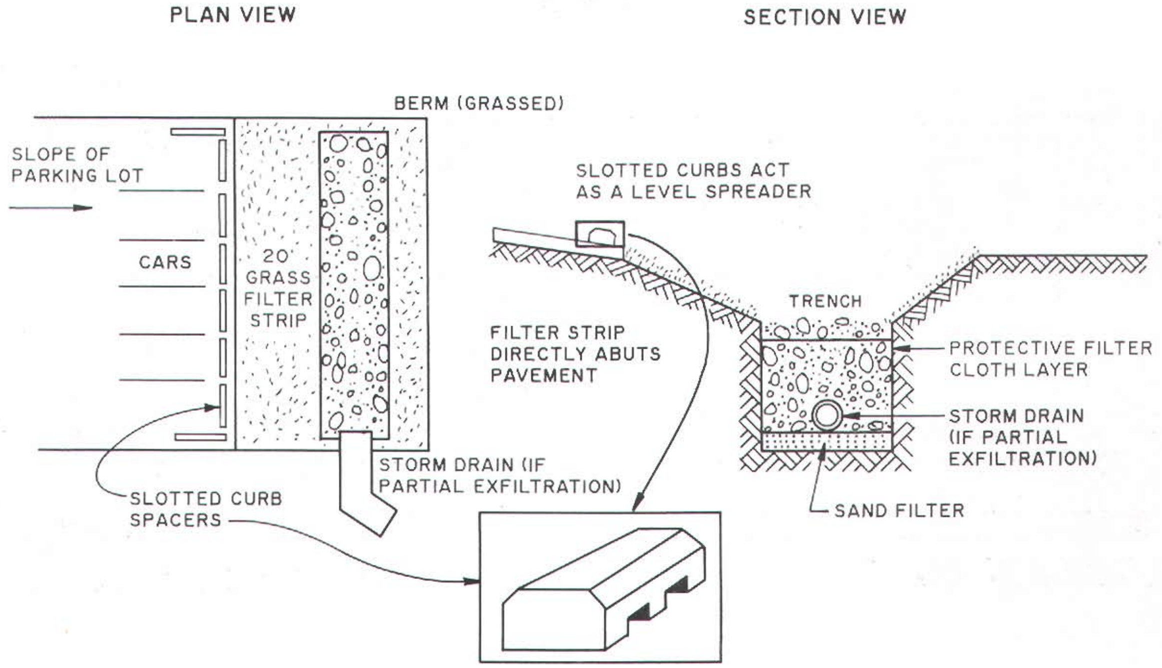
[2] *Robert Pitt, "The Incorporation of Urban Source Area Controls in Wisconsin's Priority Watershed Projects," Wisconsin Department of Natural Resources, 1986.*

[3] *Robert Pitt, Construction Site Erosion and Stormwater Management Plan and Model Ordinance (draft), Wisconsin Department of Natural Resources, May 14, 1985, revised April 23, 1987.*

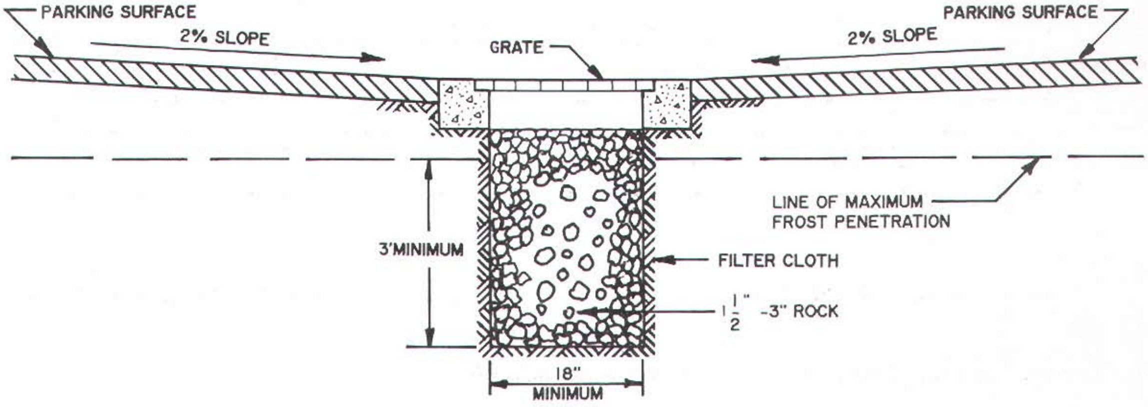
Figure 4

TYPICAL PARKING LOT INFILTRATION TRENCH INSTALLATIONS

PERIMETER TRENCH



INTERIOR TRENCH

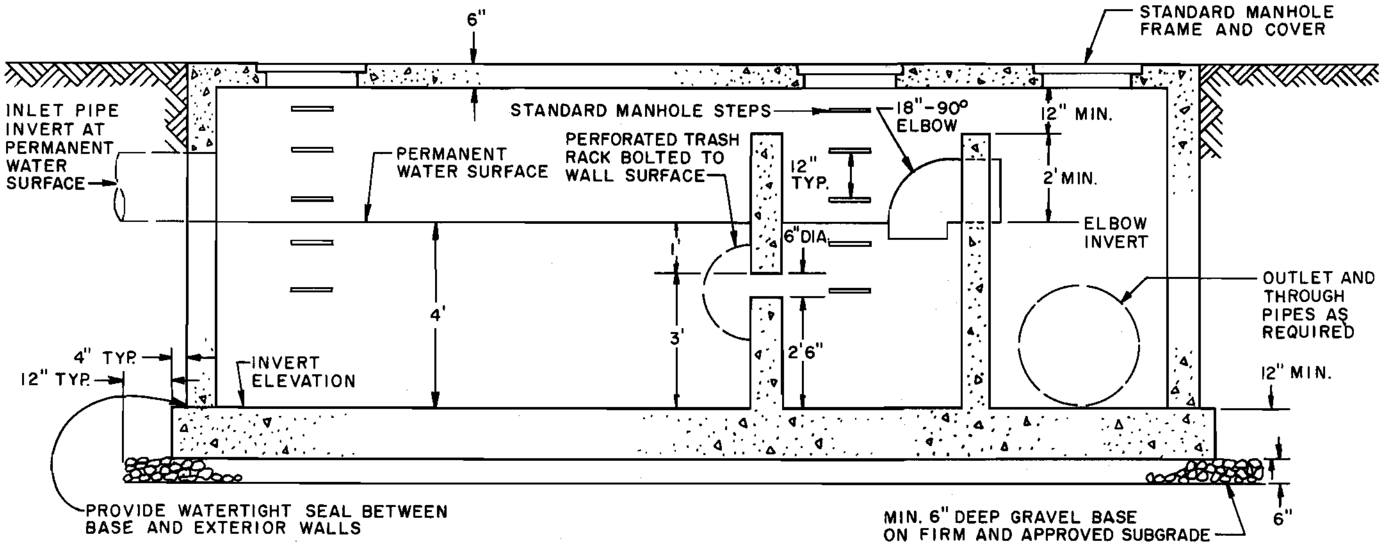


Source: Lake Tahoe Regional Planning Agency, 1978.

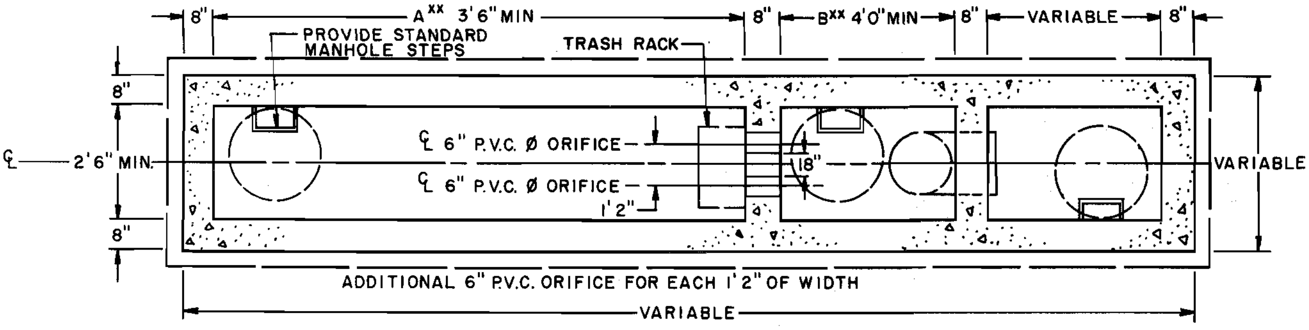
Figure 5

TYPICAL STORMWATER SEDIMENTATION-FLOTATION BASIN

SECTION



PLAN



xx WHEN COMBINED LENGTH OF OIL AND GRIT CHAMBERS EXCEEDS 12 FEET, A = 2/3 TOTAL AND B = 1/3 TOTAL.

Source: City of Rockville, Maryland; Montgomery County, Maryland; and SEWRPC.

included in the cleaning operation. Street cleaning is most effective early in spring, when the streets are laden with winter residue, and in fall, following leaf fall. Intensive street sweeping may reduce pollutant loadings during spring by up to 50 percent.[4]

Litter and pet waste control ordinances can be expected to produce pollutant reductions of only about 5 percent, as can increased leaf and clippings collection and disposal programs.

Detention and Retention Storage Facilities: As discussed previously, man-made detention and retention storage facilities and natural deep depressions and wetlands can be utilized to reduce stormwater runoff rates and volumes. Such storage areas can also produce significant reductions in nonpoint source pollutant loadings.

Along with retention, or percolation, basins and infiltration systems which are designed to store completely all tributary runoff, the wet detention basin is highly effective in reducing pollutant loadings. In wet detention basins, pollutants are removed through both sedimentation of particulates and biological assimilation of dissolved nutrients. Wet detention basins require considerable maintenance in order to function properly as nonpoint source control measures. Maintenance requirements for wet basins include mowing embankments, weed and algae control, inspection, litter removal, and periodic dredging of accumulated sediments. The cost of periodic dredging is the largest maintenance cost, but it can be reduced by confining the accumulation of most of the inflowing sediment to a settling pond at the inlet of a wet detention basin. Means of disposal of dredged sediment vary, depending on the level of contamination of the sediment. Sediments with high concentrations of toxic chemicals or metals must be disposed of in specially designed containment areas or landfills. Sediment to be dredged should be tested to determine the appropriate means of disposal.

Dry detention basins, which drain completely between flood events, are not as effective in reducing nonpoint source pollutant loadings as are wet basins. While some sediment accumulation will occur, much of it will be scoured from the bottom of the basin and discharged downstream by subsequent storm events. Dry detention basins can, however, reduce downstream bank erosion by reducing flood flows and velocities.

Roof detention storage is a measure which is sometimes proposed in areas of existing urban development. Roof drains may be retrofitted with restrictors, which permit ponding of stormwater on flat roofs, subject to the capacity of the roof to carry greater loads. The main benefit of providing roof storage of stormwater is to reduce peak rates of runoff. As with dry detention basins, roof storage can reduce erosion in localized discharge areas by reducing outflows and velocities. There are several factors which make the use of roof storage impractical as an effective stormwater management measure in southeastern Wisconsin. These include leakage into buildings of water ponded on roofs, the inability of existing roofs to carry the additional loads of rooftop ponding without structural modifications, and problems associated with freezing of ponded water and superimposed snow loads. Because of the limited applicability of roof storage, such storage should be considered for inclusion only in the design of a new structure, and where it could be demonstrated that it would be a cost-effective means for managing stormwater runoff within the context of the overall system and where the possibility of leakage and freezing problems could be addressed in the building design.

Wetlands: Wetlands can remove pollutants from stormwater runoff by sedimentation, biological assimilation, and filtration. The long flow-through times and low-flow velocities in wetlands allow suspended sediments and particulate pollutants to settle. Nutrients are assimilated by wetland plants and metals and hydrocarbons are deposited in wetland sediments. The long-term effects of toxic pollutant accumulation on the water quality and biota of wetlands has not been extensively studied. While wetlands may be effective in controlling nonpoint source pollutant loadings to downstream waters under certain conditions, the accumulation of pollutants may be harmful to the wetland ecosystem. The effects of certain nonpoint pollutants on wetlands are known. An abundance of nutrients in a wetland can lead to dominance of less desirable, nonnative plant species. Pesticides are taken up by

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[4] *Wisconsin Department of Natural Resources, Evaluation of Urban Nonpoint Source Pollution Management in Milwaukee County, Wisconsin, Executive Summary, 1983.*

certain plant species and are then released to the water column following plant decay. Because of the relatively long water retention times in wetlands, road deicing salt concentrations may exceed acceptable levels, leading to density stratification, which, in turn, may create dissolved oxygen deficiencies in the lower layers of the wetland water column. Depending on the hydrologic and hydraulic characteristics of a particular wetland, accumulated pollutants may be flushed to downstream waters during large storm events. The capacity of wetlands to remove pollutants and the long-term effects of such removal on the wetland have not been definitively established. In some cases, it may be desirable to provide facilities to reduce nonpoint source pollutant loadings prior to discharge to wetlands.

Physical/Chemical Outfall Treatment: Physical/chemical outfall treatment control measures include microscreens, dissolved air flotation, swirl concentrators, high-rate filtration, contact stabilization, and disinfection. Typically, a stormwater treatment facility would consist of a stormwater detention facility to provide a more constant flow rate followed by a physical/chemical treatment facility. The pollutant removal effectiveness of stormwater treatment facilities can range from 10 percent to more than 90 percent, depending on the treatment process and the type of pollutant removed.

Swirl concentrators are especially effective when applied to combined sanitary-stormwater sewerage systems. The process concentrates settleable solids which are then transmitted to a wastewater treatment plant. In the dissolved air flotation process, stormwater runoff is collected and air bubbles float solids to the surface, where they are skimmed off. Microscreening is used to remove fine suspended particles. The filtration process removes a large range of particle sizes through straining, impingement, settling, and adhesion. Through contact stabilization, the flow to be treated is mixed with activated sludge and the sludge then aerated in a stabilization tank, where organisms digest the organic material. Contact stabilization facilities are most efficient when the organisms in the stabilization tank are kept alive between storms; therefore, such facilities are most effectively operated in conjunction with a nearby wastewater treatment plant which will use the organisms in the treatment of dry weather flows. Disinfection is accomplished through the application of chlorine or ozone to the stormwater effluent following treatment by other means.

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

Urban nonpoint pollution sources in the Lilly Creek subbasin were evaluated by the Wisconsin Department of Natural Resources under the Menomonee River Priority Watershed Program. A mathematical water quality simulation model was applied to estimate pollutant loadings from urban nonpoint sources of pollution and to predict the effectiveness of various urban nonpoint source control measures in reducing runoff flow volumes or pollutant loadings from urban areas. To the extent practicable, the results of those analyses were refined to address site-specific conditions in the Lilly Creek subwatershed and were incorporated into the evaluation of alternative stormwater management plans and the selection of the recommended system plan for the Lilly Creek subwatershed.

FLOOD CONTROL MEASURES

The flood control element of this system plan was formulated on the basis of consideration of both structural and nonstructural floodland management measures. Those measures are intended to abate damages due to flooding from the overflow of natural streams and watercourses during floods up to, and including, the 100-year recurrence interval event. The stormwater management and floodland management measures applied within a given watershed are interdependent, and certain system components, such as some conveyance and storage facilities, may serve joint stormwater management-floodland management purposes.

Table 18 lists available structural and nonstructural measures for flood control which may be applied individually or in various combinations to portions of the streams and watercourses

Table 18

ALTERNATIVE FLOODLAND MANAGEMENT MEASURES

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

Source: SEWRPC.

within the Lilly Creek subwatershed. Structural measures tend to be more effective in achieving the objectives of flood control in already urbanized riverine areas, while nonstructural measures are generally more effective in riverine areas that have not been converted to flood-prone development but have the potential for such development. The floodland management measures set forth in Table 18 which have not been discussed previously are described briefly below. Emphasis in the description is placed on the function of each measure; on the key factors, or basic requirements, used to determine if the measure is applicable to a particular stream reach and related riverine area; and on some of the more significant general advantages and disadvantages of each measure.

Channel Modification and Enclosure

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

The function of channel modifications or enclosures is to provide a lower, hydraulically more efficient waterway through which a given flood discharge can be conveyed at a substantially lower stage relative to that which would exist under natural or prechannelized conditions. Key considerations in applying this measure include the availability of required right-of-way of sufficient width to accommodate the modified or relocated channel and the length of upstream and downstream natural channel reaches that must be modified to provide an acceptable transition from the natural channel and floodplain to the channelized or enclosed reach. As illustrated in Chapter IV of this report, channel modification and enclosure can be accomplished in a manner which permits the migration of fish in streams with a present or potential fishery of value.

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

Bridge and Culvert Alteration or Replacement

Highway and railway bridges and culverts may significantly affect upstream flood stages and downstream flood stages and discharges and thereby aggravate existing flood problems or create such problems. Bridge and culvert alteration or replacement is intended to avoid or minimize the adverse hydrologic and hydraulic effects of existing bridges and culverts on flood flows and stages. This structural measure is normally most applicable in areas where the waterway crossings are relatively old and undersized. The usefulness of this structural alternative in a watershed is contingent on identifying those bridges and culverts which produce major backwater effects as a result of inadequate hydraulic capacity and identifying those structures that are impassable during major flood events. Culvert replacement or alteration can be accomplished in a manner which permits the migration of fish in those streams with an existing or potential valuable fishery.

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

Dikes and Floodwalls

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

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

Dikes and floodwalls may create local drainage problems attributable to the accumulation of stormwater runoff which does not have access to the stream because of closed backwater gates and because either natural or man-made drainage patterns to receiving streams and watercourses are blocked by the dikes or floodwalls. Areas susceptible to the resulting inundation can be afforded protection through the provision of interior drainage systems. Such systems combine conveyance measures, or conveyance and storage measures, to transmit stormwater runoff from the landward side of dikes or floodwalls to streams or watercourses. Such systems may include all, or some, of the following elements: 1) open drainage channels, 2) cross culverts to convey stormwater under streets, highways, railways, or other embankments, 3) stormwater storage facilities, 4) storm sewers, 5) pumping stations, and 6) backwater gates. While several of these elements are generally considered to be part of the minor drainage system, their interrelationship with the major drainage and flood control systems requires that their functions be evaluated during storms in excess of the minor system design storm.

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

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

Reservation of Floodlands for Recreational and Related Open Space Uses

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

Floodland Regulations

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

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

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

Although stipulation of an essentially "no stage increase" floodway eliminates or reduces some of the problems associated with the two-district, floodway-floodplain fringe, approach to floodland regulations, several significant disadvantages remain.

Under the Department's two-district approach, filling and development of the floodland fringe area is permitted indiscriminately under specified conditions. Such filling and development may lead to a marked increase in downstream flood discharges and stages. The delineation of a floodway, by constricting the cross-sectional flow area, may also increase flood stages, thereby extending the floodplain boundary laterally and subjecting additional lands and structures to floodland regulation. Also, floodland fill for development outside the floodway limits, but within environmentally critical areas, may lead to the destruction of environmentally sensitive riverine areas.

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

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

Control of Land Use Outside Floodlands

It is important to regulate the manner in which the urban development occurs outside, as well as inside, floodlands so as to minimize the hydrologic and hydraulic impacts on floodland areas receiving runoff from tributary watershed areas. The hydrologic and hydraulic interdependence between the land surface and the streamflow regimen of a watershed suggests that areawide land use planning is an essential part of effective flood control.[5] It is important, therefore, that

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

both structural and nonstructural flood control measures be based on an areawide land use plan which considers the hydrologic-hydraulic consequences of the location of future urban development, the amount of impervious surface in that development, and the manner in which stormwater runoff from new development is controlled.

Federal Flood Insurance

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

Lending Institution and Realtor Policies

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

Utility Extension Policies

Under state regulation, sanitary sewer service may not be extended into flood hazard areas to the extent that such areas are a part of an environmental corridor.[6] Local communities may supplement this regulation by policies which prevent the extension of sewers and other public utility services, such as water supply, into any flood-prone areas. These and similar policies discourage the development of flood-prone areas and help to avoid the need to construct flood control works.

Emergency Programs

The function of an emergency program is to minimize the damage and disruption associated with flooding through a coordinated, preplanned action which is taken when a flood is impending or occurring. Such a program may include the installation of remote upstream sensors and alarms, preplanned road closures, evacuation of residents, and mobilization of portable pumping equipment to relieve the surcharge of sanitary sewers. In small watersheds, as a practical matter, the "flashy" nature of the hydrologic-

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

hydraulic system may preclude the effective implementation of any warning system as a part of the emergency program.

Structure Floodproofing and Elevation

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

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

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

Structure elevation involves raising a structure on its site so that the first floor or other most damage-prone floor is above the design flood stage. Structure raising is supplemented by basic floodproofing measures to protect the basement and other portions of the structure that remain below the design flood stage.

Basic floodproofing measures are generally considered feasible for most nonresidential structures, such as businesses, commercial buildings, and schools, even if the design flood stage is above the first-floor elevation. However, such measures generally are not technically feasible for single-family residences when the design flood stage is above the elevation of the first floor. This is the condition for which structure elevation is often the most appropriate floodproofing measure.

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

Structure Removal

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

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

ENVIRONMENTAL CONSIDERATIONS FOR STRUCTURAL FLOOD CONTROL MEASURES

The application of channel modification or flood containment measures can have many positive effects, including reducing the flood hazard to human life, reducing property damage due to flooding, increasing property values by removing existing structures from flood prone areas, and improving stormwater drainage through the provision of adequate outlet grades for storm sewers or drainage channels. Such measures can also have negative environmental impacts if they are implemented without adequate provision for mitigation of those impacts. Structural flood control measures can affect water quality, aquatic and terrestrial habitats, wetlands, groundwater resources, the aesthetics of the stream corridor, and recreation.

A stream channel formed in sand, gravel, or clay is an open hydraulic system, whose configuration, channel shape, size, and slope is determined by several interdependent factors such as the flow regimen, channel roughness, sediment load, and lining materials. Under idealized conditions, and in the absence of outside disturbances, the channel system tends toward a state of long-term equilibrium in which localized changes occur in response to natural fluctuations in flows, but the regime of the overall channel system is defined by the long-term climatologic, hydrologic, and hydraulic characteristics of the tributary watershed. When outside disturbances, such as development of land in the watershed or modification of the channel, occur, one or more of the factors determining the channel characteristics is altered. Such alteration causes adjustments of the other interdependent parameters in an effort to establish a new equilibrium. An example is the bank erosion and channel widening which occur when the channel slope is increased through straightening a channel without providing adequate measures to stabilize the bed and bank. An understanding of the responses of the stream to alterations in regime can, therefore, aid in the design of modifications which incorporate features of the original stream. In general, an alluvial stream channel consists of a low-flow stream channel, which conveys base flow and relatively frequently occurring storm flows, and a floodplain which is only inundated during more extreme floods. The critical flow for formation of the low-flow channel has been variously defined as the mean annual flow[7] and the two-year recurrence interval flow.[8] The low-flow channel, which provides the habitat for the aquatic organisms living in the stream, typically meanders to some degree and contains alternating pool and riffle sections. Riffles are shallow sections of streams which contain rocks, gravel, or other coarse substrate in which the current is swift enough to remove silt and sand. Riffles help aerate the stream and provide ideal biological habitat. Pools are deeper, slower sections of streams which provide valuable food, as well as resting and refuge areas, for fish.

Effects of Channel Modification Measures

The potential negative environmental effects of various measures associated with flood control projects are set forth in Table 19. The effects noted in the table may be classified as primary, secondary, or tertiary. For example, reductions in bank stability and increases in streambed and streambank erosion resulting from channel straightening would be primary effects, leading to the secondary effect of degradation of water quality through higher sediment loads, leading to the tertiary effect of damaging aquatic habitat.[9] Many of the adverse effects of the measures listed in Table 19 can be mitigated through incorporation of certain features of natural streams in the design of channel modifications and through the provision of instream mitigation measures for the modified channel.

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[7] *U. S. Army Corps of Engineers, Environmental Engineering for Flood Control Channels, draft, January 1987.*

[8] *Thomas R. Schueler, op. cit.*

[9] *U. S. Army Corps of Engineers, op. cit.*

Table 19

POTENTIAL NEGATIVE ENVIRONMENTAL EFFECTS OF FLOOD CONTROL MEASURES

Control Measure	Elimination of Substrate for Benthic Organisms	Reduction of Cover for Fish	Increase in Water Temperatures	Increase in Channel Photosynthesis	Degradation of Terrestrial Habitat	Reduction in Bank Stability	Scour of Streambank and Bed	Lowering of Water Tables
Snagging[a]	X	X	--	--	--	--	--	--
Clearing[b]	--	X	X	X	X	X	--	--
Channel Excavation and Straightening . . .	X	X	X	X	X	X	X	X
Channel Paving[c]	X	X	X	--	X	--	--	--
Streambank Protection	--	--	X	X	X	--	--	--
Culverts and Drop Structures	--	--	--	--	--	--	--	--
Dikes and Floodwalls	--	--	--	--	X	--	--	--

Control Measure	Wetland Drainage	Shifts in Floral and Faunal Communities in Floodplains	Decrease in Low Flow Depths in Channel	Reduction in Quantity and Diversity of Aquatic Habitat	Reduction In Fish Species Diversity and Quantity	Elimination or Hindrance of Access to Stream by Terrestrial Animals	Blocking of Fish Migration	Adverse Effects on Fish Spawning from Reduction in Frequency and Extent of Floodplain Inundation
Snagging[a]	--	--	--	--	--	--	--	--
Clearing[b]	--	--	--	--	--	--	--	--
Channel Excavation and Straightening . . .	X	X	X	X	X	--	--	X
Channel Paving[c]	--	--	X	X	X	X	--	--
Streambank Protection	--	--	--	--	--	X	--	--
Culverts and Drop Structures	--	--	--	--	--	--	X	--
Dikes and Floodwalls	--	X	--	--	--	--	--	X

[a] *Defined as the removal of debris and obstructions from the stream channel.*

[b] *Defined as clearing and debrushing of the channel banks and adjacent areas.*

[c] *Channel paving would be accomplished in conjunction with channel excavation. Only the effects directly attributable to channel paving are indicated here.*

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

There are also certain beneficial environmental, recreational, or aesthetic effects of flood control measures. Utilization of dikes and floodwalls in place of channel modifications minimizes disturbance to the existing stream channel and provides scenic overlooks, trails, and improved stream access. Snagging operations to clear debris and obstructions from the stream channel can improve the aesthetics and recreational potential of the stream.

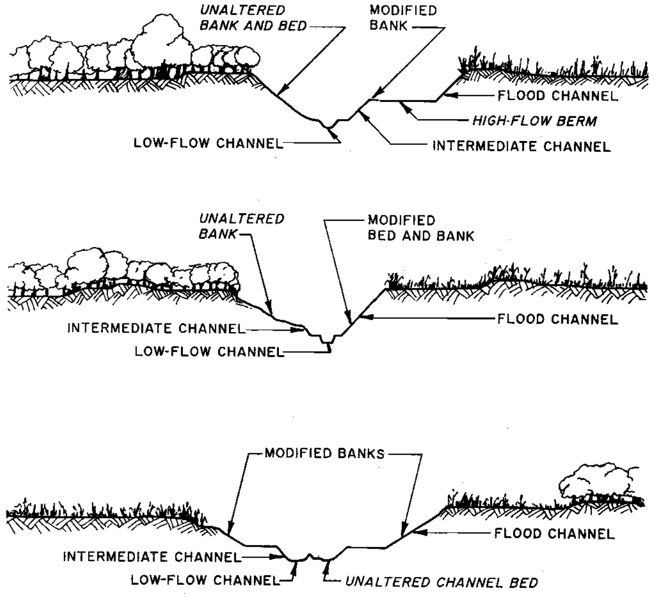
Certain flood control measures may have either adverse or beneficial environmental, recreational, or aesthetic impacts depending on the circumstances and the manner in which they are applied. Examples are the effects of channel excavation and straightening and streambank stabilization on recreational uses and aesthetics, the effects of channel excavation on dissolved oxygen levels, and the effects of streambank protection on aquatic habitat.

Environmental Mitigation Measures

Mitigation measures can be applied for the purpose of maintaining or improving aquatic and terrestrial habitat and for improving stream water quality, recreation, and aesthetics. The instream mitigation measures which are most applicable to the preservation or enhancement of aquatic habitat and the protection of water

Figure 6

TYPICAL CHANNEL MODIFICATION MEASURES UTILIZING MINIMUM STREAM DISTURBANCE TECHNIQUES



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

quality are the provision of instream habitat structures, the design of modifications which limit disturbance to the existing stream channel, and the provision of streambank and streambed protection.

Typical practice in the design of modified flood control channels calls for enlargement, and in some instances deepening, of the existing channel, using a trapezoidal channel section. Such a section eliminates the naturally occurring low-flow channel and results in lower flows being spread across the relatively flat modified channel bottom, rather than being concentrated in a low-flow channel. In streams with a significant existing or potential fish population, such channel modifications can have a profound effect. Therefore, incorporation of a meandering low-flow channel with alternating pools and riffles in the design of a modified channel can serve to mitigate potential adverse impacts on the fishery of the stream. The low-flow channel is the basic mitigative component of a modified channel, to which other necessary habitat mitigation measures may be added.

In instances in which it is necessary to modify a stream channel for flood control purposes, it may be possible to minimize the disturbance to the existing channel by modifying only a single bank, or by preserving the low-flow channel and constructing the modified flood conveyance channel in the streambanks or floodplain. Examples of channel modification measures utilizing minimum stream disturbance techniques are shown on Figure 6. The applicability of such techniques would be limited in certain urban environments where adequate right-of-way is not available.

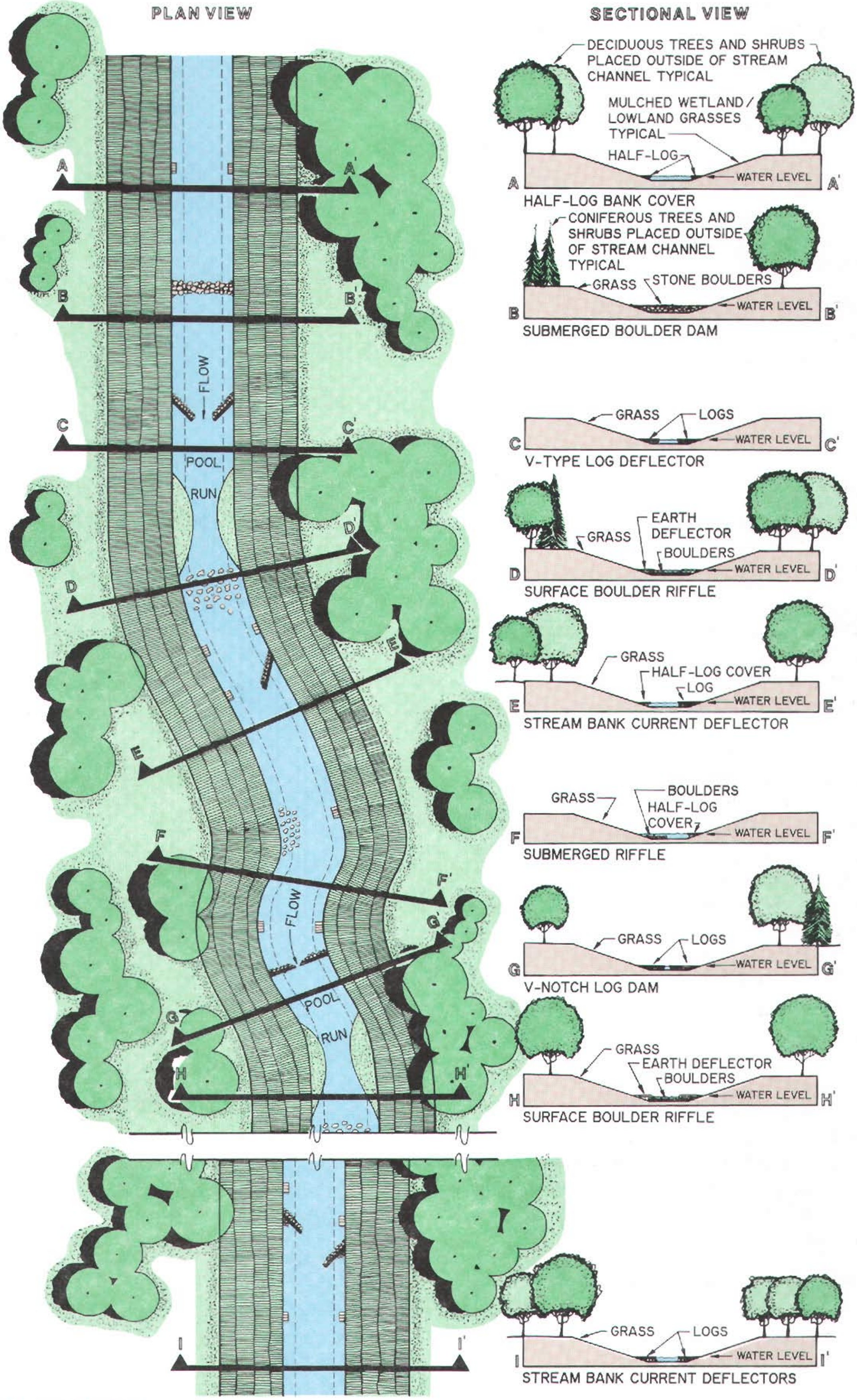
Instream habitat mitigation structures can be used to improve the aquatic habitat in an existing, undisturbed channel or to restore or improve the habitat in a modified channel. Such measures would be constructed in the low-flow channel. The applicable measures fall into the following four basic categories: sills, deflectors, random rocks, and cover. Figure 7 is a schematic of a modified stream channel which illustrates these instream habitat measures. Instream habitat mitigation measures applicable to existing or planned modified channels are given in Table 20.

New culverts can be designed to permit fish migration through the culverts and existing culverts can be retrofitted for the same purpose. The main consideration is the provision of a low-flow channel within the culvert system to concentrate low flows and to permit fish passage. Typical culvert installations which provide for fish migration are shown in Figure 8. Additional considerations in the design of culverts permitting fish migration are the velocities inside the culverts, the culvert length, and culvert entrance and exit conditions. The allowable flow velocity in the culvert for fish migration purposes is a function of fish species and age, water temperature, and culvert length. Methods to achieve the dual objectives of adequate hydraulic capacity for conveying flood flows and the provision of suitable low-flow velocities for fish passage include providing baffles or deflectors in hydraulically oversized culverts, partially burying the invert of a hydraulically oversized culvert, and providing low sills across the downstream channel to create backwater and reduce culvert flow velocities under low-flow conditions.

It is important to note that instream measures for habitat mitigation or water quality improvement measures are not likely to be effective in meeting the established objectives for stream

Figure 7

TYPICAL INSTREAM HABITAT MITIGATION MEASURES TO BE CONSIDERED FOR THE CHANNELIZED STREAMS IN THE LILLY CREEK SUBWATERSHED



Source: SEWRPC.

Table 20

SELECTED INSTREAM HABITAT REHABILITATION MEASURES FOR EXISTING AND PLANNED CHANNEL MODIFICATIONS

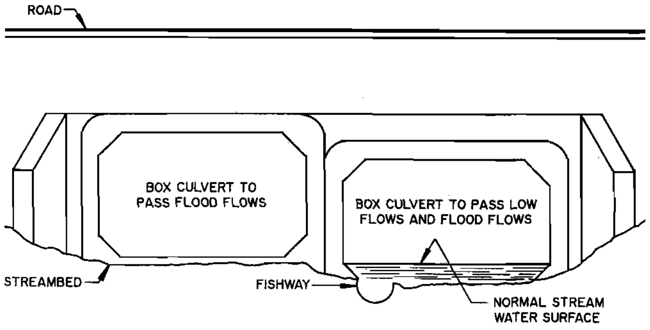
Rehabilitation Measure		Description and Application
Existing Modified Channels	Riffle and pool development	Use various methods below to create riffle-pool sequence. Riffles are sections of streams containing rocks, gravel, or other coarse substrate in which the current is swift enough to remove silt and sand. Riffles should occur at intervals equal to five to seven channel widths. A water depth of six inches is desirable. Riffles help aerate the stream and provide ideal biological habitat. Pools are deeper, slower sections of streams and provide valuable food and resting and refuge areas for fish. Pools ideally should be designed so that the sediments are not completely flushed out during storm events
	Installation of low gabion, rock, or concrete check dams	Low dams provide a pooling effect and accumulate sediment for biological habitat. Dams should be low enough to provide for fish migration
	Installation of gabion or rock wing deflectors	Wing deflectors provide a riffle-pool effect and accumulate sediment. They provide cover for fish and other aquatic life
	Use of scattered rocks	Installation of rocks creates a riffle and provides cover for fish and other aquatic life. They also temporarily trap some sediment
	Vegetation improvement	Plant erosion-resistant native grasses, shrubs, and trees as close as practical to the stream channel to provide cover, food supply, and shade. Provide buffer strip along channel
	Removal of barriers to migrating species	Remove dams, drop structures, chutes, and steep grades which cannot be crossed by migrating fish and other aquatic life. Construct alternative grade control structures
Planned Modified Channels	Channel section and grade design	The low-flow channel cross-section should approach a natural stream condition. The bottom width of the channel and the channel grade can be varied to create a riffle-pool sequence
	Avoidance of straight channels	Constructed channels should be aligned as much as possible with the natural stream curvature
	Vegetation and wetland preservation	Preserve native vegetation and wetlands as much as possible to provide shade trees and shrubs and maintain the water quality, environmental, and aesthetic benefits of wetlands
	Installation of channel bank reservoirs	Various storage measures may be incorporated into the channel bank design to temporarily store runoff, reduce size requirements for downstream channels, and accumulate sediment, thereby providing suitable biological habitat
	Avoidance of barriers to migrating species	Do not construct steep drop structures which cannot be crossed by fish or other aquatic life
	Use of construction erosions controls	Construction erosion control are essential for channel modification projects. Stabilize the exposed surface, control runoff, and prevent sediment delivery to the stream

Source: SEWRPC.

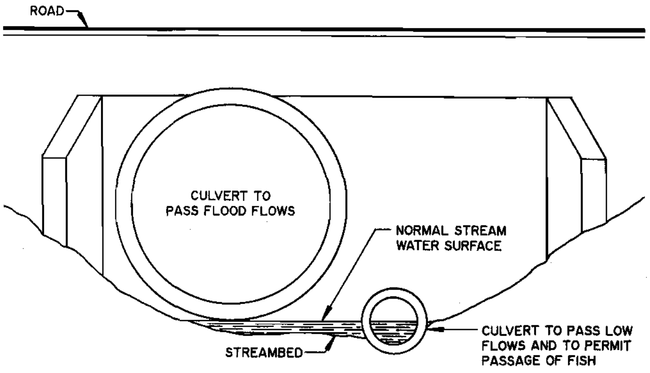
Figure 8

TYPICAL CULVERT INSTALLATIONS
TO PERMIT FISH PASSAGE

BOX CULVERT INSTALLATION



CIRCULAR CULVERT INSTALLATION



Source: SEWRPC.

habitat and quality protection unless they are implemented in conjunction with a comprehensive stormwater management plan which addresses water quantity and quality within the tributary watershed. Environmental mitigation measures must be reviewed in the overall context of associated stormwater management and flood control planning efforts to ensure compatibility with stormwater drainage and flood control objectives.

SUMMARY

This chapter has described the characteristics and functions of various stormwater management and flood control system components. The stormwater management system components which were considered include overland flow, collection, conveyance, storage, and nonpoint source water pollution control. The flood control system components include structural measures such as major channel modifications, bridge and culvert modifications or replacements, diversions, and such containment facilities as earthen dikes and concrete floodwalls. Nonstructural flood control measures include preservation of floodlands for recreational and other open space uses; land use regulation, both inside and outside floodland areas; utility extension policies; federal flood insurance; and structure floodproofing, elevation, and removal. Environmental considerations related to structural flood control measures were also discussed.

With respect to overland flow, the system plan provides general guidelines and a description of practical techniques for minimizing the rate and volume of runoff. The plan assumes that these general guidelines will be followed to the extent practicable as community development and redevelopment proceeds and the siting of buildings and the grading and improvement of surrounding sites takes place. Specific measures for overland flow, however, must be designed on a site-specific basis as urban development or redevelopment takes place.

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

With respect to stormwater conveyance facilities, the system plan contains recommendations concerning the general horizontal and vertical alignment, shape, and type of open channel conveyance facilities; the general locations and sizes of culverts; and the general alignment, depth, size, slope, and type of storm sewer facilities. The system plan also indicates the general locations of manholes and junction chambers.

The two remaining system components, stormwater storage and nonpoint source water pollution control, were presented in this chapter as additional components which may be required within stormwater management systems to meet overall system development and performance objectives. The system plan contains recommendations concerning the general location, area, volume, and elevation-discharge relationships of storage facilities. Additional details of such storage facilities must be addressed on a site-specific basis in the detailed design phase preceding construction. Criteria for such design are provided in the plan. Urban nonpoint source water pollution control measures available for use in both existing and newly developing urban settings were identified, along with an estimate of the reduction in pollutant loadings that can be achieved by each measure. To the extent practicable, the results of the Menomonee River Priority Watershed Program have been refined and integrated in the recommended plan for the Lilly Creek subwatershed. That system plan contains recommendations concerning the general location and extent of recommended nonpoint source control measures. Component details must be addressed on a site-specific basis during the facility design stage.

The selection of street cross-sections, including appurtenant drainage details, is a decision which must be based primarily on the existing or proposed land use and density of development within an area of the Village. In areas where the type and density of land use do not clearly dictate the use of either an urban or a rural street cross section, an important consideration in the selection of the cross section is the preferences of local residents and officials. Within new residential and commercial areas of the Village of Menomonee Falls, urban street cross-sections with gutters, inlets, and storm sewers have generally been constructed. Rural street cross-sections with roadside ditches were, however, utilized in many of the Village's established low density residential areas. In the preparation of the stormwater management plan, consideration was given to the use of both urban and rural street cross sections.

The system plan contains recommendations concerning the general horizontal and vertical alignment, sizes, and shapes of various structural flood control facilities. The potential effects of structural flood control facilities on water quality, habitat, wetlands, groundwater resources, aesthetics, and recreation are considered in the systems plan. Instream habitat mitigation measures are specified where appropriate.

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Chapter IV

STORMWATER MANAGEMENT OBJECTIVES, STANDARDS, AND DESIGN CRITERIA

INTRODUCTION

Planning may be defined as a rational process for formulating and meeting objectives. Consequently, the formulation of objectives is an essential task which must be undertaken before plans can be prepared. Accordingly, this chapter sets forth a set of stormwater management objectives and supporting standards related to land use development, stormwater drainage, water quality, and flood control for use in the design and evaluation of alternative stormwater management and flood control system plans for the Lilly Creek subwatershed and in the selection of a recommended plan from among those alternatives.

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

OBJECTIVES AND STANDARDS

The following stormwater management objectives were formulated both to guide the design, test, and evaluation of alternative stormwater management and flood control plans for the Lilly Creek subwatershed and also the selection of a recommended plan from among the alternatives considered:

1. The development of a stormwater management system which reduces the exposure of people to drainage-related inconvenience and to health and safety hazards and which reduces the exposure of real and personal property to damage through inadequate stormwater drainage and inundation.
2. The development of an integrated stormwater management and flood control system which will effectively serve existing and planned land uses and promote implementation of adopted local and regional land use plans.
3. The development of an integrated system of stormwater management and flood control facilities and floodland management programs which will effectively reduce flood damage under the anticipated runoff loadings generated by the existing and proposed land uses.
4. The development of a stormwater management and flood control system which will abate nonpoint source water pollution and help achieve the recommended water use objectives and supporting water quality standards for surface water bodies.
5. The development of a stormwater management system which will be flexible and readily adaptable to changing needs.
6. The development of a stormwater management and flood control system which maintain or enhance existing terrestrial, riparian, and aquatic biological communities, including fish and wildlife.
7. The development of a stormwater management and flood control system which will efficiently and effectively meet all of the other stated objectives at the lowest practicable cost.

Complementing each of the foregoing objectives is a set of quantifiable standards which can be used to evaluate the relative or absolute ability of alternative stormwater management and flood control plan designs to meet each objective. These standards are set forth in Table 21. The planning standards fall into two groups, com-

Table 21

OBJECTIVES AND STANDARDS FOR STORMWATER MANAGEMENT AND FLOOD CONTROL PLANNING IN THE LILLY CREEK SUBWATERSHED

OBJECTIVE NO. 1

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

STANDARDS

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

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

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

4. Flow of stormwater along and across the full pavement width of collector, land access, and arterial streets shall be acceptable during storm events exceeding a 10-year recurrence interval, the streets being intended to constitute integral parts of the major stormwater drainage system.

OBJECTIVE NO. 2

The development of an integrated stormwater management and flood control system which will effectively serve existing and planned land uses and will promote implementation of the adopted land use plan.

STANDARDS

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

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

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

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

5. Stormwater management and flood control systems should utilize the existing floodwater storage capacity of wetlands and open spaces to the extent practicable.

6. No channel modifications, dikes, or floodwalls shall be constructed which alter the limits of the 100-year recurrence interval floodplain for the sole purpose of creating additional developable land.

OBJECTIVE NO. 3

The development of an integrated system of stormwater management and flood control facilities and floodland management programs which will effectively reduce flood damage under the anticipated runoff loadings generated by the existing and proposed land uses.

Table 21 (continued)

STANDARDS

1. Flood control facilities shall be designed to alleviate flood damages during floods up to an including the 100-year recurrence interval event under planned land use, drainage, and channel conditions.

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

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

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

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

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

3. All new and replacement bridges and culverts over waterways, including pedestrian and other minor bridges, in addition to meeting the applicable requirements of paragraph No. 1 above, shall be designed so as to accommodate the 100-year recurrence interval flood event without raising the peak stage, either upstream or downstream, 0.01 foot or more above the peak stage for the 100-year recurrence interval flood, as established in this stormwater management and flood control plan.[b] Larger permissible flood stage increases may be acceptable for channel reaches having topographic or land use conditions which could accommodate the increased stage without creating additional flood damage potential upstream or downstream of the proposed structure.

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

5. New or replacement bridges and culverts over waterways, including pedestrian and other minor bridges, so located with respect to the stream system that the accumulation of floating ice or other debris may cause significant backwater effects with attendant danger to life, public health, or safety, or attendant serious damage to homes, industrial and commercial buildings, and important public utilities, shall be designed so as to pass the 100-year recurrence interval flood with at least 2.0 feet of freeboard between the peak stage and the low concrete or steel in the bridge span.

6. Standards 2, 3, and 5 shall also be used as the criteria for assessment of the adequacy of the hydraulic capacity and structural safety of existing bridges or culverts over waterways and thereby serve, within the context of the adopted stormwater management and flood control system plan, as the basis for crossing modification or replacement recommendations designed to alleviate flooding and other problems.

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

8. Channel modifications, dikes, and floodwalls should be restricted to the minimum number and extent absolutely necessary for the protection of existing and proposed land use development, consistent with the adopted land use plan for the Village. The upstream and downstream effect of such structural works on flood discharges and stages shall be determined. Channel modifications, dikes, or floodwalls shall not increase the height of the 100-year recurrence interval flood by 0.01 foot or more in any unprotected upstream or downstream stream reaches.[a,b] Increases in flood stages which are equal to or greater than 0.01 foot resulting from any channel, dike, or floodwall construction shall be contained within the upstream or downstream extent of the channel, dike, or floodwall improvement, except where topographic or land use conditions could accommodate the increased stage without creating additional flood damage potential.

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

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

   b. The 500-year recurrence interval flood profile.

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

Table 21 (continued)

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

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

12. Floodlands that are unoccupied by, and not committed to, urban development should be retained in essentially natural, open uses.

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

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

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

OBJECTIVE NO. 4

The development of a stormwater management and flood control system which will abate nonpoint source water pollution and help achieve the recommended water use objectives and supporting water quality standards for surface water bodies.

STANDARDS

1. Stormwater management and flood control facilities should not impede the achievement of existing water use objectives and supporting water quality standards for lakes, streams, and wetlands; nor degrade existing habitat conditions for fish and aquatic life. The applicable water use objectives for the lakes and streams concerned are shown on Map 7, and the water quality standards supporting these use objectives are presented in Table 22.[c] The recommended objectives and standards are a refinement of those set forth in the adopted areawide water quality management plan as documented in SEWRPC Planning Report No. 30, A Regional Water Quality Management Plan for Southeastern Wisconsin: 2000, Volume Three, Recommended Plan, June 1979.

OBJECTIVE NO. 5

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

STANDARDS

1. The 100-year recurrence interval storm event should be used to design and size special structures, such as roadway underpasses, requiring pumping stations.

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

OBJECTIVE NO. 6

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

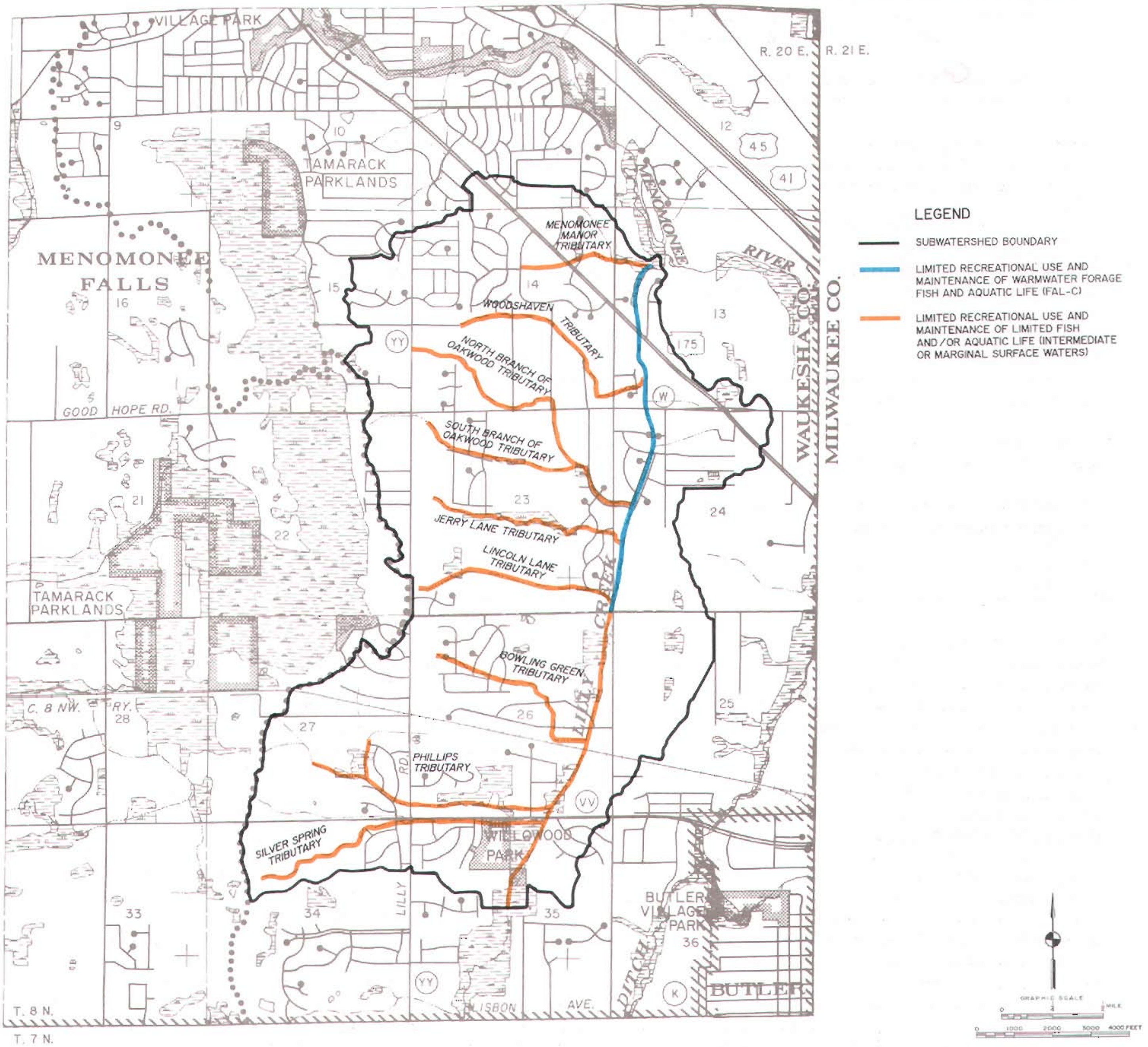
STANDARDS

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

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

Map 7

RECOMMENDED WATER USE OBJECTIVES FOR STREAMS IN THE LILLY CREEK SUBWATERSHED: 2010



Source: Wisconsin Department of Natural Resources and SEWRPC.

Table 21 (continued)

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

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

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

OBJECTIVE NO. 7

The development of a stormwater management and flood control system which will maintain or enhance existing terrestrial, riparian, and aquatic biological communities, including fish and wildlife.

STANDARDS

1. Stormwater management systems shall be designed to minimize disruption to primary and secondary environmental corridors, including the incorporated woodlands, wetlands, and wildlife habitat areas.

2. Stormwater management and flood control facilities should be designed to control sedimentation in receiving streams and to prevent the loss of fish and aquatic life habitat through streambank erosion and streambed scour.

3. To the extent practicable, drainage and flood control facilities should be designed to avoid enclosure of tributary streams identified as having significant and valuable biological and recreational uses.

[a] *Downstream reaches include the Menomonee River downstream from the mouth of Lilly Creek.*

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

[c] *The existing Wisconsin Department of Natural Resources (DNR) water quality standards for surface waters in the Lilly Creek subwatershed are given in Table 23. Those standards are set forth in Chapters NR 102, NR 104, NR 106, and NR 207 of the Wisconsin Administrative Code and are regulatory standards which are enforced by the DNR. The DNR regulatory standards presented in Table 23 differ somewhat from the planning standards utilized by the Regional Planning Commission and presented in Table 22. The Wisconsin Department of Natural Resources, being a regulatory agency, utilizes water quality standards as a basis for enforcement actions and compliance monitoring. This requires that the standards have a rigid basis in research findings and in field experience. The Commission, by contrast, must forecast conditions far into the future, documenting the assumptions used to analyze conditions and problems which may not currently exist anywhere, much less in or near southeastern Wisconsin. Consequently, sometimes more controversial standards based in emerging technology must be applied. This results from the Commission's use of the water quality standards as the basis for comparatively evaluating the relative merits of alternative plans.*

Source: SEWRPC.

Table 22

RECOMMENDED WATER QUALITY STANDARDS FOR SURFACE WATERS

Water Quality Indicator	Coldwater Fish and Aquatic Life, Recreational Use, and Minimum Standards[a]	Warmwater Fish and Aquatic Life, Recreational Use, and Minimum Standards[a]	Warmwater Forage Fish and Aquatic Life, Limited Recreational Use, and Minimum Standards[a]	Limited Fish and Aquatic Life, Limited Recreational Use, and Minimum Standards[a]
Maximum Temperature (°F)	--[b,c,d]	89[b,d]	89[b,d]	89[b,d]
pH Range (standard units)	6.0-9.0[e]	6.0-9.0[e]	6.0-9.0[e]	6.0-9.0[e]
Minimum Dissolved Oxygen (mg/l)[d]				
30-Day Mean	6.5	5.5	5.5	4.5
7-Day Mean	9.5[f]	6.0[g]	6.0[g]	5.0[h]
1-Day Mean	5.0-8.0[i]	4.0-5.0[j]	4.0-5.0[j]	3.0-4.0[k]
Absolute	3.0	2.5	2.5	1.5
Maximum Fecal Coliform (counts per 100 ml)	200-400[l]	200-400[l]	1,000-2,000-10,000[m]	1,000-2,000-10,000[m]
Maximum Total Residual Chlorine (mg/l)				
4-Day Mean	0.011	0.011	0.011	0.011
1-Hour Mean	0.019	0.019	0.019	0.019
Maximum Un-ionized Ammonia Nitrogen (mg/l)	--[n]	--[n]	--[n]	--[n]
Maximum Total Phosphorus (mg/l)				
Streams	0.1	0.1	--[n]	--[n]
Inland Lakes[o]	0.02	0.02	--	--

[a] *All waters shall meet the following minimum standards at all times and under all flow conditions: Substances that will cause objectionable deposits on the shore or in the bed of a body of water shall not be present in such amounts as to interfere with public rights in the waters of the State. Floating or submerged debris, oil, scum, or other material shall not be present in such amounts as to interfere with public rights in the waters of the State. Materials producing color, odor, taste, or unsightliness shall not be present in amounts found to be of public health significance. Unauthorized concentrations of substances are not permitted that alone or in combination with other substances present are chronically or acutely harmful or toxic to humans or to fish and aquatic life.*

[b] *There shall be no temperature changes that may adversely affect aquatic life. Natural daily and seasonal temperature fluctuations shall be maintained. The maximum temperature rise at the edge of a mixing zone above the existing natural temperature shall not exceed 5°F for streams and 3°F for lakes.*

[c] *There shall be no significant artificial increases in temperature where natural trout reproduction is to be protected. The maximum temperature shall not exceed 77°F.*

[d] *Dissolved oxygen and temperature standards apply to the entire water column within streams and to the epilimnion of stratified lakes and to the unstratified lakes; the dissolved oxygen standard does not apply to the hypolimnion of stratified inland lakes. Trends in the period of anaerobic conditions in the hypolimnion of stratified inland lakes should be considered important to the maintenance of water quality, however.*

[e] *The pH shall be within the range of 6.0 to 9.0 standard units, with no change greater than 0.5 unit outside the estimated natural seasonal maximum and minimum.*

[f] *A minimum dissolved oxygen standard of 9.5 milligrams per liter (mg/l) for a seven-day mean applies only between September 1 and April 30 for the support of embryonic and larval stages of coldwater species.*

[g] *A minimum dissolved oxygen standard of 6.0 mg/l for a seven-day mean applies only between March 15 and July 31 for the support of embryonic, larval, and early juvenile stages of warmwater species.*

[h] *A minimum dissolved oxygen standard of 5.0 mg/l for a seven-day mean applies only between March 15 and July 31 for the support of embryonic, larval, and early juvenile stages of limited species.*

[i] *A minimum dissolved oxygen standard of 8.0 mg/l for a one-day mean applies only between September 1 and April 30 for the support of embryonic and larval stages of coldwater species. For the remainder of the year, a minimum dissolved oxygen standard of 5.0 mg/l for a one-day mean applies.*

Table 22 (continued)

[j] A minimum dissolved oxygen standard of 5.0 mg/l for a one-day mean applies only between March 15 and July 31 for the support of embryonic, larval, and early juvenile stages of warmwater species. For the remainder of the year, a minimum dissolved oxygen standard of 4.0 mg/l for a one-day mean applies.

[k] A minimum dissolved oxygen standard of 4.0 mg/l for a one-day mean applies only between March 15 and July 31 for the support of embryonic, larval, and early juvenile stages of limited species. For the remainder of the year, a minimum dissolved oxygen standard of 3.0 mg/l for a one-day mean applies.

[l] Shall not exceed a monthly geometric mean of 200 per 100 milliliters (ml) based on not fewer than five samples per month, nor a monthly geometric mean of 400 per 100 ml in more than 10 percent of all samples during any month.

[m] A monthly geometric mean fecal coliform level of 1,000 most probable number per 100 milliliters (MPN/100 ml) shall not be exceeded more than 5 percent of the time, or about once every two years. A fecal coliform level of 2,000 MPN/100 ml shall not be exceeded more than 10 percent of the time. A fecal coliform level of 10,000 MPN/100 ml shall not be exceeded more than 2 percent of the time, or about one week per year.

[n] To protect fish and aquatic life, the following standards shall apply for un-ionized ammonia nitrogen (NH_3-N):

1. *The one-hour mean concentration of un-ionized ammonia nitrogen shall not exceed, more often than once every three years on the average, the numerical value given by 0.427/FT/FPH/2, where:*

$$
\begin{array}{lll}
FT & = 10^{0.03(20-TC)} & ;\ TC \le T \le 30 \\
 & \ 10^{0.03(20-T)} & ;\ 0 \le T \le TC \\
FPH & = 1 & ;\ 8 \le pH \le 9 \\
 & \ \dfrac{1+10^{7.4-pH}}{1.25} & ;\ 6.5 \le pH \le 8 \\
TC & = 20^\circ C & ;\ \text{Coldwater fish and aquatic life} \\
 & = 25^\circ C & ;\ \text{Warmwater and limited fish and aquatic life.} \\
T & = \text{Temperature of water body, in degrees C.} & \\
pH & = \text{pH of water body, in standard units.} &
\end{array}
$$

2. *The four-day mean concentration of un-ionized ammonia nitrogen shall not exceed, more often than once every three years on the average, the average numerical value given by 0.658/FT/FPH/R, where FT and FPH are as above, and:*

$$
\begin{array}{lll}
R & = 16 & ;\ 7.7 \le pH \le 9 \\
 & = 24 \left| \dfrac{10^{7.7-pH}}{1+10^{7.4-pH}} \right| & ;\ 6.5 \le pH \le 7.7 \\
TC & = 15^\circ C & ;\ \text{Coldwater fish and aquatic life.} \\
 & = 20^\circ C & ;\ \text{Warmwater and limited fish and aquatic life.}
\end{array}
$$

The extremes for temperature (0°, 30°C) and pH (6.5 standard units, 9.0 standard units) are absolute, and these standards cannot be extrapolated beyond these limits. Because the formulas are nonlinear with respect to pH and temperature, the standards used for a particular water body should be based on separate calculations reflective of the fluctuations of pH and temperature during a study period. It is not appropriate to simply apply the formulas to average pH and temperature conditions over a study period.

[o] The values presented for inland lakes are the critical total phosphorus concentrations which apply only during spring, when maximum mixing is underway.

Source: SEWRPC.

Table 23

EXISTING DEPARTMENT OF NATURAL RESOURCES WATER USE OBJECTIVES AND WATER QUALITY STANDARDS FOR SURFACE WATERS: 1988

Water Quality Parameters	Individual Water Use Objectives Applicable to Surface Waters in the Lilly Creek Subwatershed				
	Warmwater Fish and Aquatic Life (FAL-B or C)	Coldwater Fish and Aquatic Life (FAL-A)	Intermediate Fish and Aquatic Life[a]	Marginal Aquatic Life[b,c]	Recreational Use
Maximum Temperature (°F)	89[d,e]	- -[d,e,f]	89[d,e]	89[d]	- -
pH Range (standard units)	6.0-9.0[g]	6.0-9.0[g]	6.0-9.0[g]	6.0-9.0[g]	- -
Minimum Dissolved Oxygen (mg/l)	5.0[e]	6.0[e]	3.0[e]	2.0	- -
Maximum Fecal Coliform (counts per 100 ml)	- -	- -	- -	200-400[h]	200-400[h]
Maximum Total Residual Chlorine (mg/l)	0.01	0.002	0.5	0.5	- -
Maximum Un-ionized Ammonia Nitrogen (mg/l)	0.04	0.02	- -	- -	- -
Total Ammonia Nitrogen (mg/l) . . .	- -	- -	3/6[i]	- -	- -
Maximum Total Dissolved Solids (mg/l)	- -	- -	- -	- -	- -
Other	- -[j]	- -[j,k]	- -[j]	- -	- -

[a] *As set forth in NR 104.02(3)(a) and NR 104.06(2)(b) of the Wisconsin Administrative Code.*

[b] *Includes all effluent channels used predominantly for waste carriage and assimilation, wetlands, and diffuse surface waters and includes selected continuous and noncontinuous streams as specified by the DNR on the basis of field surveys and identified as "marginal surface waters." (See Wisconsin Administrative Code, Chapter NR 104.02(3)(b) and NR 104.06(2)(b).)*

[c] *May include explicitly designated agricultural drainage ditches.*

[d] *There shall be no temperature changes that may adversely affect aquatic life. Natural daily and seasonal temperature fluctuations shall be maintained. The maximum temperature rise at the edge of the mixing zone above the existing natural temperature shall not exceed 5°F for streams and 3°F for lakes.*

[e] *Dissolved oxygen and temperature standards apply to streams and the epilimnion of stratified lakes and to the unstratified lakes; the dissolved oxygen standard does not apply to the hypolimnion of stratified inland lakes. Trends in the period of anaerobic conditions in the hypolimnion of deep inland lakes should be considered important to the maintenance of water quality, however.*

[f] *There shall be no significant artificial increases in temperature where natural trout or stocked salmon reproduction is to be protected. Dissolved oxygen shall not be lowered to less than 7.0 milligrams per liter (mg/l) during the trout spawning season. The dissolved oxygen in the Great Lakes tributaries used by salmonids for spawning runs shall not be lowered below natural background levels during the period of habitation.*

[g] *The pH shall be within the range of 6.0 to 9.0 standard units, with no change greater than 0.5 unit outside the estimated natural seasonal maximum and minimum.*

[h] *Shall not exceed a monthly geometric mean of 200 per 100 milliliters (ml) based on not fewer than five samples per month, nor a monthly geometric mean of 400 counts per 100 ml in more than 10 percent of all samples during any month.*

[i] *Ammonia nitrogen (as N) at all points in the receiving water shall not be greater than 3 mg/l during warm temperature conditions, nor greater than 6 mg/l during cold temperatures, to minimize the zone of toxicity and to reduce dissolved oxygen depletion caused by oxidation of the ammonia.*

[j] *Unauthorized concentrations of substances are not permitted that alone or in combination with other materials present are toxic to fish or other aquatic life. The determination of the toxicity of a substance shall be based upon the available scientific data base. References to be used in determining the toxicity of a substance shall include, but not be limited to: Quality Criteria for Water, EPA-440/9-76-003, U. S. Environmental Protection Agency, Washington, D. C., 1976; Water Quality Criteria 1972, EPA-R3-73-003, National Academy of Sciences, National Academy of Engineering, U. S. Government Printing Office, Washington, D. C., 1974; and the Federal Register, "Environmental Protection Agency, Water Quality Criteria Documents; Availability," November 28, 1980. Questions concerning the permissible levels, or changes in the same, of a substance, or combination of substances, or undefined toxicity to fish and other biota shall be resolved in accordance with the methods specified in Water Quality Criteria 1972, and Standard Methods for the Examination of Water and Wastewater, 14th Edition, American Public Health Association, New York, 1975, or other methods approved by the Department of Natural Resources.*

[k] *Streams classified as trout waters by the DNR (Wisconsin Trout Streams, publication 213-72) shall not be altered from natural background conditions by effluents that influence the stream environment to such an extent that trout populations are adversely affected.*

Source: Wisconsin Department of Natural Resources and SEWRPC.

parative and absolute. The comparative standards, by their very nature, can be applied only through a comparison of alternative plan proposals. The absolute standards can be applied individually to each alternative plan proposal since they are expressed in terms of maximum, minimum, or desirable values.

OVERRIDING CONSIDERATIONS

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

ANALYTICAL PROCEDURES AND ENGINEERING DESIGN CRITERIA

Introduction

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

Analytical Procedures

Rainfall Intensity-Duration-Frequency Data: Fundamental data for stormwater management planning and design are the rainfall intensity-duration-frequency relationships representative of the area. Such relationships facilitate determination of the average rainfall intensity, normally expressed in inches per hour, which may be expected to be reached or exceeded for a particular duration at a given recurrence interval. Under its comprehensive water resources planning program, the Southeastern Wisconsin Regional Planning Commission has developed a set of rainfall intensity-duration-frequency relationships using both a graphic procedure and a mathematical curve fitting method. The data from the 84-year rainfall record from 1903 through 1986 collected by the National Weather Service at the National Weather Service station in Milwaukee are summarized in tabular form in Table 24 and in graphic form in Figure 9. The intensity-duration-frequency equations resulting from the analysis of the Milwaukee data are presented in Table 25. Analyses conducted by the staff of the Commission indicate that these data are valid for use not only within the Milwaukee area, but anywhere in Southeastern Wisconsin. The curves in Figure 10, which relate total rainfall to duration and frequency, were developed from the curves of Figure 9.

Design Rainfall Frequency: To ensure that the stormwater management system is able to control the stormwater runoff effectively in a cost-effective manner, storm events of specified recurrence intervals must be selected as a basis

Table 24

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

Recurrence Interval (years)	Duration and Intensity[b]						
	5 Minutes	10 Minutes	15 Minutes	30 Minutes	1 Hour	2 Hours	24 Hours
2	4.30	3.43	2.85	1.90	1.14	0.67	0.099
5	5.49	4.46	3.76	2.55	1.55	0.91	0.134
10	6.26	5.14	4.35	2.99	1.84	1.07	0.156
25	7.26	5.99	5.10	3.53	2.19	1.27	0.186
50	7.98	6.62	5.65	3.93	2.44	1.41	0.208
100	8.77	7.28	6.23	4.34	2.70	1.56	0.229

[a] *These data are based on a statistical analysis of Milwaukee rainfall data for the 84-year period 1903 through 1986.*

[b] *Intensity expressed in inches per hour.*

Source: SEWRPC.

for the design and evaluation of both the minor and major drainage systems. The selection of these design storm events should be dictated by careful consideration of the frequency of inundation which can be accepted versus the cost of protection. This involves value judgments which should be made by the responsible local officials involved and applied consistently in both the public and private sectors.

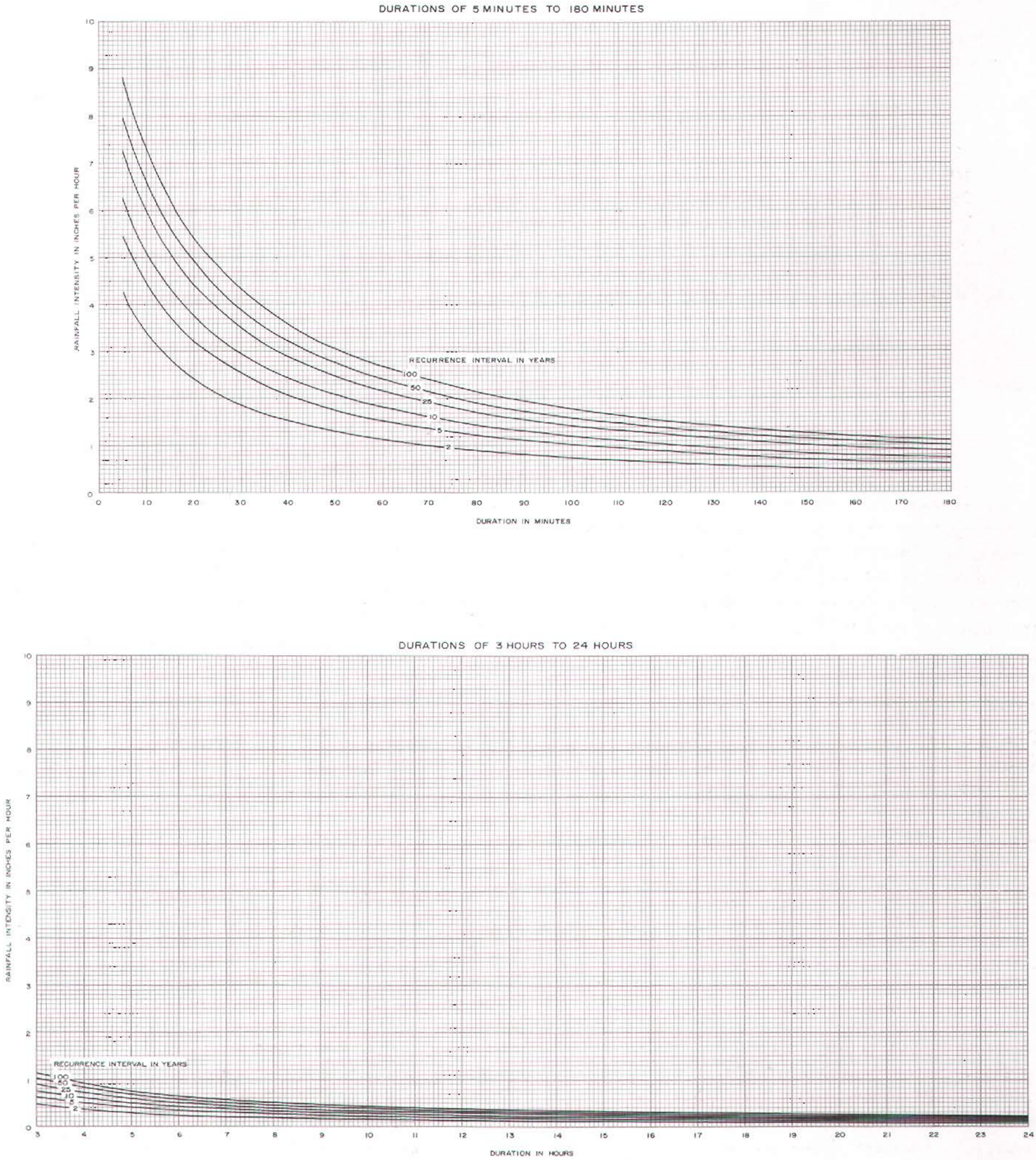
The average frequency of rainfall used for design purposes determines the degree of protection afforded by the stormwater management system. This protection should be consistent with the damage to be prevented. In practice, however, the calculation of benefit-cost ratios is generally not deemed practical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

In this respect it should be noted that the cost of storm sewers and other drainage facilities is not directly proportional to either the design storm frequency or the flow rates. A 10-year recurrence interval storm produces approximately 16 percent greater rainfall intensities and 26 percent greater runoff intensities than a five-year recurrence interval storm. This higher runoff rate requires sewer pipe diameters to be on the order of 10 percent larger. However, for practical reasons, the conduits used in drainage systems are limited to commercially available pipe sizes which, in the most frequently used range of 15- to 66-inch diameter, have incremental diameter increases of 10 to 20 percent, corresponding incremental capacity increases of 27 to 58 percent, and corresponding average in-place cost increases of 15 to 23 percent. The incremental cost increases on a systemwide basis may be expected to be on the order of about 15 percent, because only portions of any given system will require modified sizes.

Another consideration in evaluating alternative design recurrence intervals for drainage facilities is the risk of exceeding capacity. Table 26 indicates that a five-year recurrence interval event, which may be expected to occur on the average of 20 times in 100 years, has a 50 percent chance of being exceeded in about 3.5 years, a period which may be unacceptable from the point of view of public relations. In contrast, a 10-year recurrence interval event, which is expected to occur on the average of 10 times in 100 years, has a 50 percent chance of being exceeded in about seven years. A 100-year recurrence interval event, which is expected to occur on the average of one time in 100 years, has a 50 percent chance of being exceeded in about 69 years.

Figure 9

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



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

Source: SEWRPC.

Table 25

POINT RAINFALL INTENSITY-DURATION-FREQUENCY EQUATIONS FOR THE LILLY CREEK SUBWATERSHED AND THE REGION[a]

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

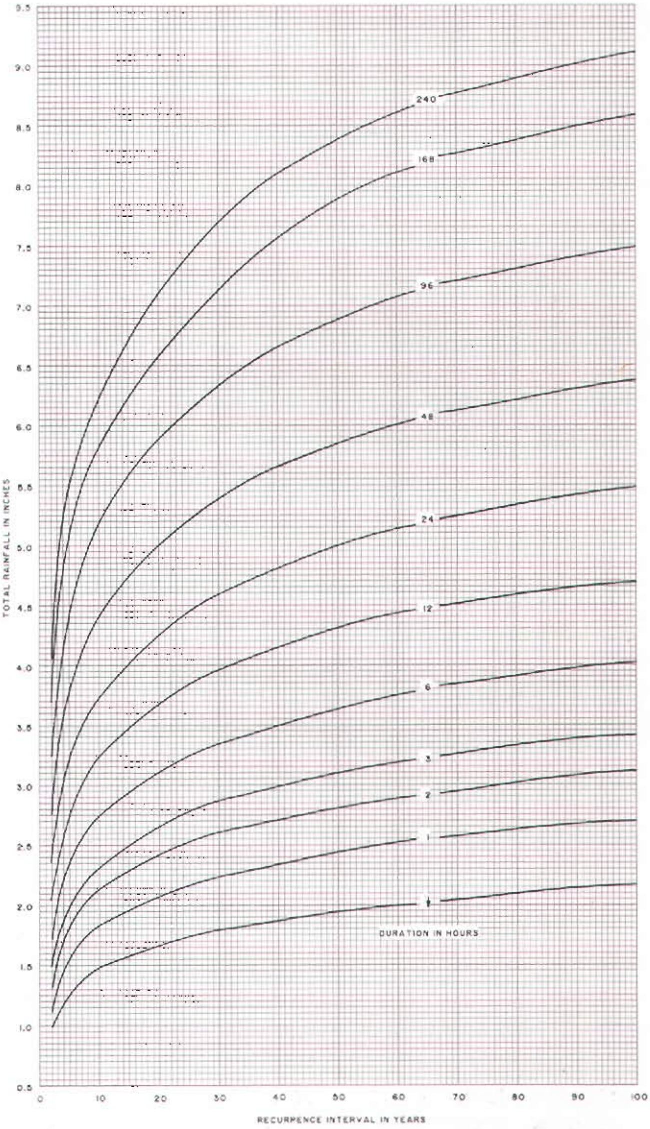
[a] *The equations are based on Milwaukee rainfall data for the 84-year period 1903 to 1986. These equations are applicable, within an accuracy of ± 10 percent, to the entire Southeastern Wisconsin Region.*

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

Source: SEWRPC.

Figure 10

POINT RAINFALL DEPTH-DURATION-FREQUENCY RELATIONSHIPS FOR THE LILLY CREEK SUBWATERSHED AND THE REGION



Source: SEWRPC.

Exceeding the capacity of the minor urban stormwater management system does not cause immediate catastrophe. On the contrary, it only means that the unaccommodated portion of the stormwater flow will begin to cause inconvenience and/or disruption of activities as it courses through the major system. In this respect, the minor system differs substantially from the major system, where exceedance of capacity could cause structure flooding and attendant major property damage.

Current Village practices related to stormwater drainage system design call for use of a five-year recurrence interval design storm in areas of existing development, a 10-year storm in areas of new development, and a 10- to 25-year storm for new trunk sewers. The selection of a 10- or 25-year design storm for trunk sewers depends on hydraulic conditions and the potential hazard to adjacent properties. Interim design guidelines enforced by the Village, pending adoption of this system plan, require stormwater detention basins to be provided for the four categories of new development listed in the section describing the existing stormwater management and flood control system in Chapter II of this report. The interim guidelines require the provision of onsite detention storage to limit the peak flow due to post-development runoff from a 100-year recurrence interval rainfall event to the peak flow from a five-year rain with the site in its undeveloped condition.

Table 26

THEORETICAL RISK OF DESIGN STORM OCCURRENCE

Average Recurrence Interval (Tr) Years	Probability that Interval between Events Will Not Be Exceeded in Period of N Years						
	5 Percent	10 Percent	25 Percent	50 Percent	75 Percent	90 Percent	95 Percent
100	300 years	230 years	139 years	69 years	29 years	11 years	5 years
10	30	23	14	7	3	1.1	0.5
5	15	12	7	3	1.4	0.6	0.3
2	6	5	3	1.4	0.6	0.2	0.1
1	3	2	1.4	0.7	0.3	0.1	0.05
0.5	1.5	1.2	0.7	0.3	0.1	0.05	0.03
0.25	0.7	0.6	0.3	0.2	0.1	0.03	0.01

NOTE: Based on:

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

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

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

Where:

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

Source: SEWRPC.

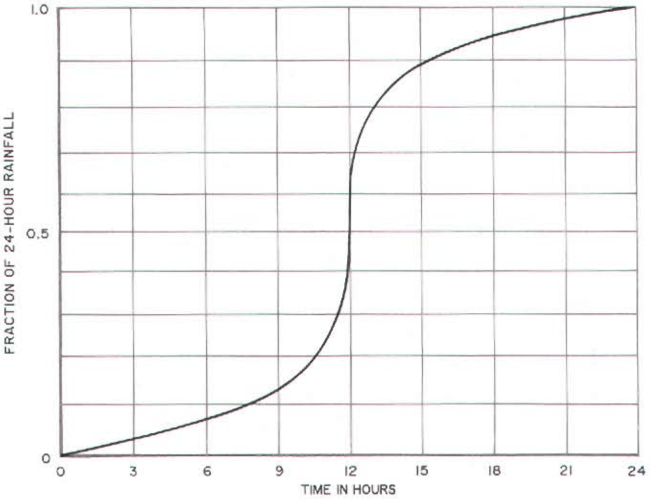
Based upon consideration of the costs and risks entailed and upon current Village practices, a 10-year recurrence interval storm was selected for use in the evaluation of existing elements, and in the design of new elements, of the minor stormwater management system for the Lilly Creek subwatershed. Although current Village practice calls for use of a five-year recurrence interval storm in areas of existing development, use of a 10-year recurrence interval storm would provide a consistent level of protection throughout the subwatershed and to provide a consistent basis for evaluating the ability of the existing system to accommodate runoff from upstream areas of new development where a 10-year storm is the design standard. The plan does, however, consider the possibility of providing upstream detention storage to avoid replacement of existing conveyance components which would have inadequate capacity for a 10-year storm under future conditions.

A 100-year recurrence interval storm was selected for use in determining areas of potential inundation along the stormwater drainage system in order to size elements of the major stormwater management system and to design flood control facilities. This recurrence interval is used by the Regional Planning Commission in its flood control planning efforts, and by federal and state agencies for floodland regulation. The 100-year recurrence interval event generally, with only certain unusual exceptions, approximates, in terms of the amount of land area inundated, the largest known flood levels that have actually occurred in the Region since its settlement by Europeans. Therefore, use of a 100-year recurrence interval event provides a conservatively safe level of protection against property damage and hazards to human health and safety from surcharge of the major stormwater management system.

The minor and major system design standards adopted for this plan are consistent with the Village standard of designing trunk storm sewers for storms with recurrence intervals from 10 to 25 years. Where hydraulic conditions are such that the major system, consisting of a storm sewer or swale which conveys the runoff from a 10-year recurrence interval storm and the entire street cross-section which conveys the runoff in excess of that from a 10-year storm, have inadequate combined capacity to convey the peak runoff from a 100-year storm without flooding adjacent buildings, the standards adopted for this plan would require that the

Figure 11

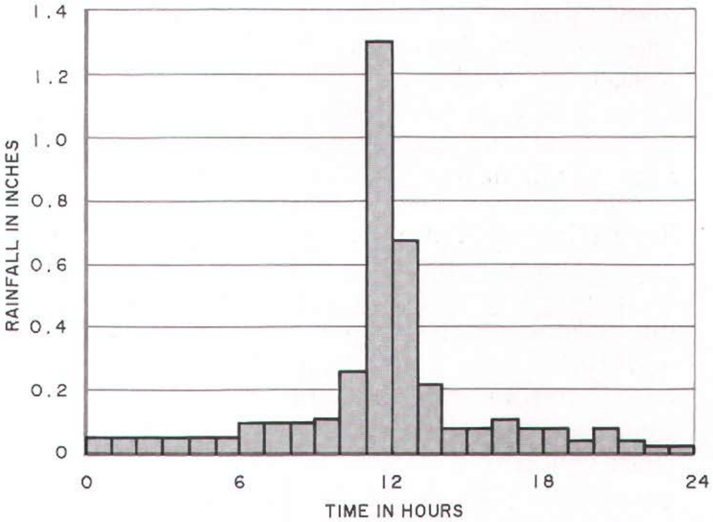
SCS TYPE II RAINFALL DISTRIBUTION



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

Figure 12

DESIGN HYETOGRAPH FOR A 10-YEAR RECURRENCE INTERVAL 24-HOUR STORM WITH HOURLY RAINFALL INCREMENTS



Source: SEWRPC.

design storm sewer or swale capacity be increased above that for a 10-year storm in order to obtain 100-year storm capacity for the major system. Therefore, the design capacity of storm sewers is flexible, subject to satisfaction of the 100-year storm capacity criterion for the major system. The most extreme application of that standard would occur in a situation where the components of the major system other than the storm sewers have zero capacity, such as at an existing mid-block sag in the street with no adequate outlet channel. In such a situation, the 100-year storm capacity standard for the major system could require the storm sewer to be sized to convey the runoff from a 100-year storm.

Time Distribution of Design Rainfall: The U. S. Soil Conservation Service (SCS) synthetic 24-hour Type II rainfall distribution was used to convert the desired rainfall amount for a given duration and frequency to a design storm.[1] The SCS Type II time distribution is shown in Figure 11 and the design storm pattern, or hyetograph, for a 10-year recurrence interval storm is given in Figure 12. The Type II rainfall distribution is considered to be applicable to most of the United States, including all of Wisconsin. The stormwater management and flood control plan presented here must give consideration to the effects of both storms of short duration and high intensity, which are critical for the determination of the peak discharges from relatively small, urbanized catchment areas served by conveyance systems, and storms of longer duration and greater runoff volume, which are critical for developing flood hydrographs used to size detention basins or to design flood control measures along the main channel of Lilly Creek. The Type II distribution is appropriate in an application such as this because it includes the effects of intense, short duration rainfalls while also producing runoff volumes characteristic of storms of longer duration.

Additional Hydrologic and Hydraulic Data: Data on the hydrologic and hydraulic characteristics of the study area were available from the files of the Commission, including data on soils, topography, drainage patterns of the natural streams and watercourses, waterway openings of related bridges and culverts and related flood hazard areas, wetlands, and areas with existing flood problems. Topographic maps prepared in 1987 and 1988 by Waukesha County and the Commission to Commission specifications at a scale of one inch equals 100 feet with contours

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[1] *U. S. Soil Conservation Service, Urban Hydrology for Small Watersheds, 2nd Edition, "Technical Release 55," 1986, Appendix B. Subsequently referred to as "TR-55."*

at two-foot intervals and 1985 Commission ratioed and rectified aerial photographs at a scale of one inch equals 400 feet were used in the analyses. Stormwater drainage system maps, construction plans, as-built surveys, development plans, and other pertinent information were obtained from the files of the Village and from Ruekert & Mielke, Inc., Consulting Engineers. These materials were evaluated and included in the body of resource materials drawn upon in the analytic and design phases of the work.

Simulation of Hydrologic, Hydraulic, and Nonpoint Source Pollutant Delivery Processes and Instream Habitat Evaluation: Quantification of the stormwater flow rates and volumes and of nonpoint source pollutant loading rates under both existing and probable future land use conditions allows sound, rational decisions to be made concerning stormwater management. Such quantification aids in determining the type, location, and configuration of stormwater management facilities and is essential to sizing facilities such as storm sewers, roadside swales, open channels, culverts and bridges, storage and pumping facilities, and nonpoint source pollution abatement measures. Rainfall-runoff modeling techniques were used under the study to quantify stormwater flow rate and volume in both the minor and major drainage systems.

Two mathematical simulation models were used to analyze flows in, and design system components of, the minor and major stormwater drainage systems.

1. The U. S. Army Corps of Engineers HEC-1 "Flood Hydrograph Package" model was utilized to develop, combine, and route the flood hydrographs generated for each catchment area in the subwatershed. That process of combining and routing hydrographs yielded total flood hydrographs at critical points along the natural watercourses in each subwatershed.

   Flood hydrographs for catchment areas with predominately rural land uses under existing and probable future conditions were developed using the U. S. Soil Conservation Service dimensionless unit hydrograph option of HEC-1. Under this procedure, runoff is determined by subtracting interception, infiltration, and surface storage losses from the design storm amounts. Such losses are determined using a runoff curve number calculated from the land use and hydrologic soil group distributions in a given subbasin. A unit hydrograph, representing one inch of runoff from a given subbasin for a given duration of rainfall excess, was developed for each subbasin by applying timing parameters characteristic of the subbasin to the SCS standard dimensionless unit hydrograph. The subbasin flood hydrograph was generated by applying each time increment of rainfall excess to the unit hydrograph and then summing the individual hydrographs for each storm time increment, according to the principal of superposition.

   Future condition flood hydrographs for some subbasins currently in urban land uses with engineered stormwater management facilities and for subbasins which are planned to undergo conversion of land from rural to urban uses were developed using the kinematic wave hydrograph development and routing option of HEC-1. To apply the kinematic wave procedure, the existing or planned stormwater drainage system for a given catchment area is idealized as several elements representing the overland flow, collection, and conveyance characteristics of the system. The kinematic wave form of the Saint Venant equations for one-dimensional, gradually varied unsteady flow are then solved to generate and route flood hydrographs through the drainage system. Rainfall excess amounts are determined by the SCS method already referenced.

   The HEC-1 model also has options for hydrograph combination and routing through stream channels and storage facilities. Those options were used to combine and route hydrographs from rural and urban areas and to size storage facilities. The HEC-1 model enables the evaluation of a complex hydrologic network, accounting for the effects of natural and man-made storage reservoirs on downstream peak flow rates.

2. The U. S. Army Corps of Engineers HEC-2 "Water Surface Profiles" model for gradually varied steady flow was used to determine flood stages and flood hazard areas along the streams which are part of the major drainage system. Flood profiles were

developed using the 100-year recurrence interval flood flows for existing, 1985, land use, drainage, and channel conditions and for planned ultimate land use conditions, with both existing and planned stormwater drainage and channel conditions. Where those profiles indicated the existence of problem flooding areas during the 100-year recurrence interval flood under planned land use conditions with recommended stormwater management measures in place, HEC-2 was used to evaluate alternative modifications to the channel and/or hydraulic structures for the purpose of alleviating the identified flooding problems.

The accuracy of the hydrologic and hydraulic simulation models for existing land use and channel conditions was checked by simulating flows and flood stages in Lilly Creek for the major storms of August 6, 1986, and September 9 through 11, 1986. Appropriate adjustments to the model were made to match the observed flood profiles, which were based on high water mark surveys provided by the Village. The results of the hydrologic simulations were also checked by comparison with previous analyses made for the Menomonee River watershed study, for the Village's Lilly Creek channelization proposal, and for the Lilly Creek stormwater management study prepared for the Village by Ruekert & Mielke, Inc., Consulting Engineers; and by application of the U. S. Geological Survey flood frequency equations for urban areas of Wisconsin.[2]

For the future design of specific minor system conveyance components with relatively small drainage areas and uncomplicated drainage networks, it is recommended that the Rational Method or U. S. Soil Conservation Service TR-55 methods be used to estimate flows.[3,4] If detention storage is to be provided, it is recommended that the TR-55 method for sizing detention basins be used. Experience indicates the Rational Method and the TR-55 methods should provide good results in the design of the components of relatively small, less complex drainage systems and the results obtained with those methods should be consistent with this system plan. For major system components and minor system components involving complex systems with relatively large drainage areas, it is recommended that design flows be computed using the hydrologic model developed for this stormwater management and flood control system plan by the Regional Planning Commission.

The Source Loading and Management Model (SLAMM) was applied by the Wisconsin Department of Natural Resources (DNR) to the urban nonpoint source pollution control studies conducted for the Menomonee River watershed nonpoint source control plan. The results of those studies were integrated into the system plan to the extent practicable. SLAMM was used to estimate existing pollutant contributions from various land use areas. The model was also used to estimate pollutant contributions under planned land use conditions; to evaluate the effects of various pollution abatement measures; and to determine nonpoint source pollutant loadings and to determine the effects of infiltration devices, including roadside swales, on SCS runoff curve numbers. Analyses with the SLAMM model were made using historical precipitation data from 1981. A considerable amount of urban nonpoint source pollutant loading data was collected in that year under the Nationwide Urban Runoff Program. Those data were used to calibrate the SLAMM model.

The U. S. Fish and Wildlife Service Habitat Suitability Index (HSI) model was applied by the DNR to establish habitat criteria for the fish species which are currently managed in the Lilly Creek subwatershed. Those criteria were then applied to evaluate the effects of alternative

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[2] *Duane H. Conger, Estimating Magnitude and Frequency of Floods for Wisconsin Urban Streams, U. S. Geological Survey Water-Resources Investigations Report 86-4005, prepared in cooperation with the Wisconsin Department of Transportation, the Milwaukee Metropolitan Sewerage District, and the Southeastern Wisconsin Regional Planning Commission, December 1986.*

[3] *SEWRPC Technical Record, Vol. 2, No. 4, April-May 1965.*

[4] *U. S. Soil Conservation Service, op. cit.*

stormwater management and flood control measures on the aquatic habitat of Lilly Creek. Where practicable, the results of that evaluation were incorporated into the recommended system plan.

Criteria and Assumptions

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

The location and size of inlets and culverts, as a part of the minor stormwater drainage system, are dictated by the allowable stormwater spread and depth of flow in streets and by the attendant interference with the safe movement of pedestrian and vehicular traffic.

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

With regard to inlets and parallel roadside culverts, it was generally assumed that these system components will be designed to provide sufficient capacity to intake and pass all flow in the tributary gutters or swales from storms up to and including the 10-year recurrence interval event. In cases where the street cross-section would not have sufficient hydraulic capacity to convey the difference in flow between a 10- and a 100-year event, inlets and roadside culverts were sized to provide capacity in excess of that needed for a 10-year event in order to insure proper functioning of the major drainage system. In the systems planning, critical locations were selected at which to check the specified overland and swale flow depths.

Roadside Swales: At the systems planning level, only recommendations relating to the general configuration, size, approximate depth, slope, and type of roadside swales are provided. More detailed engineering at the project development level will be needed to determine precise depth, location, and horizontal and vertical alignment of the swales and the best response to constraints posed by structures and utilities.

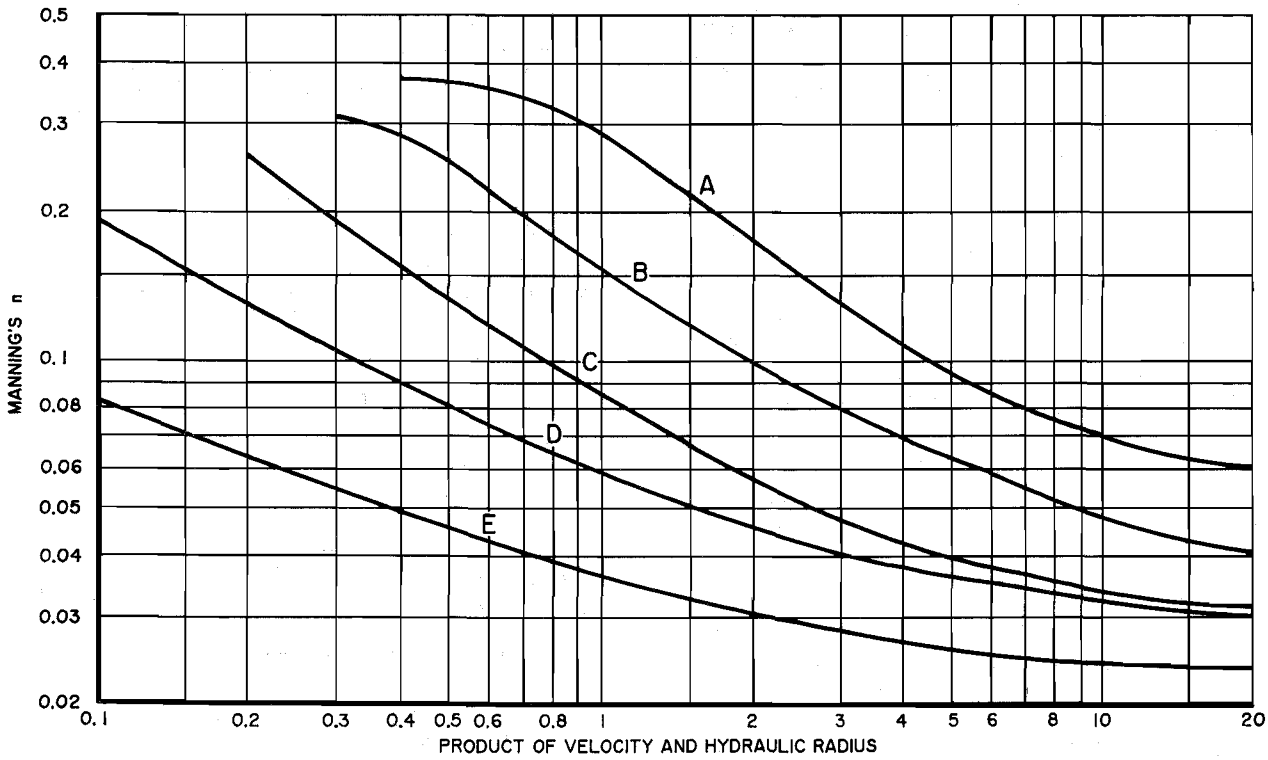
In the systems planning, the Manning equation was used together with the cross-sectional area of flow to determine the required hydraulic capacity of swales. A Manning's "n" value corresponding to retardance level "D" in Figure 13 was assumed for well-constructed, properly-maintained, frequently-mowed, grass-lined roadside drainage swales, such as may be expected to exist adjacent to the front yards in residential areas. A Manning's "n" value corresponding to retardance level "C" in Figure 14 was assumed for properly constructed but less frequently maintained, in a one- to two-month mowing cycle, grass-lined roadside drainage swales commonly found in rural areas.

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

1. Swales were assumed, in general, to be located in public street rights-of-way and to follow the street alignments and gradients.
2. Swale cross sections were assumed to be triangular with side slopes of one vertical on four horizontal adjacent to the roadway and no steeper than one vertical on three horizontal away from the roadway, as shown in Figure 2 in Chapter III of this report. Where practicable, a trapezoidal cross section was assumed with the bottom width selected to promote infiltration if found to be cost effective.
3. Swales were assumed to be designed to accommodate the peak runoff expected

Figure 13

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



Source: U. S. Soil Conservation Service.

from a minor, or 10-year recurrence interval, storm when flowing full and without freeboard.

4. All swales were assumed to be designed to provide a maximum flow velocity of five feet per second during the design storm event.

5. The minimum depth of swales below the street shoulder was assumed to be 1.5 feet, while the maximum depth was assumed to be three feet.

Cross Culverts and Channel Enclosures: Cross culverts, which are a common feature of open drainage systems, are used to convey stormwater under a street, highway, railway, or embankment. Channel enclosure involves placing a stream channel in a covered conduit to convey runoff in order to reduce overland flooding. Because the hydraulic methods used to design channel enclosures are often the same as those used to size culverts, these two components are here addressed with one set of criteria. Depending on the length of a given channel enclosure, there may also be similarities to a storm sewer. At the systems planning level, recommendations concerning the location and size of cross culverts and channel enclosures are provided. More detailed engineering at the project development level will be needed to determine precise depth, location, and horizontal and vertical alignment of the culverts and channel enclosures; the type of material to be used; and the best response to constraints posed by structures and utilities.

The hydraulic capacity of any culvert is affected by its cross-sectional area, shape, entrance geometry, length, slope, construction material, and depth of ponding at the inlet and outlet, details which must be addressed at the project development level. Culvert flows are classified as having either inlet or outlet control, that is, whether the discharge capacity is controlled by the inlet or outlet characteristics. Typical inlet control and outlet control culvert conditions are shown in Figure 14. Under inlet control conditions, the discharge capacity of a culvert is controlled at its entrance by the depth of headwater, the entrance shape and cross-sectional area, and the type of entrance edge. Under outlet control conditions, the discharge capacity of a culvert is influenced by the headwater depth, tailwater depth, entrance shape and cross-sectional area, the type of entrance edge, and by the cross-sectional area, shape, slope, length, and roughness of the culvert barrel.

In planning the system, required culvert or channel enclosure sizes were determined by evaluating multiple constraints and selecting a size which appeared appropriate to meet all requirements best. Nomographs and capacity charts are available in the literature for varying pipe shapes, sizes, materials of construction, and entrance conditions.

Manning's "n" values, as shown in Figure 15, were assumed for properly installed and maintained corrugated metal pipe and pipe arch culverts or channel enclosures. A Manning's "n" value of 0.013 was assumed for well-constructed, precast concrete pipe culverts or channel enclosures flowing full. Where analyses indicated that pipes would flow less than full at design loading, the hydraulic element charts set forth in Figures 16 and 17 were used in the solution of Manning's equation or were computed directly in the simulation model. Hydraulic conditions for major system components under major storm event conditions were evaluated on a case-by-case basis.

The following criteria and assumptions were used in the development of culvert sizes for the stormwater management system plan. Additional criteria relating to environmental mitigation features for culverts are given in a subsequent section of this chapter.

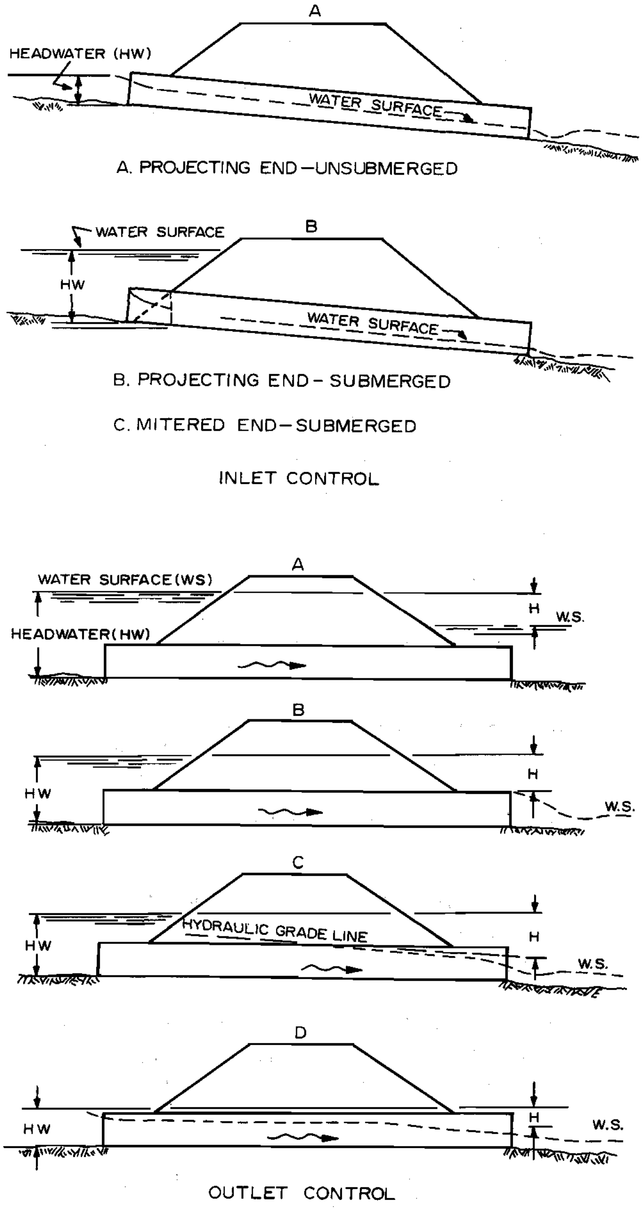
1. Cross culverts and channel enclosures were designed to meet street, highway, expressway, and railway overtopping Standards No. 2a through 2d under Objective No. 3 in this chapter.

2. The culvert locations were assumed to provide a direct exit, avoiding an abrupt change in direction at the outlet end and, preferably, also at the inlet end.

3. The minimum culvert size used was 12 inches in diameter.

Figure 14

CULVERT HYDRAULIC CONDITIONS



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

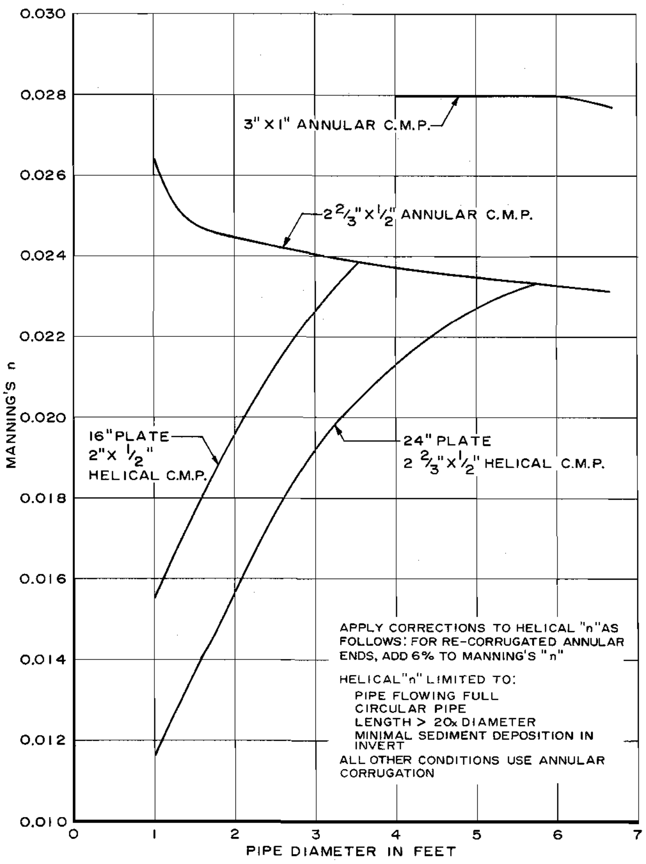
4. Cross culverts were assumed to be laid on a constant gradient.

5. New culverts were assumed to be circular pipes or pipe arches, constructed of corrugated metal.

6. Culvert and channel enclosure inlets were assumed to be unblocked.

Figure 15

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



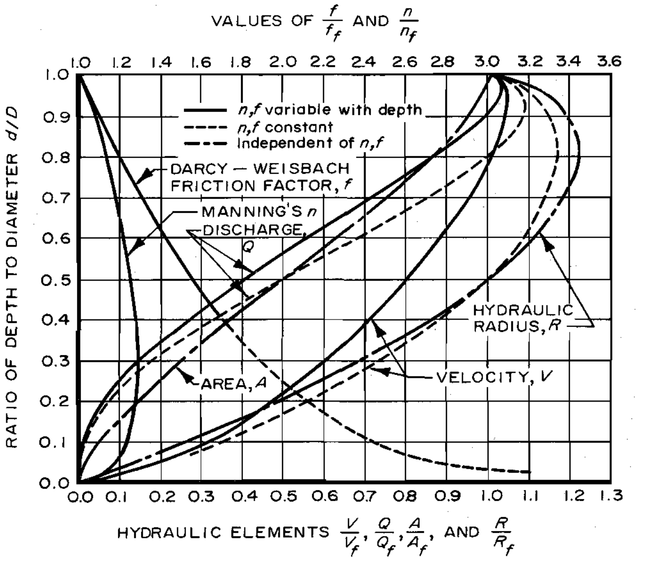
Source: *U. S. Department of Transportation, Hydraulic Flow Resistance Factors for Corrugated Metal Conduits, and SEWRPC.*

7. Manholes were assumed to be provided at all slope and alignment changes in channel enclosures.

8. Appropriate energy dissipation and/or erosion protection should be provided at the inlets and outlets of culverts and channel enclosures. The type of protection will be dictated by site-specific hydraulic considerations determined during the facility design phase.

Figure 16

HYDRAULIC ELEMENTS GRAPH FOR CIRCULAR SEWERS

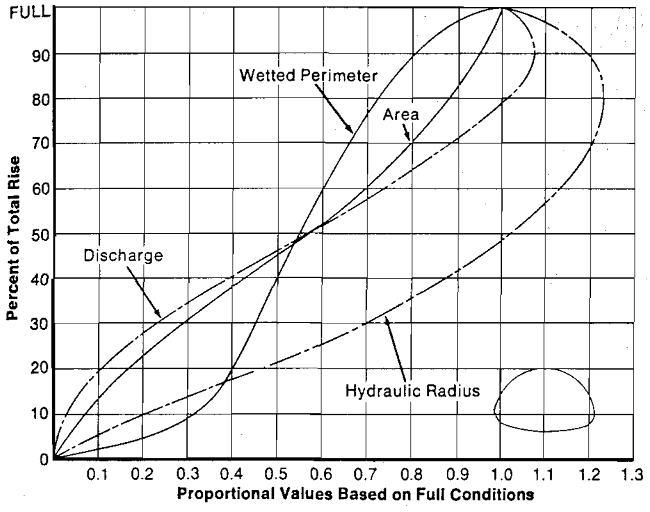


NOTE: n, V, Q, AND R REPRESENT PARTIAL FLOW CONDITIONS. n_f, V_f, Q_f, AND R_f REPRESENT FULL FLOW CONDITIONS.

Source: American Society of Civil Engineers.

Figure 17

HYDRAULIC PROPERTIES OF CORRUGATED STEEL AND STRUCTURAL PLATE PIPE ARCHES



Source: American Iron and Steel Institute.

Open Drainage Channels: Open drainage channels in and along exclusive rights-of-way are a necessary and appropriate component of the total stormwater drainage and flood control system within the subwatershed. Such channels may, in certain areas, serve as part of the minor drainage system, as, for example, in parks and cemeteries, in some commercial and industrial areas, and in some low-density residential areas.

Such channels form part of the major stormwater drainage system as well. In some areas of the subwatershed, open drainage channels, together with roadside swales, may serve as the sole component of the engineered stormwater drainage system to convey surface runoff to the receiving natural stream system. To achieve flood control objectives, portions of the receiving stream system may have to be modified to convey anticipated flows safely.

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

In the system planning, the Manning's equation was used to determine the hydraulic capacity of open channels. Careful consideration was given to allowable grades and depths of flow to prevent unacceptable velocities and damage to the facilities and adjacent land uses. Where backwater effects were important or where flood hazard areas were delineated, the HEC-2 step backwater simulation model was used.

The following criteria relating to the details of the open drainage channels for stormwater drainage and flood control were used in the development of the stormwater management plan and can also be used as guidelines in facility design. Additional criteria relating to environmental mitigation features for open channels are given in a subsequent section of this chapter.

1. All open drainage channels were designed to accommodate the peak runoff from a 100-year recurrence interval storm under planned land use and channel conditions.

2. Where practicable, major flood control channels were designed with a two-foot freeboard above the design flood elevation.

3. Features to mitigate adverse impacts on fish and wildlife habitat were incorporated into the design of channel modifications.

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

5. Alternative cross-sections for modified channels using turf or riprap lining are shown on Figure 18. Selected design criteria for the various alternative channel types are summarized in Figure 18 and Table 27.

    a. Turf-lined, or Type A, channels were used to the maximum extent practicable. Where there was adequate right-of-way, such channels were assumed to have maximum side slopes of one vertical on four horizontal. In no instance would the side slopes be steeper than one vertical on two horizontal. A Manning's "n" value of 0.030 to 0.035 was used. The velocity during a 100-year recurrence interval flood was limited to a maximum of six feet per second. A maintenance access road was assumed to be located along the top of the bank, or along a 12-foot-wide maintenance bench, as shown on Figure 19. Where deemed necessary for environmental protection, as discussed later in this chapter, a base-flow channel was provided.

    b. Riprap-lined, or Type B, channels were provided if erosive velocities were expected to develop in turf-lined channels. A typical channel section for this situation is shown as Type B on Figure 19. Where feasible, riprap-lined channel side slopes were assumed to be one vertical on three horizontal, but in no case steeper than one vertical on two horizontal. A Manning's "n" value of 0.035 was used. The velocity during a 100-year recurrence interval flood was limited to a maximum of 10 feet per second. Where deemed needed for environmental protection, a base-flow channel was provided.

6. Where right-of-way restrictions or hydraulic considerations prevent use of turf-lined channels, fully- or partially-lined concrete channels could be used, as shown on Figure 19, Types C through F. A Manning's "n" value of 0.015 should be used for concrete channels. Composite turf- and concrete-lined channels should be designed using the appropriate "n" for each segment of the channel cross section.

   a. Partially turf-lined, or Type C, channels with concrete invert could be used in residential areas where necessary because of right-of-way or other limitations. Where practical, the turf-lined side slopes should be one vertical on four horizontal, but in no instance steeper than one vertical on two horizontal. During the 100-year recurrence interval flood, the maximum velocity should be limited to six feet per second.

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

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

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

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

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

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

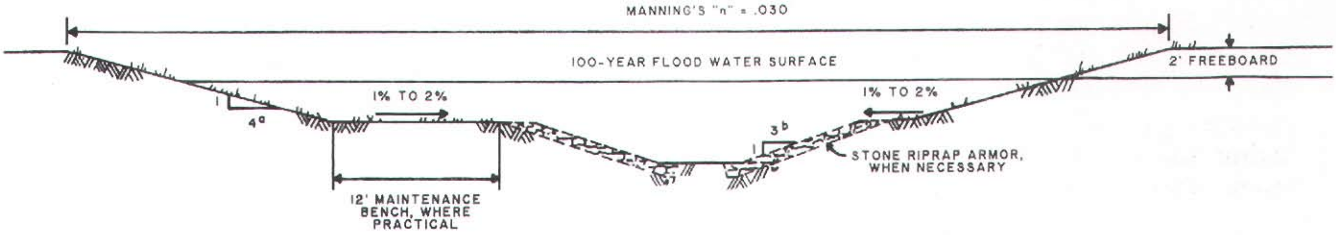
10. Appropriate energy dissipation and erosion protection were assumed to be provided at any grade control structures. The type of protection will be dictated by site-specific hydraulic considerations.

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

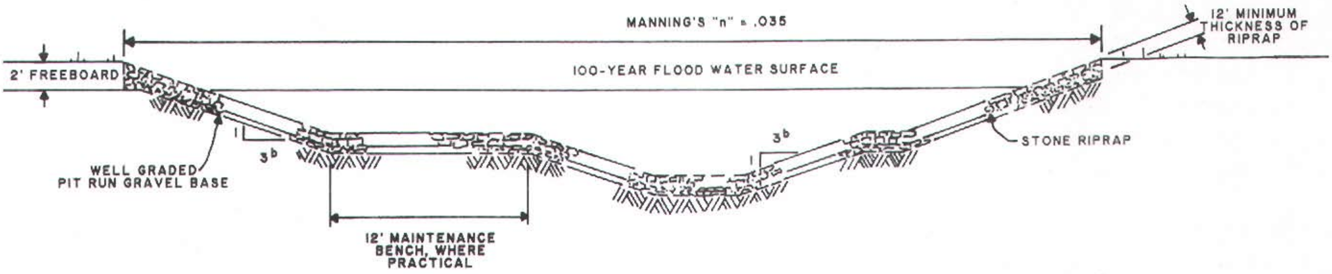
Figure 18

TYPICAL MODIFIED CHANNEL CROSS-SECTIONS

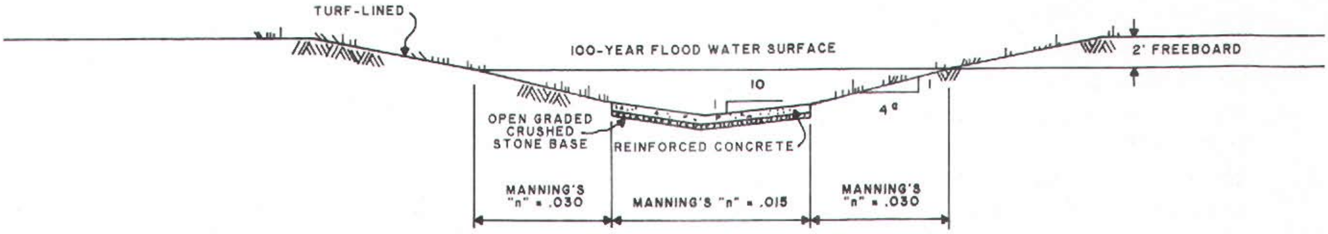
TYPE A
TURF-LINED CHANNEL



TYPE B
RIPRAP-LINED CHANNEL



TYPE C
PARTIALLY TURF-LINED WITH CONCRETE INVERT CHANNEL



TYPE D
PARTIALLY CONCRETE-LINED CHANNEL

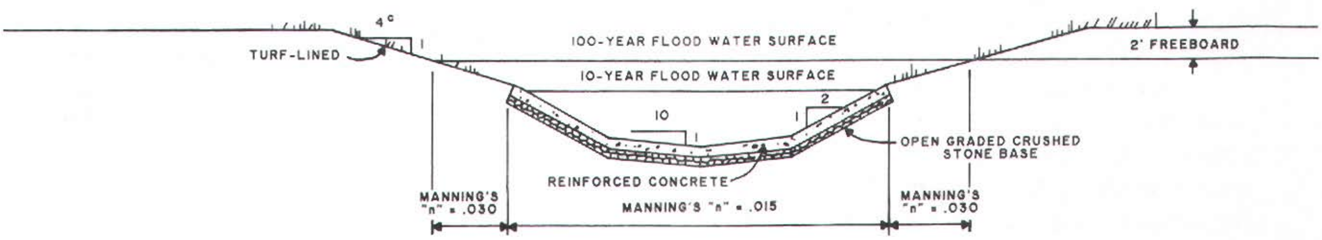
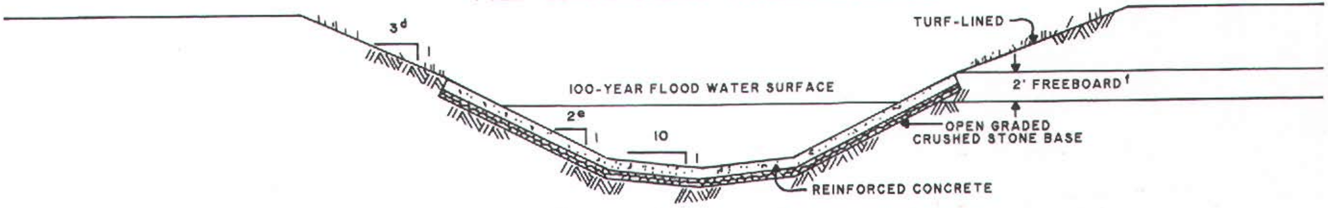
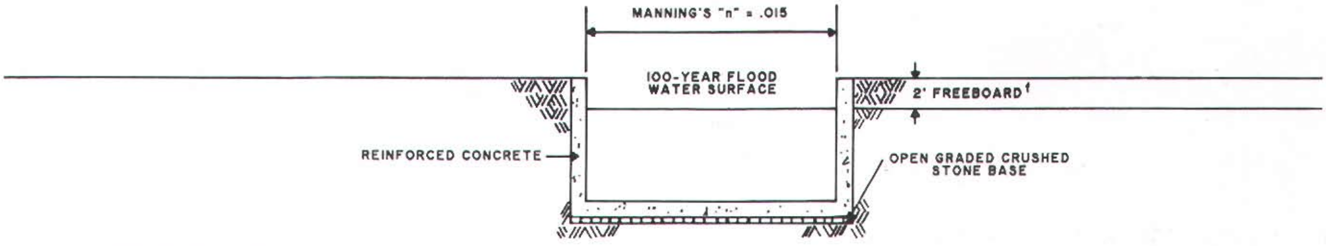


Figure 18 (continued)

TYPE E
FULLY CONCRETE-LINED TRAPEZOIDAL CHANNEL



TYPE F
CONCRETE-LINED RECTANGULAR CHANNEL



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

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

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

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

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

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

Source: SEWRPC.

Source: SEWRPC.

Table 27

CHANNEL MODIFICATION DESIGN CRITERIA

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

[a] Only the channel bottom is concrete.

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

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

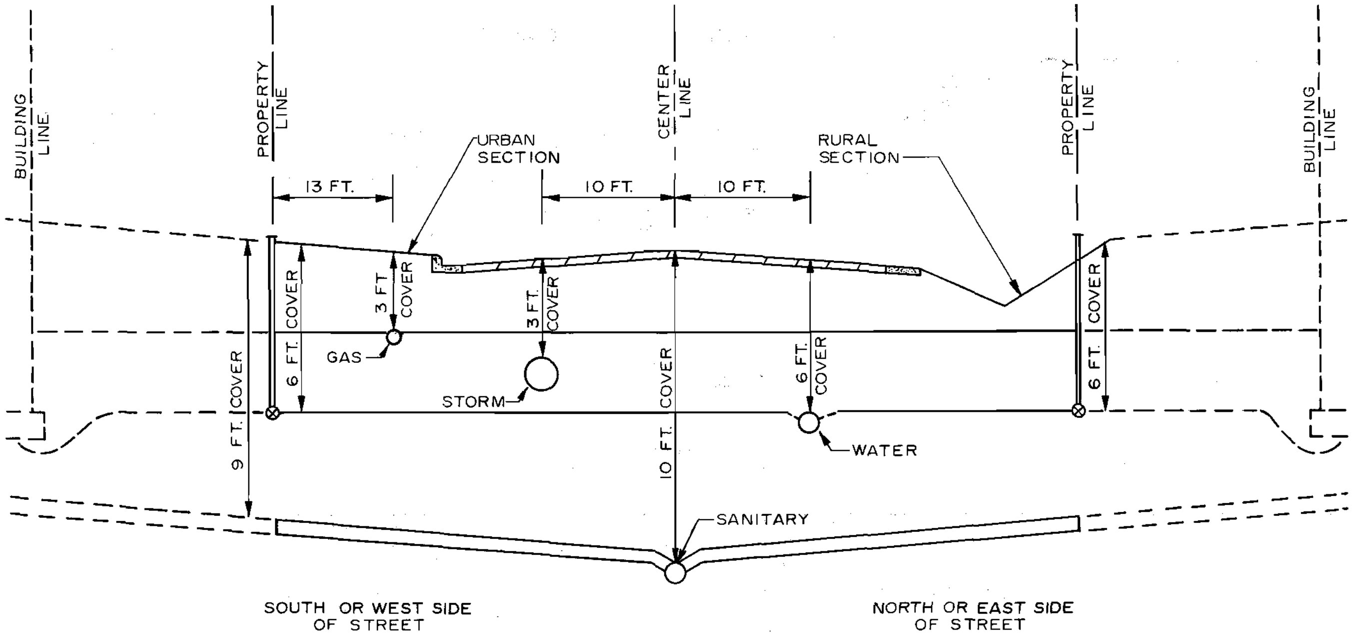
Source: SEWRPC.

Storm Sewers: At the systems planning level, only recommendations for the general configuration, size, approximate invert elevation, slope, and type of storm sewer facilities are provided. More detailed engineering at the facility design level will be needed to determine the precise invert elevation, location, and horizontal and vertical alignment of the sewer, the type of material used for the sewer, and the best response to constraints posed by structures and other utilities. It is recommended that, to the extent practicable, stormwater management facilities be located generally as shown in Figure 19.

In the system planning, Manning's equation was used together with the cross-sectional area of flow to determine the hydraulic capacity of sewers. Values for the Manning's roughness

Figure 19

SUGGESTED UTILITY LOCATIONS IN THE VILLAGE OF MENOMONEE FALLS



Source: Village of Menomonee Falls and SEWRPC.

coefficient "n" vary with the type and conditions of the sewer, the depth of flow in the sewer, and the diameter of the sewer. A Manning's "n" value of 0.013 was assumed to be typical of well-constructed, precast, concrete pipe sewer lines. Manning's "n" values for existing corrugated metal storm sewer lines were determined using Figure 15.

Where the analyses indicated the sewers would flow less than full at design loading, either the hydraulic element chart set forth in Figure 16 was used to determine the critical characteristics or those characteristics were computed directly in the simulation model.

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

1. Storm sewers were assumed generally to be located in public street rights-of-way and to follow the street alignments and gradients.
2. Storm sewers were designed to accommodate the peak runoff expected from a minor, that is, a 10-year recurrence interval, storm when flowing full.
3. The minimum pipe size used was 12 inches in diameter.
4. The minimum desirable velocity during the design storm event was assumed to be 2.5 feet per second.
5. Planned storm sewer outlet invert elevations were assumed to be above the channel bottom elevations of the receiving watercourses. This criterion assumes periodic cleaning and maintenance of stream channels.
6. The minimum depth of cover over the top of the sewer was assumed to be three feet.

Stormwater Storage Facilities: Natural storage of stormwater is provided during overland flow in surface depressions, vegetated areas, and pervious soils. Natural storage can be enhanced by preserving open areas, woodlands, wetlands, ponds, and areas with large infiltration capacities. These attributes can be utilized in a stormwater management system at less cost than would be required for the incorporation of artificial storage facilities. Artificial storage facilities include constructed onsite swales, roadside swales, temporary storage facilities on parking lots and other open areas, and retention and detention basins. Under this system planning effort, stormwater storage facilities were considered for one or more of the following purposes: 1) stormwater drainage, 2) nonpoint source pollution control, and 3) flood control.

At the systems planning level, recommendations concerning only the location, type, approximate size, and capacity of storage facilities and outlet flow constraints are provided. More detailed engineering at the project development level will be needed for precise location, configuration, and sizing of storage facilities and to specify such details as the inlet and outlet control facilities. In planning the system, required storage volumes for stormwater drainage or flood control were calculated using the HEC-1 simulation model. Required wet detention basin sizes for nonpoint source pollution control were determined using procedures developed by the Wisconsin Department of Natural Resources. Extended detention volumes for streambank and erosion control were determined using procedures developed by the Metropolitan Washington Council of Governments.[5,6] The following criteria relating to storage facilities were used in the development of the stormwater management system plan:

1. Storage facilities were sized to control a range of storms depending upon intended purposes. Storage facilities intended to serve as components of the minor drainage system were sized to control storms with recurrence intervals ranging from two to 10 years, under planned land use and channel system conditions. Storage facilities designed as components of the downstream floodland management system were sized to accommodate storms with recurrence intervals ranging from two to 100 years. Storage systems planned for water quality purposes were designed to control storms with recurrence intervals up to and including two years.
2. Where practical, storage facilities for stormwater drainage purposes were designed to limit the design outflow to no more than the capacity of the existing downstream conveyance and storage systems.
3. The effects of storage facilities on the possible increase in the frequency, duration, and magnitude of downstream flooding under future conditions as compared to existing conditions was carefully examined. Routing through a storage facility significantly flattens the outflow hydrograph in comparison to the inflow hydrograph. Peak flows are reduced and the duration of peak, or near-peak, flows increased. When prolongation of near-peak flows causes those flows to coincide with near-peak flows of upstream or downstream tributaries, the storage facilities were designed so as not to increase combined future downstream flood peaks to an unacceptable level.
4. Storage depths on parking lots, truck stops, and similar open spaces were assumed to not exceed six inches during the design flood event.
5. Storage facilities which include dams or earth embankments to detain floodwaters were assumed to include an emergency spillway in order to pass flows up to and including those resulting from a 100-year recurrence interval storm safely, with appropriate freeboard.

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

The following criteria and assumptions relating to stormwater pumping facilities were used in the development of the stormwater management system plan. They may also be used as guidelines in facilities design.

1. Consideration was given to the feasibility of providing gravity drainage as an alternative to pumping facilities.

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[5] *Thomas R. Schueler, Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMPs, Metropolitan Washington Council of Governments, 1987.*

[6] *Thomas R. Schueler, personal communication, July 1990.*

2. An evaluation was made of the ability of the pumping station and any associated gravity drainage facilities to provide protected areas with relief from flooding during storms ranging up to and including the 100-year recurrence interval storm. The possibility of different frequency storms occurring simultaneously over the protected area and the entire area tributary to the main receiving stream was considered.

3. The pumping station was assumed to have a gravity overflow to the major drainage system.

4. The pumps were assumed to be high-capacity, low-head centrifugal pumps with constant-speed motors designed for intermittent service.

Bridge and Culvert Alteration or Replacement: The following design criteria were used in the system planning in considering bridge and culvert alteration or replacement. These criteria may also be used as guidelines in facilities design.

1. Bridge and culvert alterations were designed to meet street, highway, expressway, and railway overtopping Standards No. 2a through 2d under Objective No. 3 in this chapter.

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

3. Except at structures where blockage of the waterway opening was identified as an historic problem, backwater computations assumed proper waterway opening design and maintenance so that the full waterway opening of each proposed or existing bridge or culvert was available for the conveyance of flood flow.

4. At existing structures where significant blockage of the waterway opening was known to have occurred consistently during past floods, the backwater computation for determination of the design flood profile under existing conditions was made assuming partial blockage of the opening commensurate with available historic observations.

5. Manning's "n" values as shown in Figure 16 were used for properly installed and maintained corrugated metal pipe and pipe arch culverts. A Manning's "n" value of 0.032 was used for all structural plate pipe and pipe arch culverts.

6. A Manning's "n" value of 0.013 was used for well-constructed concrete pipe flowing full.

7. Where analyses indicate that pipes would flow less than full at design loading, the hydraulic element charts set forth in Figures 17 and 18 were used to determine critical characteristics required for solution of Manning's equation.

8. Criteria 2, 4, and 8, stated previously for cross culverts, were applied.

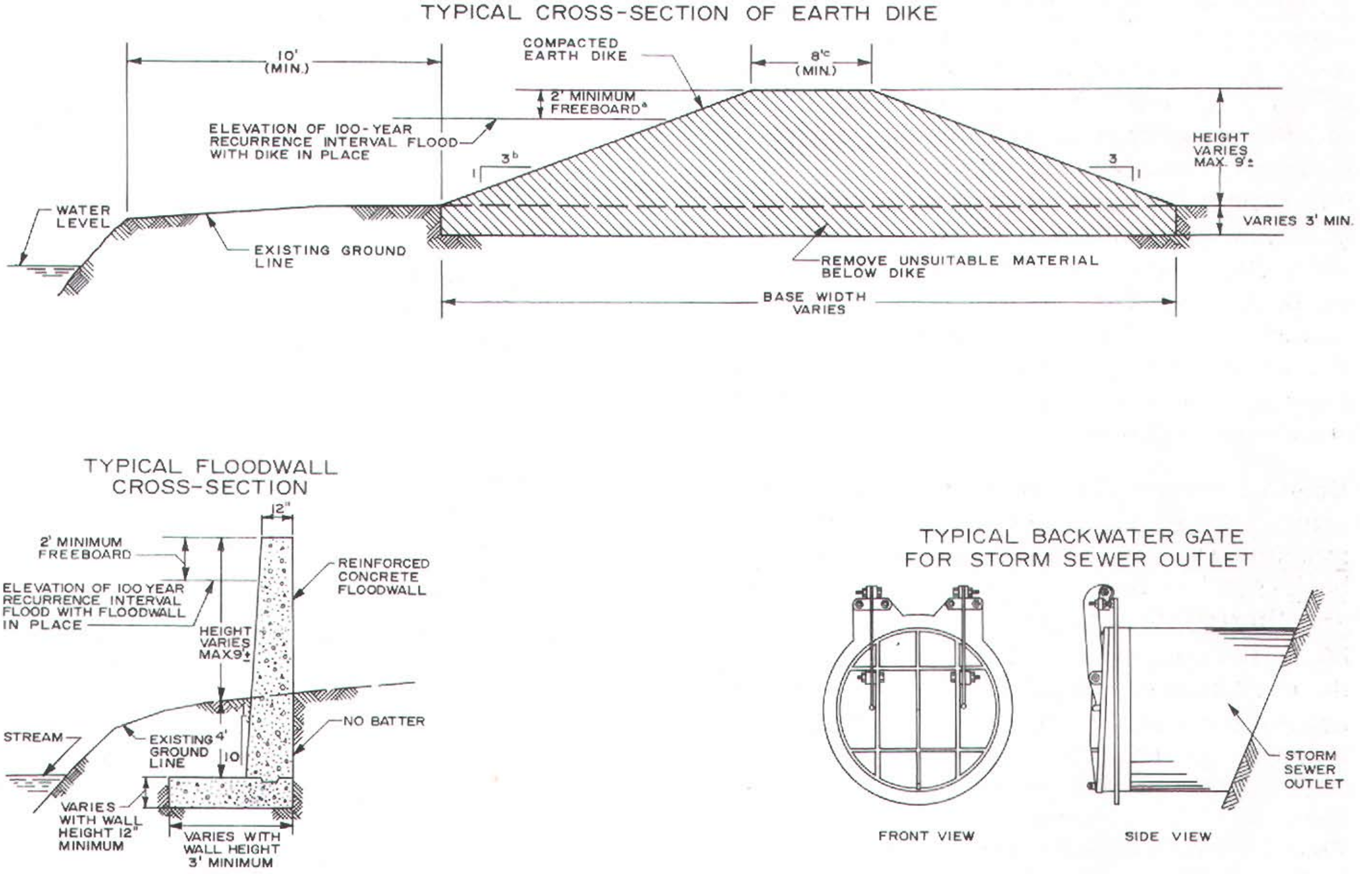
Dikes and Floodwalls: Where the floodplain topography is flat and there is considerable damage-prone development in the floodplain, dikes and floodwalls can be used to provide flood control. Typical dike and floodwall cross sections are shown in Figure 20. The following design criteria were used in the system planning for dikes and floodwalls. These criteria may also be used as guidelines in facilities design.

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

2. Dikes or floodwalls were designed so as not to increase the height of the 100-year recurrence interval flood by 0.01 foot or more in any unprotected upstream or downstream stream reaches. Increases in flood stages equal to or greater than 0.01 foot resulting from any dike or floodwall construction were contained within the upstream or downstream extent of the dike

Figure 20

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



Source: Water Resources Research Institute and SEWRPC.

or floodwall, except where topographic or land use conditions could accommodate the increased stage without creating additional flood damage potential.

3. In cases where a dike or floodwall was intended to protect human life, the minimum dike or floodwall top elevation was determined using whichever of the following produced the highest profile:

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

   b. The 500-year flood profile.

The height of low dikes or floodwalls which are not intended to protect human life were based on the high water surface profiles for the 100-year recurrence interval flood and were designed to be capable of passing the 100-year recurrence interval flood with a freeboard of at least two feet.

4. Dike slopes were normally assumed to be one vertical on three horizontal, and not steeper than one vertical on two and one-half horizontal.

5. For dikes with heights of six feet or less, the minimum top width was assumed to be six feet. For dikes with heights greater than six feet, the minimum top width was assumed to be eight feet.

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

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

Detailed criteria for construction site pollutant control are given in the DNR Wisconsin Construction Site Best Management Practice Handbook (April 1989). The Construction Site Erosion and Stormwater Management Plan and Model Model Ordinance (draft, 1985), also prepared by the DNR, contains detailed design procedures for nonpoint source pollution control measures. The following general criteria for nonpoint source control measures were considered in the development of this stormwater management plan. These criteria may also be used as guidelines in facilities design.

1. Pretreatment of storm runoff to infiltration devices was considered to minimize clogging and reduce maintenance. Such pretreatment was assumed to consist typically of a sedimentation box. The addition of a sedimentation-flotation basin to trap oil and grease was considered when the device would be constructed in a commercial area.

2. Where grass swales are intended to maximize infiltration the longitudinal slope assumed was less than 5 percent. Perforated drainage pipes were assumed to be used only where the longitudinal slopes were less than 3 percent.

3. Where feasible, to avoid short-circuiting of flow and to maximize the efficiency of wet detention basins, the minimum basin length-to-width ratio was set at three to one or baffles were assumed to be provided to increase the flow length.

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

5. Design of retention basins and other infiltration systems at the facilities level requires site-specific investigations to establish design parameters and to avoid groundwater contamination. Important considerations related to the assessment of the potential for groundwater contamination are soil permeability, depth to the water table, depth to bedrock, and the existing and potential future uses of the receiving groundwater. For this system planning effort, the location of infiltration systems was limited to areas covered by relatively permeable Hydrologic Soil Group A or B soils, where the depth to the seasonally high water table is greater than five feet, and where the site land slopes do not exceed 5 percent.

6. The maximum area draining to a single infiltration trench was assumed to be five acres.

Environmental Mitigation Measures: Features to mitigate adverse impacts on fish and wildlife habitat were considered in the design of channel modifications. At the systems planning level, only the type and general location and configuration of environmental mitigation measures were considered. Tables 28 through 30 set forth design features appropriate for achieving various environmental goals. Tables 31 and 32 can aid in the determination of the suitability of certain design features for application in various stream and watershed settings. The detailed design of mitigation measures will require an inventory of site-specific conditions and will need to be coordinated with associated stormwater management and flood control measures.

Table A-1 in Appendix A to this report presents the results of application of the U. S. Fish and Wildlife Service Habitat Suitability Index model for selected fish species currently managed in the Lilly Creek subwatershed. The model was used to determine optimum values of various habitat variables. The optimum values for the species included in the table are as stringent as, or more stringent than, the corresponding values for other species which occur, or may occur, in Lilly Creek. The criteria given in the table and the associated criteria listed in Table A-2 are intended to be considered in the facilities design of instream habitat mitigation measures for all species managed in the subwatershed.[7]

The following general criteria for environmental mitigation measures in streams with an existing or potential valuable fishery were considered in the development of the system plan. The criteria, along with those in Appendix A, can also serve as guidelines in facilities design.

1. As shown on Figure 13 of Chapter III of this report, stream modifications were designed to include a small channel for concentration of base flows and a flood channel for the conveyance of large flows.
    a. Where the existing stream channel provides adequate aquatic habitat, low-flow channels in alluvial material should approximate the size, shape, and sinuosity of the existing stream channel to be modified. Based on observations of existing conditions in Lilly Creek, the average width of the low-flow channel should be from two to four feet and the depth should be about one foot. To sustain fish and other aquatic life during periods of low flow, deeper pools, which extend below the mean streambed elevation, should be provided.
    b. The flood channel, in combination with the low-flow channel, should be designed to accommodate the peak runoff from a 100-year recurrence interval storm under planned land use and channel conditions.
2. Where practicable, channel modifications should be designed to minimize the disturbance of the existing stream channel by retaining the existing low-flow channel and modifying only one side of the flood channel.
3. Culverts should be designed to permit fish migration.

ECONOMIC EVALUATION

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

The economic evaluations conducted under this stormwater management and flood control

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[7] *The optimum values in Table A-1 were developed based solely on considerations related to fish habitat. The detailed criteria and parameters given in that table were not specifically addressed at the systems planning level. They are presented to provide guidance for the facilities design of instream habitat mitigation measures. In order to strike an acceptable balance between the objectives of stormwater management, flood control, and preservation of instream habitat consistent with this systems plan, it may be necessary to accept less than optimum values of some or all habitat indices. The primary purpose of this planning effort is to provide an integrated system of stormwater drainage, nonpoint source controls, and flood control measures by accommodating hydraulic and nonpoint source pollutant loadings under planned land use conditions. The provision of instream habitat mitigation measures was considered for those stream reaches with an existing or potential valuable fishery where channel modifications are recommended to meet stormwater management and flood control objectives and in stream reaches which are not recommended for modification but where stream enhancemnt measures are desirable for the improvement of the fishery resource.*

Table 28

SELECTION OF ENVIRONMENTAL MITIGATION FEATURES FOR MODIFIED CHANNELS

Environmental Feature	Environmental Goals[a]						
	Limit Bed and Bank Erosion	Avoid Bed Aggradation	Prevent Groundwater Lowering	Improve Low-Flow Conditions	Improve Water Quality	Improve Preserve Fish Habitat	Aquatic Habitat Diversity
Selective Clearing and Snagging	--	X	X	X	X	--	--
Traditional Clearing and Snagging	--	X	--	--	--	--	--
Low- and Normal-Flow Channels	--	X	--	X	X	--	X
Diversions and Impass Channels	--	--	--	X	--	--	--
Meandering Alignments	X	--	--	--	--	--	X
Pool and Riffle Grades	--	--	--	X	X	X	X
Single Bank Modification	X	--	--	--	X	X	X
Grade Control Structures	X	--	--	--	--	--	--
Armor	X	--	--	--	--	X	X
Rigid Linings	X	--	--	--	--	--	--
Bank Protection	X	X	--	--	--	--	--
In-Stream Habitat Structures	--	--	--	X	X	X	X
Water Level Control Structures	--	--	X	X	X	X	X
Fishways	--	--	--	--	--	X	--
Substrate Construction	X	--	--	--	--	X	X
Oxbow and Bendway Maintenance	--	--	X	--	X	X	X
Greentree Areas	--	--	X	--	--	--	--
Vegetative Plantings	X	--	--	--	--	--	--
Placement and Shaping of Spoil	--	--	--	--	--	--	--
Preservation of Cutoff Islands	--	--	--	--	--	--	--
Sediment Traps	--	X	--	X	--	--	--
Scheduling Work for Environmental Reasons	--	--	--	--	--	X	--
Vegetative Buffer Strips	X	X	--	--	X	X	--
Revegetation of Disturbed Areas	X	X	--	--	--	--	--
Special Structures for Stream-Based Recreation	--	--	--	--	--	--	--
Trails	--	--	--	--	--	--	--
Picnic Areas, Campgrounds, Etc.	--	--	--	--	--	--	--
Playgrounds, Sport Fields, Etc.	--	--	--	--	--	--	--
Passive Recreational Areas	--	--	--	--	--	--	--

planning program include capital cost estimates and annual operation and maintenance cost estimates. Capital costs include construction contract costs plus engineering, inspection, and contract administration costs. Cost data for stormwater drainage and flood control measures are presented in Appendix B. Cost data for urban nonpoint source pollution control measures were obtained from from SEWRPC Technical Report No. 31, recently prepared by the Regional Planning Commission.[8]

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[8] *SEWRPC Technical Report No. 31, Costs of Urban Nonpoint Source Water Pollution Control Measures, June 1991.*

Table 28 (continued)

Environmental Feature	Environmental Goals[a]						
	Reduce Loss of Riparian Vegetation	Improve Terrestrial Habitat Diversity	Mitigate Wetlands	Improve In-Stream Aesthetics	Improve Streamside Aesthetics	Improve In-Stream Recreation	Improve Streamside Recreation
Selective Clearing and Snagging	X	X	- -	X	- -	X	X
Traditional Clearing and Snagging	- -	- -	- -	- -	- -	- -	- -
Low- and Normal-Flow Channels	- -	- -	- -	X	- -	X	- -
Diversions and Impass Channels	- -	- -	- -	- -	- -	- -	- -
Meandering Alignments	- -	- -	- -	X	- -	- -	- -
Pool and Riffle Grades	- -	- -	- -	X	- -	X	- -
Single Bank Modification	X	X	- -	X	X	- -	X
Grade Control Structures	- -	- -	- -	- -	- -	- -	- -
Armor	- -	- -	- -	X	- -	- -	- -
Rigid Linings	- -	- -	- -	- -	- -	- -	- -
Bank Protection	- -	- -	- -	- -	- -	- -	- -
In-Stream Habitat Structures	- -	- -	- -	X	- -	X	- -
Water Level Control Structures	- -	- -	- -	X	- -	X	- -
Fishways	- -	- -	- -	- -	- -	- -	- -
Substrate Construction	- -	- -	- -	- -	- -	- -	- -
Oxbow and Bendway Maintenance	- -	- -	X	X	- -	X	- -
Greentree Areas	X	X	X	- -	X	- -	X
Vegetative Plantings	X	X	- -	- -	X	- -	- -
Placement and Shaping of Spoil	- -	X	X	- -	X	- -	- -
Preservation of Cutoff Islands	X	X	- -	- -	X	- -	X
Sediment Traps	- -	- -	- -	X	- -	- -	- -
Scheduling Work for Environmental Reasons	- -	- -	- -	- -	- -	X	- -
Vegetative Buffer Strips	X	X	- -	- -	X	- -	X
Revegetation of Disturbed Areas	X	X	- -	- -	X	- -	- -
Special Structures for Stream-Based Recreation	- -	- -	- -	- -	- -	X	- -
Trails	- -	- -	- -	- -	- -	- -	X
Picnic Areas, Campgrounds, Etc.	- -	- -	- -	- -	- -	- -	X
Playgrounds, Sport Fields, Etc.	- -	- -	- -	- -	- -	- -	X
Passive Recreational Areas	- -	- -	- -	- -	- -	- -	X

[a] *"X" indicates that the environmental feature has been or can be used to achieve the environmental goal.*

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

Where feasible, construction cost curves for entire stormwater drainage components are presented in Appendix B to this report. Such curves are given for storm sewers, dikes, floodwalls, circular culverts, and pumping stations. For other structural drainage and flood control measures, unit construction costs for each element of the particular measure are tabulated. Unit cost tabulations are provided for bridge alteration or replacement, channel modifications, and channel enclosures. Where site-specific conditions were expected to result in unit

Table 29

SELECTION OF BANK PROTECTION FEATURES TO MEET ENVIRONMENTAL GOALS

Protection Feature	Environmental Goals[a]							
	Riparian Habitat Diversity	Riparian Habitat Value	Substrate for Benthic Macro-Invertebrates	Fish	Water Quality	Aesthetics	Stream Access	Aquatic Habitat Diversity
Composite Revetment	X	X	X	--	--	X	X	X
Reinforced Revetment	X	X	X	--	--	--	--	--
Windrow Revetment	X	X	--	--	--	X	X	--
Modified Revetment	X	X	--	--	--	X	X	--
Berm Preservation, Protection, and Restoration	X	X	--	--	--	X	X	X
Toe Protection	X	X	--	--	--	X	X	--
Excavated Bench Design	X	X	--	--	--	X	--	--
Bank Sloping and Revegetation	X	X	--	--	--	X	X	--
Channel Relocation	X	X	--	--	--	--	X	--
Vegetation	--	X	--	--	X	X	--	X
Tree Retards and Revetments	--	--	--	X	--	--	--	X
Gabions	--	--	X	--	--	--	--	--
Fencing and Buffer Strips	X	X	X	--	X	X	--	--
Fence Retards	X	--	--	--	--	--	--	--
Jetties and Vegetation	X	X	--	X	--	--	--	--
Grade Control Structures	X	X	--	--	--	--	--	--
Earth Core Dikes	X	X	--	X	--	X	--	--
Selective Clearing	X	X	--	--	--	X	--	X
Revegetation of Riprap	--	X	--	--	--	X	X	X
Construction Scheduling	--	--	--	X	X	--	--	--
Floating Plant Construction	X	X	--	--	--	--	--	--
Stream Corridor Management	X	X	--	--	--	X	X	--

[a] *"X" indicates that the bank protection measure has been or can be used to achieve the environmental goal.*

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

costs which would vary from the generalized data of Appendix B, unit costs were adjusted appropriately.

Figures B-1 through B-8 and Tables B-1 through B-8 in Appendix B represent 1989 construction or operation and maintenance costs based on an Engineering News-Record Construction Cost Index (CCI) of 4,725. When estimating total project costs, the costs obtained from those figures and tables should be adjusted by using the CCI for the year of the estimate and increased by 35 percent to account for engineering, administration, and contingencies. Where applicable, the cost of land acquisition or easements should be added.

The cost data presented in Appendix B were obtained from bid tabulations for other recent flood control and drainage projects within the Village of Menomonee Falls and the Region from studies conducted by the U. S. Army Corps of Engineers and from the Dodge Guide to Public Works and Heavy Construction Costs.[9] Where pre-1989 data were used in the development of cost curves or unit costs, the CCI was used to adjust the costs to 1989.

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[9] *Leonard A. McMahon, author, and Percival E. Pereira, editor, 1982 Dodge Guide to Public Works and Heavy Construction Costs, Annual Edition No. 14, 1981.*

Table 30

SELECTION OF ENVIRONMENTAL MITIGATION FEATURES FOR DIKES AND FLOODWALLS

	Environmental[a]						
Environmental Feature	Improve Fish Habitat	Improve Wetlands Habitat	Improve Uplands Habitat	Reduce Loss of Riparian Vegetation	Improve Water Quality	Improve Recreation	Improve Aesthetics
Avoidance of Sensitive Areas	--	X	X	X	--	--	X
Tree Preservation	--	--	X	X	--	--	X
Alignment to Increase Riverside Land Area	--	X	X	X	--	X	--
Minimizing Cleared Areas	--	X	X	X	--	--	X
Overbuilt Levees	--	--	X	--	--	X	X
Overdesign of Drainage Ditches	--	X	--	X	--	--	X
Erosion Control During Construction	X	X	--	--	X	--	X
Special Borrow Pit Designs	X	X	--	--	--	X	X
Special Designs for Collection Ponds	X	X	--	--	--	X	--
Flushing of Ponds and Wetlands	X	X	--	--	X	--	X
Freshwater Diversions	X	X	--	--	X	--	X
Water Control Structures	X	X	--	--	X	--	--
Artificial Islands	--	X	X	--	--	--	X
Fishery Shelters in Borrow Pits	X	--	--	--	--	X	--
Fish Stocking	X	--	--	--	--	X	--
Marsh Vegetation Establishment	--	X	--	--	--	X	X
Beneficial Uses of Excavated Material	--	X	X	--	--	X	X
Artificial Nesting and Perching Structures	--	X	X	--	--	--	--
Seeding and Plantings for Wildlife	--	X	X	--	--	X	--
Wildlife Brush Piles	--	--	X	--	--	X	--
Controlled Access to Wildlife Areas	--	X	X	--	--	--	--
Special Fencing for Wildlife	--	--	X	--	--	X	--
Roads and Trails	--	--	--	--	--	X	--
Special Vegetative Plantings for Aesthetics	--	--	X	--	--	--	X
Interpretive Centers, Observation Areas, and Culturally Important Sites	--	--	--	--	--	X	X
Boat Ramps and Access, Fishing Access	--	--	--	--	--	X	--
Swimming Beaches	--	--	--	--	--	X	--
Folding or Removable Floodwalls	--	--	--	--	--	X	X
Special Architectural Treatments for Floodwalls, Buildings, and Other Structures	--	--	--	--	--	--	X
Walking Inspections	--	--	X	--	--	--	X
Selective Vegetation Management	--	--	X	--	--	X	X
Modified Mowing, Burning, Grazing, and Chemical Vegetation Control Techniques	--	--	X	X	--	X	X
Irrigation	--	--	X	--	--	--	X

[a] *"X" indicates that the environmental feature has been or can be used to achieve the environmental goal.*

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

Table 31

SUITABILITY OF ENVIRONMENTAL MITIGATION FEATURES FOR VARIOUS PROJECT SETTINGS: MODIFIED CHANNELS

Stream Reach Characteristics	Environmental Features[a]										
	Selective Clearing and Snagging	Low- and Normal-Flow Channels	Diversions and Bypass Channels	Meandering Alignments	Pool and Riffle Grades	Single Bank Modification	Grade Control Structures	Armor	Rigid Linings	Bank Protection	In-Stream Habitat Structures
Channel Pattern											
Braided	--	X	--	X	X	--	--	X	--	--	X
Meandering	--	--	O	O	--	--	--	--	--	--	--
Straight to Sinuous	--	--	--	†	--	--	--	--	--	--	--
Pool Riffle Sequence											
Well Developed	--	--	--	--	O	--	--	--	--	--	--
Not Well Developed	--	--	--	--	†	--	--	--	--	--	--
Sediment Transport											
High Bedload	--	X	--	†	†	--	--	†	--	--	X
High Suspended Load	--	--	--	--	--	--	--	--	--	--	†
Moderate to Low	--	O	--	--	--	--	--	--	--	--	O
Channel Stability											
Stable	--	--	--	--	--	--	--	O	--	--	--
Unstable	--	†	--	†	†	--	--	†	--	--	X
Substrate											
Bedrock	--	--	†	--	--	--	--	X	--	--	†
Gravel	--	--	--	--	--	--	--	O	--	--	O
Sand	--	†	--	--	X	--	--	X	--	--	†
Silt-Clay-Organic	--	--	--	--	--	--	--	†	--	--	--
Bank Cohesiveness											
High	--	--	--	--	--	--	--	--	--	--	--
Low	--	--	--	†	†	--	†	--	--	--	--
Slope											
High (>0.04)	--	--	--	--	--	--	--	--	--	--	X
Moderate (0.002-0.04)	--	--	--	--	--	--	--	--	--	--	O
Low (<0.002)	--	--	--	†	--	--	--	--	--	--	†
Setting											
Urban	--	--	--	--	--	--	--	--	--	--	--
Rural	--	--	--	--	--	--	--	--	--	--	--
Discharge (bankfull)											
Low (<1,000 cfs)	--	--	--	--	--	--	--	--	--	--	O
Medium (1,000-10,000 cfs)	--	--	--	--	--	--	--	--	--	--	†
High (>10,000 cfs)	--	--	--	--	--	--	--	†	--	--	X
Annual Flow Regime											
Ephemeral	--	X	--	--	X	--	--	--	--	--	X
Extreme Variation	--	--	--	--	--	--	--	--	--	--	†
Normal	--	--	--	--	--	--	--	--	--	--	--
Water Quality, Chemical											
Poor	--	X	--	--	--	--	--	--	--	--	X
Fair	--	--	--	--	--	--	--	--	--	--	--
Good	--	--	--	--	--	--	--	--	--	--	--
Fishery											
Little or None	--	--	--	--	--	--	--	--	--	--	X
Cold Water	--	--	--	--	--	--	--	--	--	--	O
Warm Water	--	--	--	--	--	--	--	--	--	--	†

Watershed Characteristics	Environmental Features[a]										
	Selective Clearing and Snagging	Low- and Normal-Flow Channels	Diversions and Bypass Channels	Meandering Alignments	Pool and Riffle Grades	Single Bank Modification	Grade Control Structures	Armor	Rigid Linings	Bank Protection	In-Stream Habitat Structures
Land Use											
Urban	--	--	--	--	--	--	--	--	--	--	†
Agricultural	--	--	--	--	--	--	--	--	--	--	†
Rangeland	--	--	--	--	--	--	--	--	--	--	--
Forested	--	--	--	--	--	--	--	--	--	--	--
Precipitation											
Humid	--	--	--	--	--	--	--	--	--	--	--
Winter Temperature											
Cold (streams freeze over)	--	--	--	--	--	--	--	--	--	--	†
Moderate to Warm	--	--	--	--	--	--	--	--	--	--	--
Terrain											
Hilly	--	--	--	--	--	--	--	--	--	--	O
Flat	--	--	--	--	--	--	--	--	--	--	--
Watershed Equilibrium											
Relatively Undisturbed	--	--	--	--	--	--	--	--	--	--	O
Highly Disturbed	--	†	--	--	†	--	--	†	--	--	†

Table 31 (continued)

Stream Reach Characteristics	Environmental Features[a]										
	Water Level Control Structures	Fishways	Substrate Construction	Oxbow and Bendway Maintenance	Greentree Areas	Vegetative Plantings	Placement and Shaping of Spoil	Sediment Traps	Vegetative Buffer Strips	Revegetation or Disturbed Areas	Special Structures for Stream-Based Recreation
Channel Pattern											
Braided	X	- -	X	X	- -	- -	- -	†	- -	- -	- -
Meandering	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
Straight to Sinuous	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
Pool Riffle Sequence											
Well Developed	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
Not Well Developed	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
Sediment Transport											
High Bedload	X	- -	X	†	- -	- -	- -	O	- -	- -	- -
High Suspended Load	- -	- -	- -	†	- -	- -	- -	- -	- -	- -	- -
Moderate to Low	- -	- -	- -	O	- -	- -	- -	- -	- -	- -	- -
Channel Stability											
Stable	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
Unstable	†	- -	X	- -	- -	- -	- -	- -	- -	- -	- -
Substrate											
Bedrock	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
Gravel	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
Sand	†	- -	X	- -	- -	- -	- -	O	- -	- -	- -
Silt-Clay-Organic	- -	- -	X	- -	- -	- -	- -	- -	- -	- -	- -
Bank Cohesiveness											
High	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
Low	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
Slope											
High (>0.04)	X	- -	- -	- -	X	- -	- -	- -	- -	- -	- -
Moderate (0.002-0.04)	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
Low (<0.002)	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
Setting											
Urban	- -	- -	- -	- -	X	- -	- -	- -	- -	- -	- -
Rural	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
Discharge (bankfull)											
Low (<1,000 cfs)	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	†
Medium (1,000-10,000 cfs)	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
High (>10,000 cfs)	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
Annual Flow Regime											
Ephemeral	X	†	X	†	X	- -	- -	- -	- -	- -	- -
Extreme Variation	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
Normal	- -	- -	- -	O	- -	- -	- -	- -	- -	- -	- -
Water Quality, Chemical											
Poor	†	X	X	†	- -	- -	- -	- -	- -	- -	X
Fair	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
Good	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
Fishery											
Little or None	- -	X	X	- -	- -	- -	- -	- -	- -	- -	- -
Cold Water	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
Warm Water	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -

Watershed Characteristics	Environmental Features[a]										
	Water Level Control Structures	Fishways	Substrate Construction	Oxbow and Bendway Maintenance	Greentree Areas	Vegetative Plantings	Placement and Shaping of Spoil	Sediment Traps	Vegetative Buffer Strips	Revegetation or Disturbed Areas	Special Structures for Stream-Based Recreation
Land Use											
Urban	- -	- -	- -	- -	X	- -	- -	- -	- -	- -	- -
Agricultural	- -	- -	- -	- -	- -	- -	- -	- -	O	- -	- -
Rangeland	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
Forested	- -	- -	- -	- -	O	- -	- -	- -	- -	- -	- -
Precipitation											
Humid	- -	- -	- -	- -	O	O	- -	- -	- -	- -	- -
Winter Temperature											
Cold (streams freeze over)	†	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
Moderate to Warm	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
Terrain											
Hilly	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
Flat	- -	- -	- -	- -	O	- -	- -	- -	- -	- -	- -
Watershed Equilibrium											
Relatively Undisturbed	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
Highly Disturbed	- -	†	- -	- -	- -	- -	- -	- -	- -	- -	- -

[a] *X — Has proven unsuccessful in most cases, likely to be unsuccessful due to physical constraints or inappropriate.*
† — Potential problems likely to be encountered. Successful applications may require special designs or considerable maintenance.
O — Has often proven successful.
- - — Little or no information or conditions not a consideration.

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

Table 32

SUITABILITY OF ENVIRONMENTAL MITIGATION FEATURES FOR VARIOUS PROJECT SETTINGS: DIKES AND FLOODWALLS

	Land Use Type			Levee Material				Levee Size		
Environmental Features[a]	Urban/ Urbanizing	Rural-Forested	Rural-Agricultural	Pervious Earthen	Impervious Earthen	Impervious Core	Floodwall	Standard	Overbuilt	Limitations[b]
Avoidance of Sensitive Areas	†	O	†	O	O	O	O	O	†	1, 2, 4, 5, 6, 8
Tree Preservation	O	O	†	†	O	O	O	O	†	3, 4, 7
Alignment to Increase Riverside Land Area	X	O	†	O	O	O	O	O	O	1, 2, 6, 9
Minimization of Cleared Area	O	O	†	†	O	†	†	O	†	7
Overbuilt Levee	†	O	X	X	O	†	X	- -	- -	3, 5, 7, 9
Overdesigning Drainage Ditches	†	†	†	X	O	O	X	†	O	12
Erosion Control During Construction	O	O	O	O	O	O	O	O	O	- -
Special Designs for Borrow Pits	†	†	†	O	O	X	X	†	O	3, 5, 8, 10, 11, 12, 24
Special Designs for Interior Drainage Ponds	O	X	X	X	†	O	†	†	O	2, 8, 10, 11, 12
Flushing Ponds and Wetlands	O	O	O	O	O	O	X	O	O	1, 10, 11
Freshwater Diversions	X	O	O	X	O	X	X	O	O	5, 8, 10, 13, 14
Water Control Structures	†	O	†	O	O	O	O	O	O	1, 8, 10, 11, 13, 14, 23
Artificial Islands	O	O	O	O	†	†	†	O	O	1, 3, 5, 10, 14, 15, 16, 17
Fishery Shelters in Borrow Pits	O	†	X	O	O	O	O	O	O	11, 12, 15, 17, 18
Fish Stocking	O	X	X	O	O	O	O	O	O	10, 11, 21
Marsh Vegetation Establishment	O	O	O	O	O	X	X	O	O	1, 10, 19, 23, 27
Beneficial Uses for Excavated Material	†	O	†	O	†	X	X	O	O	1, 5, 10, 15, 18
Artificial Nesting and Perching Structures	†	O	†	O	O	O	O	O	O	1, 11, 13, 17
Seeding and Planting for Wildlife	†	O	†	X	†	†	X	†	O	3, 4, 11, 19
Wildlife Brush Piles	X	O	†	O	O	X	X	O	O	3, 17, 22
Controlled Access to Wildlife Areas	†	O	†	O	O	X	X	O	O	6, 8, 23
Special Fencing for Wildlife	X	O	O	O	O	O	X	O	O	6, 7, 17, 22
Uses for Levee Access Roads and Trails	O	†	X	X	O	O	X	†	O	7, 13, 23, 24
Aesthetic Considerations for Plantings	O	X	X	X	O	O	O	†	O	3, 4, 23, 27
Interpretive Centers, Observation Areas, and Culturally Important Sites	O	X	X	X	O	O	O	O	O	8, 9, 13, 23, 24
Boat Ramps and Access, Fishing Access	O	†	X	O	O	O	O	O	†	3, 8, 23, 24, 25
Swimming Beaches	O	X	X	O	†	†	†	O	O	1, 8, 10, 13, 23, 24, 25
Special Architectural Treatments for Floodwalls, Buildings, Etc	O	X	X	X	X	X	O	O	O	1, 7, 27
Walking Inspection	O	O	O	X	O	O	X	X	O	7, 13, 27
Modified Mowing, Burning, Grazing, and Chemical Vegetative Control Techniques	†	†	O	†	†	†	X	O	O	1, 3, 16, 18, 19, 26, 27, 28
Irrigation	O	X	X	O	O	O	X	O	O	3, 13, 23, 17, 28

[a] O — Has often proved successful.
† — Potential problems may be encountered; successful application may require special knowledge, special designs, or considerable maintenance.
X — has proven unsuccessful in most cases; likely to be unsuccessful due to physical constraints or inappropriate.
- - — Little or no information or conditions not a consideration.

[b] Limitation index numbers refer to the following limitations:

1. *Optimum site may not be available.*
2. *May cause more expensive engineering modifications to ensure levee integrity.*
3. *May increase potential for seepage and/or erosion.*
4. *Natural vegetative plant succession and response to changed conditions may make feature hard to implement.*
5. *Adequate borrow is needed.*
6. *Resistance from public is possible.*
7. *Maintenance and access may be hampered or costs increased.*
8. *Existing land use may not be compatible.*
9. *Additional land would be required.*
10. *Amount, quality, and seasonal variations in water resources may limit applicability.*
11. *Wildlife or fish pest problems may develop.*
12. *Improvement may be short-lived.*
13. *Operation and maintenance activities for measure may be excessive.*
14. *Subjective development of management objectives is required.*
15. *Other, more valuable, areas and habitats could be destroyed.*
16. *Separate permit may be required.*
17. *Unwanted debris may be created in flood.*
18. *Water and/or air quality problems could development with implementation.*
19. *Unwanted vegetation types may develop.*
20. *Costs may be excessive.*
21. *Interspecific relations between species may limit usefulness.*
22. *Barriers to wildlife movement may be created.*
23. *Vandalism/illegal entry may occur.*
24. *Public access is needed along with protection of the landowner from liability claims.*
25. *Usable resource should already exist at site.*
26. *Implementation would be the responsibility of the landowner.*
27. *Feature requires unique knowledge by personnel charged with implementing.*
28. *Public health and safety may be adversely affected.*

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

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

Cost estimating data and procedures for nonstructural flood control methods are given in Tables B-9 through B-11. The data were developed from past studies by the Regional Planning Commission and from studies conducted within the Region by the U. S. Army Corps of Engineers. These data represent total 1989 costs and should not be increased for engineering, administration, and contingencies.

For both structural and nonstructural flood control measures and urban nonpoint source pollution control measures, the adopted base cost data are those which are considered most applicable to the types of projects considered for the Lilly Creek stormwater management and flood control plan. The cost data presented in Appendix B and SEWRPC Technical Report No. 31 were used in the economic evaluation of alternative systems plans, and are not intended to be used for project estimating purposes. Actual costs will vary from these estimates, reflecting site-specific conditions, local availability and supply of materials, and labor costs. Any necessary land acquisition costs were estimated utilizing the latest available state equalized assessed valuations.

SUMMARY

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

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

1. The development of a stormwater management system which reduces the exposure of people to drainage-related inconvenience and to health and safety hazards and which reduces the exposure of real and personal property to damage through inadequate stormwater drainage and inundation.
2. The development of an integrated stormwater management and flood control system which will effectively serve existing and planned land uses and will promote implementation of the adopted land use plan.
3. The development of an integrated system of stormwater management and flood control facilities and floodland management programs which will effectively reduce flood damage under the anticipated runoff loadings generated by the existing and proposed land uses.
4. The development of a stormwater management and flood control system which will abate nonpoint source water pollution and help achieve the recommended water use objectives and supporting water quality standards for surface water bodies.
5. The development of a stormwater management system which will be flexible and readily adaptable to changing needs.
6. The development of a stormwater management and flood control system which will efficiently and effectively meet all of the other stated objectives at the lowest practicable cost.
7. The development of a stormwater management and flood control system which will maintain or enhance existing terrestrial, riparian, and aquatic biological communities, including fish and wildlife.

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

In addition to presenting and discussing the objectives and standards established for the Lilly Creek stormwater management and flood control plan, this chapter presents the engineering design criteria and analytic procedures which were used to design and size the alternative plan elements. These criteria can also serve as a basis for the more detailed design of system components. Criteria and procedures were developed and presented for estimating stormwater flow rate and volume and for designing street cross-sections, swales, culverts and channel enclosures, storm sewers, open channels, storage facilities, pumping facilities, bridge and culvert alterations or replacements, dikes and floodwalls, water quality management measures, and environmental mitigation measures.

Chapter V

EVALUATION OF EXISTING AND ALTERNATIVE FUTURE STORMWATER MANAGEMENT AND FLOOD CONTROL SYSTEMS

INTRODUCTION

This chapter presents the findings of an inventory and evaluation of the existing stormwater management and flood control systems serving the Lilly Creek subwatershed and describes and evaluates alternative stormwater management and flood control plans designed to serve that subwatershed under planned ultimate development conditions.

Following this introductory section, the second section of this chapter presents the findings of the inventory and evaluation of the existing stormwater management system in the Lilly Creek subwatershed. As indicated in Chapter IV of this report, a 10-year recurrence interval storm event was used to evaluate the minor system components consisting of backyard and sideyard swales, roadside swales, curbs and gutters, inlets, storm sewers, storage facilities, and related appurtenances. A 100-year recurrence interval storm event was used to evaluate the major system components, including the entire street cross-section and interconnected drainage swales and watercourses.

The third section describes and evaluates alternative conceptual approaches to stormwater management which could be applied in the subwatershed to mitigate existing stormwater management problems and accommodate runoff under planned ultimate development conditions.

The fourth section presents three specific alternative stormwater drainage system plans for the subwatershed. The components of each alternative plan are listed, and capital and operation and maintenance costs are set forth.

The fifth section evaluates the three alternative drainage plans. The impacts of the alternative drainage plans on the prevention of flooding along Lilly Creek and on the control of nonpoint sources pollutants are qualitatively assessed at this stage of the planning process. The alternatives to be considered for inclusion in the recommended stormwater management system plan are selected by hydrologic unit, enabling formulation of a recommended plan which best meets the objectives and supporting standards set forth in Chapter IV of this report.

The sixth section presents and evaluates four specific alternative nonpoint source pollution control plans for the subwatershed. These alternative plan components are described and capital and operation and maintenance costs are provided. The alternative nonpoint source pollution control plans are evaluated within the context of the Menomonee River watershed plan,[1] the regional water quality management plan for southeastern Wisconsin,[2] the Menomonee River watershed nonpoint source control plan recently prepared by the Wisconsin Departments of Natural Resources and Agriculture, Trade and Consumer Protection,[3] and the alternative stormwater drainage plans.

The sixth section also integrates the recommended stormwater drainage and nonpoint source control measures into a preliminary recommended stormwater management plan for the subwatershed. The design of the preliminary recommended plan was based upon careful consideration of many factors, with primary emphasis, however, upon the degree to which the recommended stormwater management objectives and supporting standards are satisfied. Most important among the considerations were those relating to cost, to the ability of the system

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[1] *SEWRPC Planning Report No. 26, A Comprehensive Plan for the Menomonee River Watershed, Vol. 2, Alternative Plans and Recommended Plan, October 1976.*

[2] *SEWRPC Planning Report No. 30, A Regional Water Quality Management Plan for Southeastern Wisconsin: 2000, Vol. 3, Recommended Plan, June 1979.*

[3] *Wisconsin Department of Natural Resources and Wisconsin Department of Agriculture, Trade and Consumer Protection, A Nonpoint Source Control Plan for the Menomonee River Priority Watershed Project, draft, 1990.*

components to accommodate flows resulting from the design storm events without exacerbating downstream drainage and flooding problems, and to the ability of the system components to abate nonpoint source pollution.

The seventh and final section describes and evaluates four alternative flood control plans for the main stem of Lilly Creek.

EVALUATION OF THE EXISTING STORMWATER DRAINAGE AND FLOOD CONTROL SYSTEM

Introduction

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

The evaluation of the existing stormwater drainage system was directed toward the storm sewers, storage facilities, open channels, roadside swales, and culverts of the minor system and toward the open watercourses and related bridges and culverts of the major system. In the evaluation it was assumed that the backyard and sideyard drainage swales and the storm sewer inlets would have adequate capacity to convey the stormwater flows generated by storms up to and including the 10-year recurrence interval event to the receiving conveyance and storage facilities of the minor system. In addition, it was assumed that the street cross-sections and interconnecting drainage swales of the major system would have adequate capacity to convey the stormwater flows generated by storms in excess of the 10-year recurrence interval event and up to the 100-year recurrence interval event to the watercourses of the major system, except at locations such as mid-block sags and streets with extremely flat slopes where the alternatives were specifically designed to handle flows up to those generated by a 100-year event. The system components assumed to be adequate for the purpose of designing and evaluating alternative system plans were, however, subject to quantitative analysis in the development of the recommended plan.

Physical Characteristics

The 5.65-square-mile Lilly Creek subwatershed was divided into 199 catchment areas for analytical purposes, as shown on Map 8. Those catchment areas were aggregated into 16 subbasins with outlets discharging directly to Lilly Creek. The existing stormwater drainage systems are primarily a combination of roadside swales and open channels with associated culverts, roadway curbs and gutters, storm sewer inlets, and storm sewers, together with the streams to which the outlets of the engineered and constructed system components discharge. The existing stormwater drainage systems are described in Chapter II of this report.

Hydraulic Capacities of Conveyance Systems and Comparison with Anticipated Storm Flows

The hydraulic capacity of conveyance facilities, storm sewers, roadside swales, culverts, and open channels, is determined by the shape and dimensions of the cross-section of the facility, by the composition, lining, elevation and gradient of the facility, and by the roughness of the surface as represented by Manning's "n" value. The methods used to determine the hydraulic capacity of the system components are described in Chapter IV of this report. The capacities of storm sewers, storage facilities, open channels and culverts, and selected watercourses were calculated.

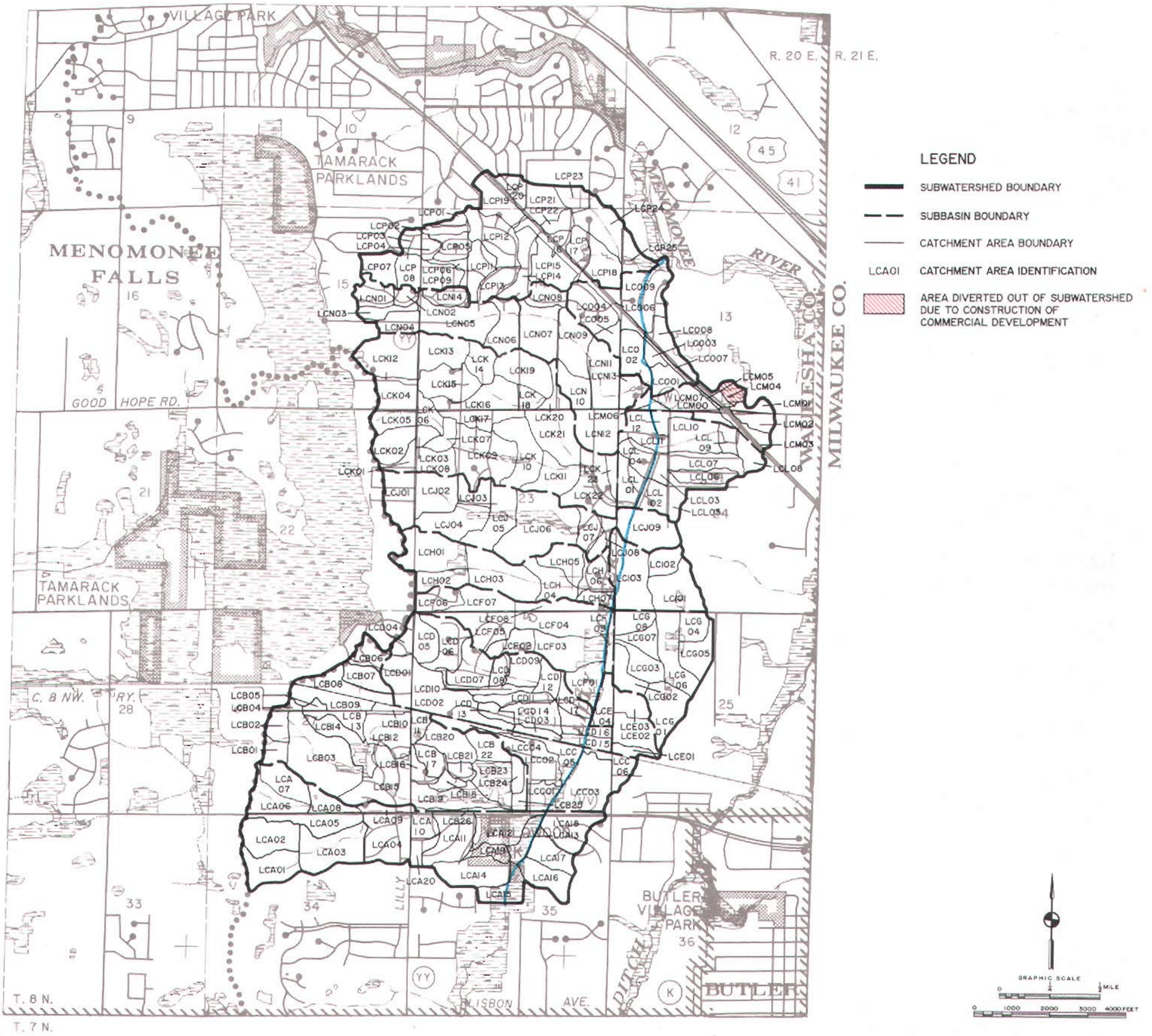
Peak rates of stormwater runoff, as determined by the hydrologic and hydraulic characteristics of each catchment area, were estimated utilizing the methods described in Chapter IV of this report. Peak rates of flow were also estimated for intermediate locations upstream of catchment area outlets in order to determine the hydraulic loadings, as appropriate, on each segment of the storm sewer and drainage channel. Where these stormwater flows exceed the capacities of conveyance facilities, surface ponding, flooding, and surcharging of upstream or downstream drainage facilities may be expected to occur.

Identified Problem Areas

The calculated capacities of each of the components of the existing drainage system were compared to the anticipated stormwater flow rates in order to identify those areas where

Map 8

SUBBASINS WITHIN THE LILLY CREEK WATERSHED



Source: SEWRPC.

problems may be expected under design storm conditions. As already noted, the evaluation considered the capacity of the minor system components in relation to the stormwater flows and volumes generated by a 10-year recurrence interval rainfall event and the capacity of the major system components in relation to the stormwater flows and volumes generated by a

100-year recurrence interval rainfall event. In identifying problems in the existing system, consideration was given to the potential impact of excessive flows. In some cases, problems would not be anticipated, even though the capacity of the system component would be exceeded, for example, in inundated areas that are or would be in open space use and in which no buildings, transportation facilities, or other damage-prone improvements were affected; and in areas where Standard No. 3 of Objective No. 1 as set forth in Chapter IV, relating to acceptable levels of street flooding during a 10-year recurrence interval event, was satisfied.

Map 6 in Chapter II shows the general locations of existing stormwater drainage and flooding problems within the subwatershed as identified by the Village based on historic observations. The hydrologic and hydraulic analyses conducted for this study verified the existence of the most significant problems shown on Map 6 and identified additional system components that have inadequate hydraulic capacity under existing and/or planned land use conditions.

The problems identified through analysis include potential flooding of buildings due to inadequate hydraulic capacity of existing roadside swales, open channels, natural streams, culverts, bridges, and storm sewers. In addition, areas of significant stream bank erosion related to stormwater drainage were identified by the Village and the Wisconsin Department of Natural Resources (DNR), as set forth in Chapter II of this report.

DESCRIPTION AND EVALUATION OF ALTERNATIVE STORMWATER MANAGEMENT APPROACHES

Introduction

As indicated in Chapter II of this report, urban land use within the planning area may be expected to increase significantly between 1985 and the time at which ultimate planned development of the subwatershed is achieved. In the absence of mitigating measures, this urbanization may be expected to produce an increase in the peak rate and volume of stormwater runoff for a given storm event and may also increase the frequency with which damaging floods occur. Stormwater runoff from urban land also contains different types, and, in some cases, increased amounts, of pollutants when compared to stormwater runoff from undeveloped land. Increased urbanization, accordingly, may be expected to place increased demands on the existing stormwater management system, requiring additional engineered facilities to accommodate the increased loadings. The facilities are designed to minimize the occurrence of stormwater management problems and the associated disruption of the urban environment and adverse water quality impacts.

To accommodate these increased loadings and to abate existing, as well as future, stormwater management problems, several stormwater management approaches were considered. These approaches to stormwater management were first evaluated on a conceptual basis, considering the technical feasibility, applicability, and advantages and disadvantages of each approach. Elements of the most feasible approaches were then incorporated into systems-level alternative stormwater management plans for the Lilly Creek subwatershed.

Alternative Stormwater Management Approaches

Alternative approaches to stormwater management that were considered for application in the Lilly Creek area included conventional conveyance, centralized detention, decentralized or onsite detention, centralized retention, decentralized or onsite retention, "blue-green" systems, and nonstructural measures. Pertinent characteristics of each of these alternative approaches are set forth in Table 33. The general feasibility and applicability of each approach were determined on the basis of consideration of these characteristics.

Storm Sewer Conveyance: This conveyance approach would utilize storm sewers and turf-lined, concrete-lined, or composite channels and related appurtenances to provide for the collection and rapid conveyance of stormwater runoff to the receiving streams within the urban service area. The major advantages of this type of system are the minimization of onsite inconvenience because the water is rapidly collected and conveyed downstream, and ready applicability to both existing and newly developing urban areas. Nonpoint source pollution abatement measures appropriate under this approach would be increased street and parking lot sweeping, improved leaf collection, construction site erosion control, pet waste control, onsite infiltration devices, selection of building and construction materials which reduce the runoff contribution of metals and other toxic pollutants, and public education programs. Properly designed, constructed, and maintained storm sewers present no hazard to the public health and safety; help to lower groundwater levels, thereby helping to stabilize pavements and other structures; help to maintain dry basements, thereby minimizing the need for the energy inefficient operation of sump pumps; and minimize the infiltration and inflow of clear water into sanitary sewerage systems. The hydraulic design procedures for storm sewer systems, as well as the construction techniques, are simple, well developed, and commonly used. The disadvantages of the conveyance approach are that downstream peak flows and stages may be increased, leading to a possible increase in areas of inundation and in the potential for streambank erosion, streambed scour, and loss of habitat; additional control measures are required to remove pollutants from the runoff; stream baseflows may be reduced due to the loss of some stormwater infiltration when open channels and grassed swales are replaced with storm sewers; there is little potential for multipurpose uses of the system; and this approach usually has a high capital cost.

Since most of the new development occurring in the Lilly Creek subwatershed relies on storm-sewer conveyance systems, supplemented with dry detention basins, further application of the conveyance approach would represent a continuation of the existing practices and policies for new development. Hence, this approach would probably be understood and accepted by local public officials and citizens alike. Technically, existing stormwater drainage problems, as well as probable future problems, could be most surely and effectively abated using the conveyance approach. In the Lilly Creek subwatershed, existing natural and man-made detention basins located downstream from some areas of planned development would attenuate peak flows from areas served by conveyance systems, thereby reducing the downstream impacts of increased flows. Given the advantages of the conveyance approach, it was considered in the development of the alternative stormwater management plans.

Roadside Swale Conveyance: This conveyance approach would utilize roadside swales and grass-lined or natural channels to provide collection and conveyance of stormwater runoff to receiving streams. The major advantages of this type of system are relatively low capital cost; some reduction in peak flow rates and volumes during more frequent storms in comparison with storm sewer conveyance due to increased flow travel times, in-line storage, and infiltration of runoff through the swale sides and bottom; maintenance of stream baseflow through infiltration of runoff; and a reduction in nonpoint source pollutant loadings due to infiltration and filtering. The disadvantages of the roadside swale conveyance approach include potential safety hazards, relatively high maintenance costs, difficulties in adapting such a system to areas of medium- and high-density development where right-of-way is limited and driveway culverts are closely spaced, and the potential for groundwater contamination, particularly when used in industrial areas.

Because 95 percent of the soils occurring in the subwatershed are classified as poorly or very poorly drained, infiltration of stormwater runoff through the sides and bottom of grassed roadside swales would be limited. Based on hydrologic modeling conducted for this study, roadside swale conveyance would be expected to reduce peak flow rates or volumes by only 10 percent or less during large storms with recurrence intervals ranging from 10 to 100 years. In general, this degree of peak flow reduction would not be sufficient to reduce the size of the conveyance and storage components of the stormwater management system.

At present, there is extensive application of roadside swale conveyance systems within the areas of existing suburban and low-density residential development within the subwatershed. The general policy of the Village is to retain the existing roadside swales in areas of low-density development and, in certain problem areas, to retrofit swales with underlying storm sewers. Use of roadside swale conveyance systems outside areas of low-density development may be resisted by public officials and citizens. Given the potential advantages of the roadside swale conveyance approach, it was considered in the development of the alternative stormwater management plans, particularly in areas of existing or planned suburban and low-density residential development and in areas of planned medium-density residential development.

Centralized Detention: A centralized detention approach would utilize major surface or subsurface detention facilities to provide temporary

Table 33

CHARACTERISTICS OF ALTERNATIVE STORMWATER MANAGEMENT APPROACHES

Characteristic	Conveyance	Centralized Detention	Onsite Detention	Centralized Retention	Onsite Retention	"Blue-Green" System	Nonstructural
Function	Provide for the collection of stormwater runoff and the rapid conveyance of stormwater from the area so as to minimize disruptive and possibly damaging surface ponding in streets and low-lying areas and possible inundation of residential and other sites and structures	Provide for the temporary storage of stormwater runoff in the service area for subsequent slow release to downstream channels or storm sewers, thus minimizing disruption and damage within and downstream of the service area and reducing the required size and therefore cost of any constructed downstream conveyance facilities	Provide for the temporary storage of stormwater runoff at small sites located close to the source of the runoff to be controlled	Provide for the storage of stormwater runoff for subsequent evaporation and infiltration to groundwater, thus removing the area runoff from the surface drainage system and reducing the required size and therefore cost of downstream conveyance facilities	Provide for the storage of stormwater runoff for subsequent evaporation and infiltration to groundwater at small sites located close to the source of generation of the runoff to be retained	Provide for the temporary storage and/or conveyance of stormwater runoff using natural or vegetated channels which slow the runoff rate and allow a portion of the runoff to infiltrate into the soil	Primarily to reduce damages from excessive stormwater runoff and flooding, rather than controlling the runoff rates or flood levels themselves
Components Principal	Improved open drainage channels, storm sewers, and roadside swales	Surface or subsurface detention facilities	Parking lot storage facilities Rooftop storage facilities Relatively small detention facilities Swales, over-sized channels, and diversions	Surface retention facilities Construction site erosion and pet waste control	Relatively small surface retention facilities Subsurface infiltration systems (drywells, etc.)	Open vegetated channels Swales Natural surface depressions and wetlands Over-sized channels Ponds and lakes Construction site erosion and pet waste control	Floodproofing of structures Relocation of structures Land use regulations Open space and floodland preservation Increased street and parking lot sweeping Improved leaf collection Construction site erosion and pet waste control
Secondary	Storm inlets Culverts Outfalls Manholes Increased street and parking lot sweeping Improved leaf collection Construction site erosion and pet waste control	Open drainage channels Storm inlets Culverts Outfalls Manholes Inlet and outlet works and/or pumping facilities Construction site erosion and pet waste control	Same as centralized detention	Open drainage channels Storm inlets Culverts Outfalls Manholes	Same as centralized retention	A "blue-green" system may be supplemented with storm sewers, storm inlets, outfalls, manholes, and culverts	Can be used with other stormwater management facilities
Applicability	Suitable for installation in existing and newly developing urban areas	Most suitable for incorporation in newly developing urban areas if suitable surface or subsurface sites are available	Suitable for installation in existing and newly developing urban areas. May be more suitable than centralized detention in many existing urban areas because of reduced site requirements	Most suitable for incorporation in newly developing urban areas with permeable soils but may be used in existing urban areas if suitable sites are available	Same as centralized retention	Suitable for incorporation in developing urban areas. A "blue-green" system may be undesirable in moderate- or high-density urban development and it may be difficult to develop an economically feasible open channel system which can accommodate the high peak flows from developed urban areas	Suitable for implementation in existing and newly developing urban areas
Downstream Impact Quantity	Tends to significantly increase—relative to predevelopment conditions—downstream discharges, stages, and areas of inundation	May be designed to cause no significant increase, relative to predevelopment conditions, in downstream discharges, stages, and areas of inundation. Decreased discharges, stages, and areas of inundation are possible	Same as centralized detention, although onsite detention facilities are designed for smaller storms and shorter detention times than are centralized detention facilities	Same as centralized detention	Same as onsite detention	May be designed to allow storm runoff to be temporarily stored in a low gradient channel, reducing downstream peak discharge	Minimal impact, although preservation of open space lands may maintain higher levels of natural storage and infiltration than if these lands were developed
Quality	A relatively low level of removal of pollutants from nonpoint sources would be achieved by a storm sewer conveyance system, but significant levels of removal are possible with a roadside swale system	Provides for removal, by the natural settling process, of sediment and other suspended material, thus reducing the pollutant loading on receiving waters. Provides an opportunity for physical-chemical treatment such as disinfection, coagulation-flocculation, and swirl concentration	Provides some pollutant removal, but may be less than by centralized detention if detention time is shorter. Less opportunity for physical-chemical treatment than with centralized facilities	Provides removal of suspended and settleable pollutants but dissolved pollutants may percolate to the water table without reduction	Same as centralized retention	Provides for removal of pollutants in storm runoff by infiltration into the soil, settling of solids, and filtration by vegetation	Minimal impact

Table 33 (continued)

Characteristic	Conveyance	Centralized Detention	Onsite Detention	Centralized Retention	Onsite Retention	"Blue-Green" System	Nonstructural
Multipurpose Capability	Storm sewers serve only a stormwater collection and conveyance function Open drainage channels can provide a focus for development of linear park and open space areas	Quantity control Quality control Can provide park and open space areas	Same as centralized detention	Quantity control Quality control Recreation benefits Aesthetic benefits Groundwater recharge Wildlife habitat	Same as centralized retention	Quantity control Quality control Park and open space areas Aesthetic benefits Wildlife habitat	Park and open space areas
Operation and Maintenance Requirements	Periodic cleaning and repair of storm inlets, channels, and storm sewers required Maintenance of open channel lining material required Increased street and parking lot sweeping Improved leaf collection	Pumping and/or inlet-outlet control operation and maintenance required Insect and odor control may be required Periodic cleaning and maintenance of facility lining required Dam maintenance may be required	Same as centralized detention except that maintenance of onsite facilities may be less intensive but required at a larger number of *sites*	Operation and maintenance required Sediment removal required Insect control may be required Weed and algae control and water pollution control may be required Bank maintenance required	Same as centralized retention except that maintenance of onsite facilities may be less intensive but required at a larger number of sites	Periodic cleaning of channels and inlets required Maintenance of open channel vegetative cover required	Increased street and parking lot sweeping Improved leaf collection
Impact on Sanitary Sewer System	Surcharging of storm sewers accompanied by inundation of streets may result in infiltration of stormwater from storm sewers to adjacent sanitary sewers and inflow of stormwater into sanitary sewers through manholes. Flow in excess of stormwater channel capacity may also result in surface inundation and inflow into sanitary sewers	Runoff volumes in excess of available storage volume, and runoff rates in excess of the capacity of tributary storm sewers and channels, accompanied by inundation of streets may result in infiltration of stormwater from storm sewers to adjacent sanitary sewers and inflow of stormwater into sanitary sewers through manholes	Same as centralized detention	Percolation waters may result in excessive infiltration of stormwater into sanitary sewers	Same as centralized retention	Exceedence of channel capacity accompanied by inundation of streets may result in infiltration of stormwater into adjacent sanitary sewers and inflow of stormwater into sanitary sewers through manholes	Minimal
Hazards	Minimal hazard associated with storm sewers High velocities in roadside swales and improved open channels may pose a safety hazard, particularly to children	Minimal hazard associated with subsurface storage, but surface storage may pose a health and safety hazard, particularly to children	Ponded water in parking lots, small detention facilities, and swales may pose a health and safety hazard, particularly to children, though the size and depth of onsite facilities are frequently minimal	Ponded water may pose a health and safety hazard, particularly to children	Ponded water may pose a health and safety hazard, particularly to children, though the size and depth of onsite facilities are frequently minimal	Flowing channels may pose a health and safety hazard, particularly to children	Minimal
Hydrologic-Hydraulic Analysis	Requires determination only of the peak rate of flow associated with a specified recurrence interval. This is normally obtained with the relatively simple and widely accepted rational method	Requires determination of both a peak rate and a *volume of inflow* associated with a specified recurrence interval, an estimate of allowable outflow rate and storage, and design of pumps or control works to satisfy the discharge conditions. A hydrograph-developing technique must be used to simulate peak flow and volume conditions	Same as centralized detention	Requires determination of both a peak rate and a volume of inflow associated with a specified recurrence interval and estimate of percolation rate and storage to satisfy conditions. A hydrograph-developing technique must be used to simulate peak flow and volume conditions	Same as centralized retention	Requires determination of peak rate of flow, flow volumes, velocity, and flow depths. This can be obtained by using the hydrograph-developing technique	Requires delineation of areas affected by flooding and poor stormwater drainage. The Hydrologic Engineering Center (HEC-2) model may be used to determine flood stages under various recurrence interval storm events
Ability to Meet Stormwater Management Objectives and Supporting Standards	All objectives and supporting standards can be met	All objectives and supporting standards can be met	All objectives and supporting standards can be met	All objectives and supporting standards can be met	All objectives and supporting standards can be met	Some objectives and supporting standards would probably not be met because of the difficulty in accommodating the design flows efficiently and economically using this approach	This alternative would not satisfy the recommended objectives and supporting standards by itself, and must be combined with other alternatives

Source: SEWRPC.

storage of stormwater runoff for subsequent slow release to downstream channels or storm sewers. The centralized detention facilities would be located on a limited number of strategic sites to maximize benefits, yet not all areas would drain to a centralized facility. The centralized detention facilities could be supplemented by improved conveyance facilities as necessary. Nonpoint source pollution control can be provided through the inclusion of a permanent pond within the detention facility and through measures such as construction site erosion control, pet waste control, and selection of building and construction materials to reduce the runoff of metals and other toxic pollutants.

The major advantages of a centralized detention approach are that if properly applied, the facilities can limit the effects of urban development on downstream discharges, areas of inundation, stream bank erosion, streambed scour, and aquatic habitat; a substantial amount of nonpoint source pollutants can be removed; the size and resultant cost of downstream conveyance facilities can be reduced and the need for upgrading existing facilities can sometimes be avoided; the facilities can be combined with recreation and open space areas to provide multipurpose areas; and habitat can be provided for wildlife and waterfowl. The disadvantages of a centralized detention approach are that large, relatively level, open areas are usually required, thereby severely reducing the availability of potential sites in areas of existing development; the facility may not be cost-effective if the site costs cannot be offset by the savings of providing smaller conveyance facilities downstream; the operation and maintenance requirements may be substantial; for a permanent pool facility, the ponded water may be perceived as a public health and safety hazard; and odor and insect problems may be produced. While readily applicable as an integral part of large-scale urban development proposals, the approach is more difficult to apply to areas of existing urban development.

Within the Lilly Creek area, centralized detention facilities could be used to abate some of the existing and potential stormwater management problems. Higher maintenance requirements and an opposition to ponds or dry basins in urban areas by some citizens for aesthetic or health and safety reasons may make this approach unacceptable in some locations. Because of its potential benefits, however, the centralized detention approach was considered in the development of the alternative stormwater management plans.

Onsite Detention: Like centralized detention, onsite detention provides for the temporary storage of stormwater runoff, but the storage sites are located close to, or at, the source of runoff generation. Hence, these detention sites tend to be smaller than centralized detention facilities. Onsite detention measures include small detention basins, parking lot storage, swales, and large channels with gentle slopes. Onsite detention is, in effect, included in all alternative approaches to stormwater management in the Lilly Creek subwatershed, since the Commission recommends the preservation of most of the remaining floodlands, wetlands, and other natural open areas, all of which effectively serve as onsite detention areas. The onsite detention systems, like the centralized detention systems, can also be supplemented by improved conveyance facilities. The nonpoint source control offered by detention can be improved through the inclusion of a permanent pond within the detention basin, along with measures such as construction site erosion control, pet waste control, and selection of building and construction materials which reduce the runoff contribution of metals and other toxic pollutants.

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

The onsite detention approach could be used to abate the existing and potential stormwater runoff problems in the Lilly Creek subwatershed. Although there may be citizen opposition to ponded water in urban areas, the smaller affected sites and greater availability of potential sites may make this approach more acceptable than the centralized approach. Because of its potential benefits, the onsite detention approach was considered in the development of the alternative stormwater management plans. However, because of the difficulty in implementation of this type of storage system on an areawide scale, this option was not considered where more effective and efficient centralized storage sites were available.

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

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

The topography, locally high groundwater levels, and poorly drained soils of the Lilly Creek subwatershed are not favorable for the construction of retention facilities. Therefore, centralized retention facilities were not considered further in the development of the alternative stormwater management plans.

Onsite Retention: Like centralized retention, onsite retention provides for the temporary storage and subsequent infiltration and/or evaporation of stormwater runoff, but the storage sites are located close to, or at, the source of runoff generation. Hence, these sites tend to be smaller than centralized retention facilities. Onsite retention measures include above-ground and subsurface infiltration systems. Nonpoint source control measures appropriate under the onsite retention approach may include various types of infiltration devices, construction site erosion control, pet waste control, and selection of building and construction materials which reduce the runoff contribution of metals and other toxic pollutants.

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

Onsite retention was not considered further in the development of alternative stormwater management plans because of unfavorable topography, soils, and groundwater levels in the subwatershed.

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

The advantages of the "blue-green" approach are that downstream peak flows may be reduced; pollutants in stormwater runoff may be removed by filtration through the soil and vegetation, by biological uptake, and by sedimentation; the "free-form" open channels and related drainage areas can serve as part of park and open space sites following the multi-use concept; habitat areas for wildlife and waterfowl can be maintained or enhanced; construction costs may be lower than those of systems relying more heavily on constructed facilities; and the aesthetic qualities of a "natural" drainage system may be particularly attractive to some citizens. The disadvantages of the "blue-green" approach are that it may make it difficult to develop an open channel system which can effectively accommodate the high peak flows generated from medium- to high-density urban areas served by storm sewers; the flowing channels may be perceived as a safety hazard; the channels are difficult to properly clean and maintain; and some citizens and local public officials may oppose open channel flow in urban areas.

Within the Lilly Creek subwatershed there are "blue-green" system components, including natural channels and wetlands, which could be used to abate stormwater runoff problems. Although there may be some citizen opposition to the short-term standing and flowing water, and to the more extensive land areas required, the maintenance and use of the existing "blue-green" system features were considered in the development of each of the alternative stormwater management plans.

Nonstructural Measures for Stormwater Drainage and Nonpoint Source Pollution Control: The nonstructural approach to stormwater drainage primarily involves reducing damages from unusually high stormwater runoff and inundation rather than controlling the runoff rates or inundation levels themselves. Nonstructural measures include structure floodproofing, relocation of structures, land use regulations, and open space and floodland preservation.

Appropriate nonstructural nonpoint source abatement measures, or source control measures, may include increased street and parking lot sweeping, improved leaf collection and catch basin cleaning, construction site erosion control, pet waste control, restricted use of fertilizers and pesticides, proper disposal of motor vehicle fluids, reduced use of street-deicing salt, and selection of building and construction materials which reduce the runoff contribution of metals and other toxic pollutants.

The nonstructural approach is not in itself an alternative in that in medium- to high-density urban areas stormwater management problems usually cannot be abated by nonstructural measures alone, although the magnitude of these problems may be reduced. Hence, nonstructural measures are usually considered only in combination with the alternative approaches described above.

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

Because of their adaptability and potential for cost savings, nonstructural measures were considered in the development of the alternative stormwater management plans.

ALTERNATIVE STORMWATER DRAINAGE PLANS

Introduction

Utilizing the alternative stormwater management measures described above, the following three alternative stormwater drainage plans were developed for the Lilly Creek subwatershed: 1) storm sewer conveyance with selected open channel conveyance and existing detention storage, 2) open channel conveyance with selected storm sewer conveyance and existing detention storage, and 3) maximum detention storage with a combination of open channel and storm sewer conveyance.

During the alternative plan development and evaluation stage, components of the minor drainage system, such as storm sewers, roadside swales, and detention facilities, were considered, as were such components of the major drainage system as major engineered drainage channels, natural watercourses, and detention facilities. In areas where existing and proposed urban street patterns were established, the alternative plans included a complete system of minor system components. In areas planned to be developed for urban use but for which no street layout, or only a preliminary layout, has been established, only certain key components of the minor system such as trunk storm sewers and roadside swales, important open drainage channels and culverts, and centralized detention facilities could be considered explicitly. Smaller collector storm sewers, culverts, curbs and gutters, and inlets could be considered only implicitly, through the simulation modeling. Nonpoint source pollution abatement measures were carefully considered in the development and evaluation of the alternative drainage system plans. The nonpoint source pollution control alternatives are described in a subsequent section of this chapter and are integrated into the preliminary recommended stormwater management plan. Each alternative stormwater management plan proposes preservation of natural wetlands and floodplains for storage purposes and for integration with conveyance facilities.

In order to compare and evaluate the alternative stormwater management plans, the Lilly Creek subwatershed was divided into 12 hydrologic units. Each unit was composed of one or more subbasins tributary to the same reach of Lilly Creek, to the same conveyance system component, or to a detention facility and its associated downstream conveyance system. A description of individual components and the estimated costs are presented for each hydrologic unit under each alternative plan. The hydrologic unit boundaries are shown on Maps 9, 10, and 11.

Stormwater Drainage Alternative Plan No. 1: Storm Sewer Conveyance with Selected Open Channel Conveyance and Existing Detention Storage

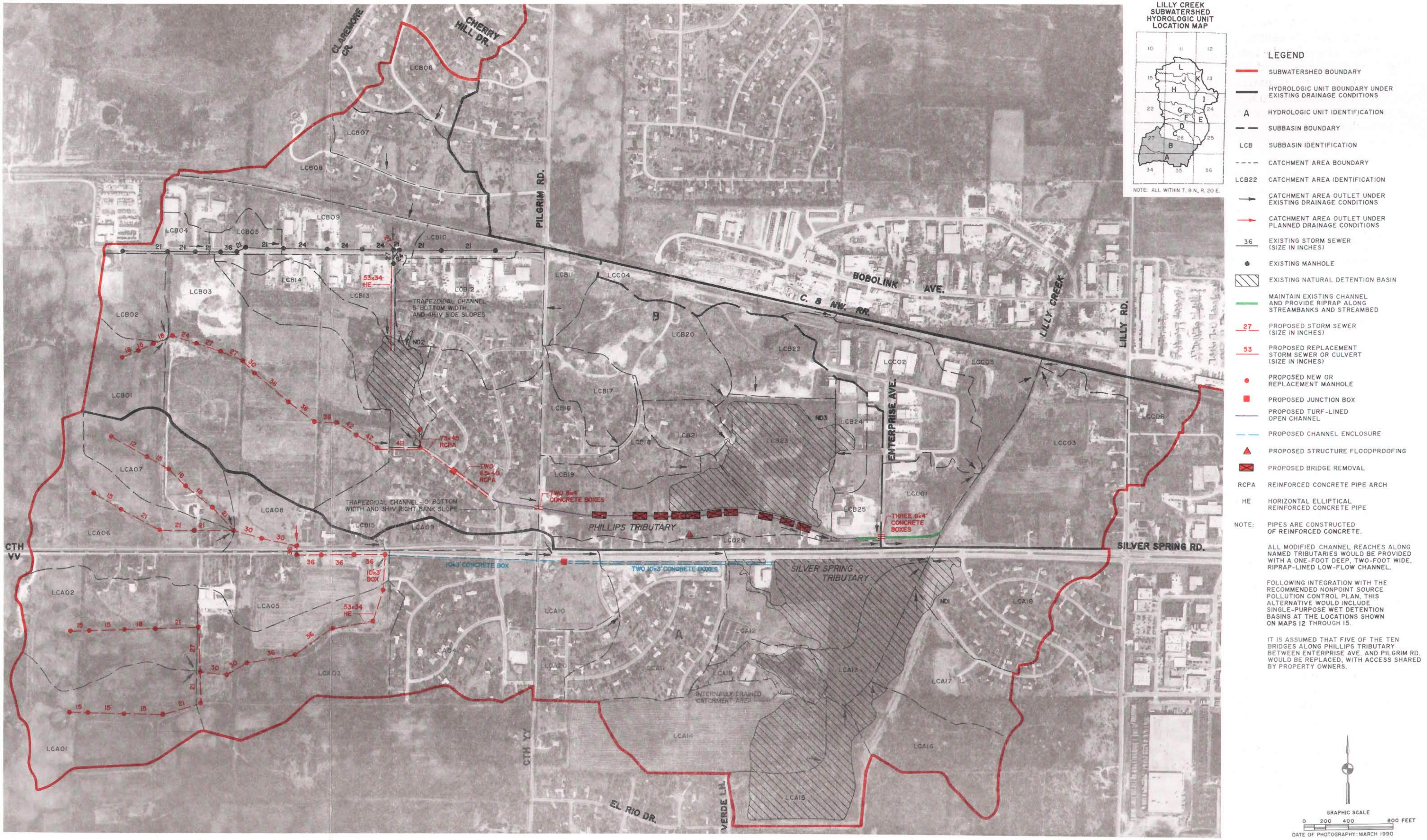
This alternative plan primarily involves the provision of new storm sewers and engineered open channels to abate existing stormwater runoff problems and to effectively serve planned new urban development in the subwatershed. Where possible, the existing stream channels of Lilly Creek and its tributaries are maintained. Map 9 shows the approximate location and alignment of the measures proposed under the alternative. Table 34 presents the salient characteristics and estimated costs of the new storm sewers and channels comprising this alternative plan.

This alternative includes approximately 56,000 lineal feet of new storm sewers in areas of planned development, and 16,300 lineal feet of new storm sewers in areas of existing development which are currently served by open channels, or by open channels in conjunction

Map 9

STORMWATER DRAINAGE ALTERNATIVE PLAN NO. 1: STORM SEWER CONVEYANCE WITH SELECTED OPEN CHANNEL CONVEYANCE AND EXISTING DETENTION STORAGE

HYDROLOGIC UNITS A AND B



NOTE: PIPES ARE CONSTRUCTED OF REINFORCED CONCRETE.

ALL MODIFIED CHANNEL REACHES ALONG NAMED TRIBUTARIES WOULD BE PROVIDED WITH A ONE-FOOT DEEP, TWO-FOOT WIDE, RIPRAP-LINED LOW-FLOW CHANNEL.

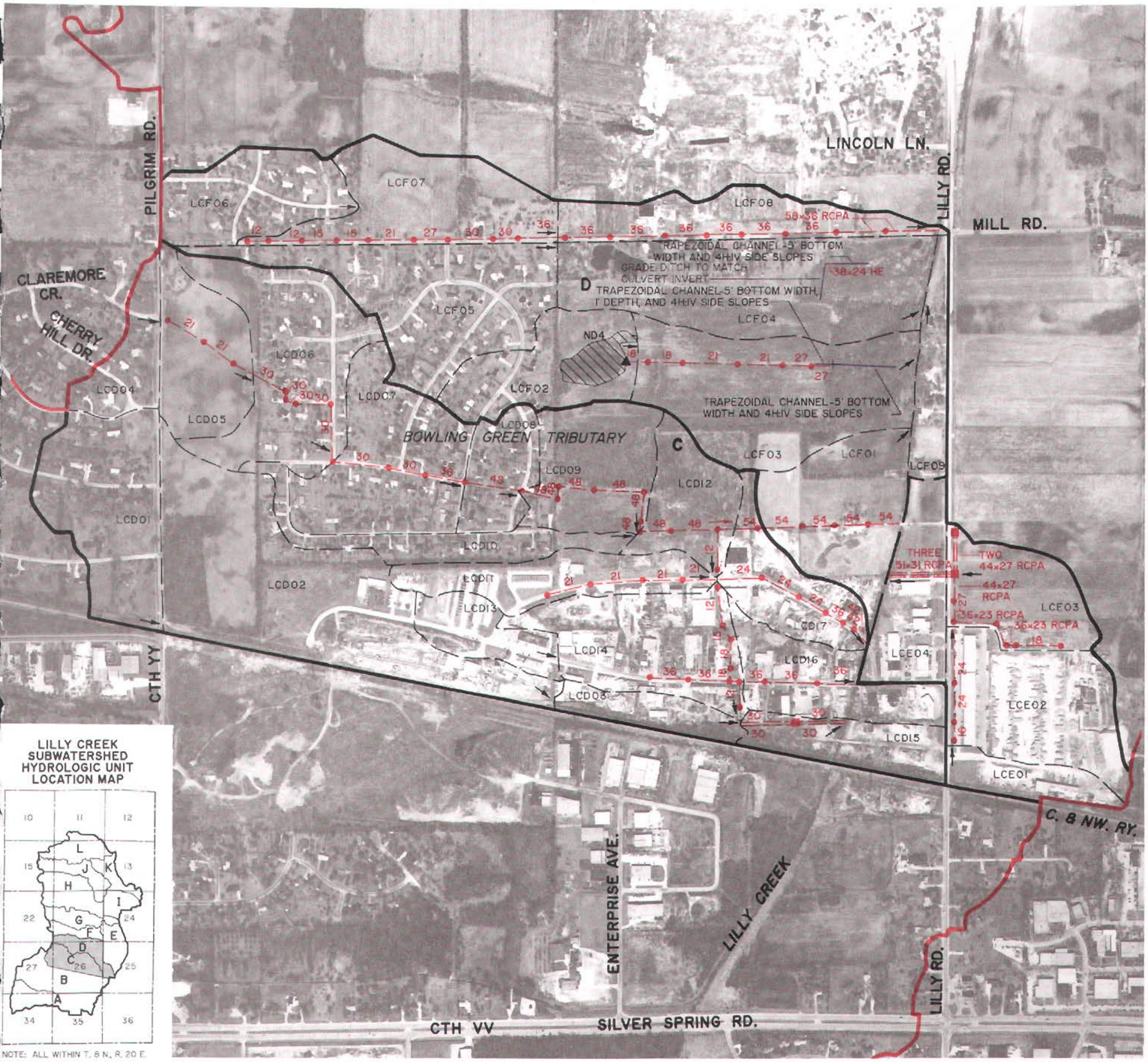
FOLLOWING INTEGRATION WITH THE RECOMMENDED NONPOINT SOURCE POLLUTION CONTROL PLAN, THIS ALTERNATIVE WOULD INCLUDE SINGLE-PURPOSE WET DETENTION BASINS AT THE LOCATIONS SHOWN ON MAPS 12 THROUGH 15.

IT IS ASSUMED THAT FIVE OF THE TEN BRIDGES ALONG PHILLIPS TRIBUTARY BETWEEN ENTERPRISE AVE. AND PILGRIM RD. WOULD BE REPLACED, WITH ACCESS SHARED BY PROPERTY OWNERS.

Map 9 (continued)

STORMWATER DRAINAGE ALTERNATIVE PLAN NO. 1: STORM SEWER CONVEYANCE WITH SELECTED OPEN CHANNEL CONVEYANCE AND EXISTING DETENTION STORAGE

HYDROLOGIC UNITS C AND D



LEGEND

—	SUBWATERSHED BOUNDARY
—	HYDROLOGIC UNIT BOUNDARY UNDER EXISTING DRAINAGE CONDITIONS
D	HYDROLOGIC UNIT IDENTIFICATION
— —	SUBBASIN BOUNDARY
LCF	SUBBASIN IDENTIFICATION
- - - -	CATCHMENT AREA BOUNDARY
LCF03	CATCHMENT AREA IDENTIFICATION
→	CATCHMENT AREA OUTLET UNDER EXISTING DRAINAGE CONDITIONS
→	CATCHMENT AREA OUTLET UNDER PLANNED DRAINAGE CONDITIONS
▨	EXISTING NATURAL DETENTION BASIN
30	PROPOSED STORM SEWER (SIZE IN INCHES)
30	PROPOSED REPLACEMENT STORM SEWER OR CULVERT (SIZE IN INCHES)
●	PROPOSED NEW OR REPLACEMENT MANHOLE
■	PROPOSED JUNCTION BOX
—	PROPOSED TURF-LINED OPEN CHANNEL
▲	PROPOSED OUTLET STRUCTURE
RCPA	REINFORCED CONCRETE PIPE ARCH
HE	HORIZONTAL ELLIPTICAL REINFORCED CONCRETE PIPE
NOTE:	PIPES ARE CONSTRUCTED OF REINFORCED CONCRETE.

ALL MODIFIED CHANNEL REACHES ALONG NAMED TRIBUTARIES WOULD BE PROVIDED WITH A ONE-FOOT DEEP, TWO-FOOT WIDE, RIPRAP-LINED LOW-FLOW CHANNEL.

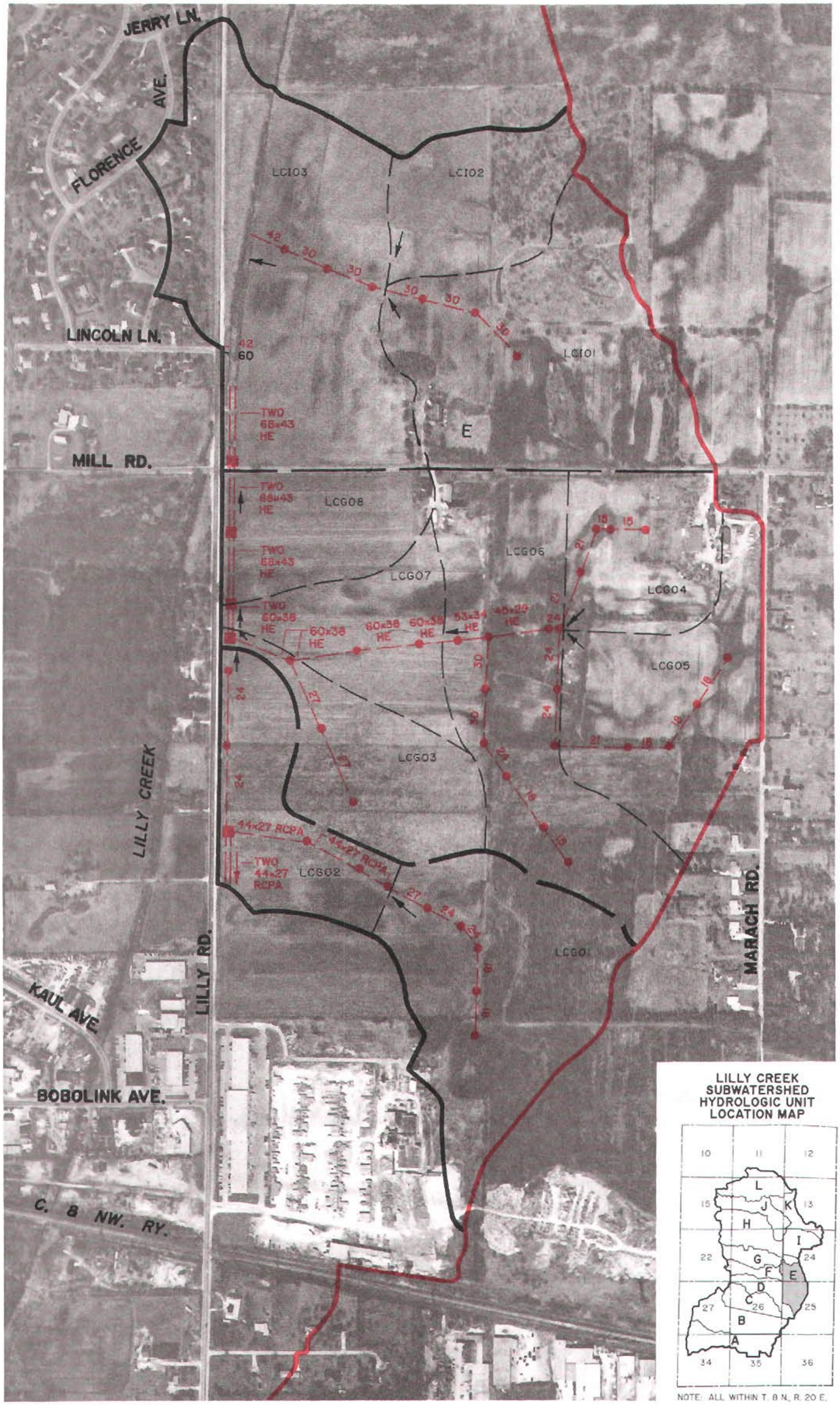
FOLLOWING INTEGRATION WITH THE RECOMMENDED NONPOINT SOURCE POLLUTION CONTROL PLAN, THIS ALTERNATIVE WOULD INCLUDE SINGLE-PURPOSE WET DETENTION BASINS AT LOCATIONS SHOWN ON MAPS 12 THROUGH 15.

GRAPHIC SCALE
0 200 400 800 FEET
DATE OF PHOTOGRAPHY: MARCH 1990

Map 9 (continued)

STORMWATER DRAINAGE ALTERNATIVE PLAN NO. 1: STORM SEWER CONVEYANCE WITH SELECTED OPEN CHANNEL CONVEYANCE AND EXISTING DETENTION STORAGE

HYDROLOGIC UNIT E

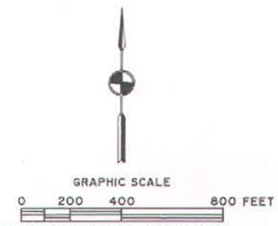


LEGEND

▬	SUBWATERSHED BOUNDARY
▬	HYDROLOGIC UNIT BOUNDARY UNDER EXISTING DRAINAGE CONDITIONS
▬ ▬	HYDROLOGIC UNIT BOUNDARY UNDER PLANNED DRAINAGE CONDITIONS
E	HYDROLOGIC UNIT IDENTIFICATION
— —	SUBBASIN BOUNDARY
LCG	SUBBASIN IDENTIFICATION
- - - -	CATCHMENT AREA BOUNDARY
LCG07	CATCHMENT AREA IDENTIFICATION
→	CATCHMENT AREA OUTLET UNDER EXISTING DRAINAGE CONDITIONS
→	CATCHMENT AREA OUTLET UNDER PLANNED DRAINAGE CONDITIONS
60	EXISTING STORM SEWER (SIZE IN INCHES)
●	EXISTING MANHOLE
42	PROPOSED STORM SEWER (SIZE IN INCHES)
●	PROPOSED NEW OR REPLACEMENT MANHOLE
■	PROPOSED JUNCTION BOX
RCPA	REINFORCED CONCRETE PIPE ARCH
HE	HORIZONTAL ELLIPTICAL REINFORCED CONCRETE PIPE

NOTE: PIPES ARE CONSTRUCTED OF REINFORCED CONCRETE

FOLLOWING INTEGRATION WITH THE RECOMMENDED NONPOINT SOURCE POLLUTION CONTROL PLAN, THIS ALTERNATIVE WOULD INCLUDE SINGLE-PURPOSE WET DETENTION BASINS AT THE LOCATIONS SHOWN ON MAPS 12 THROUGH 15.

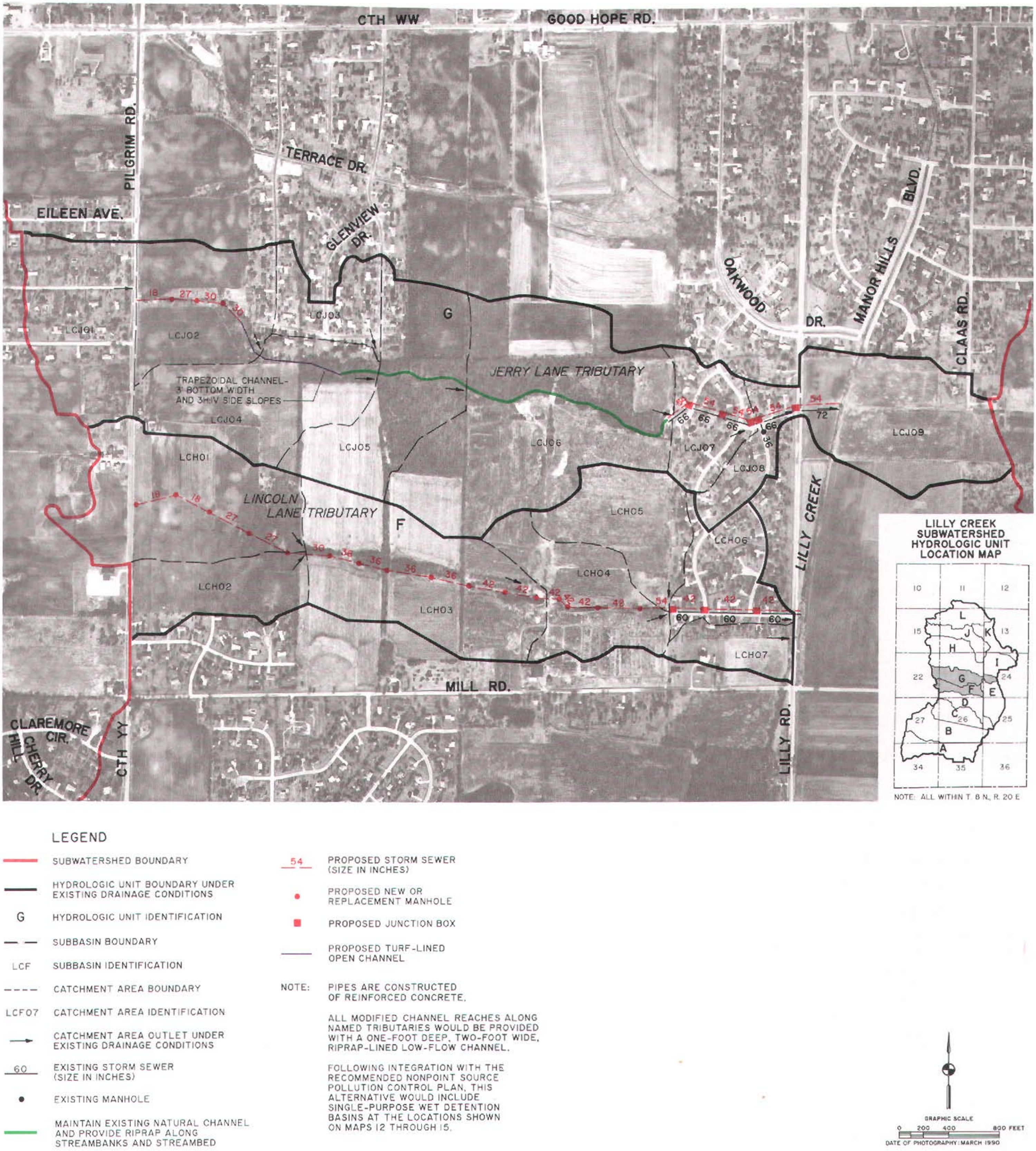


DATE OF PHOTOGRAPHY: MARCH 1990

Map 9 (continued)

STORMWATER DRAINAGE ALTERNATIVE PLAN NO. 1: STORM SEWER CONVEYANCE WITH SELECTED OPEN CHANNEL CONVEYANCE AND EXISTING DETENTION STORAGE

HYDROLOGIC UNITS F AND G



LEGEND

🟥	SUBWATERSHED BOUNDARY
▬	HYDROLOGIC UNIT BOUNDARY UNDER EXISTING DRAINAGE CONDITIONS
G	HYDROLOGIC UNIT IDENTIFICATION
— · —	SUBBASIN BOUNDARY
LCF	SUBBASIN IDENTIFICATION
- - - -	CATCHMENT AREA BOUNDARY
LCF07	CATCHMENT AREA IDENTIFICATION
→	CATCHMENT AREA OUTLET UNDER EXISTING DRAINAGE CONDITIONS
60	EXISTING STORM SEWER (SIZE IN INCHES)
●	EXISTING MANHOLE
🟩	MAINTAIN EXISTING NATURAL CHANNEL AND PROVIDE RIPRAP ALONG STREAMBANKS AND STREAMBED
54	PROPOSED STORM SEWER (SIZE IN INCHES)
●	PROPOSED NEW OR REPLACEMENT MANHOLE
■	PROPOSED JUNCTION BOX
—	PROPOSED TURF-LINED OPEN CHANNEL

NOTE: PIPES ARE CONSTRUCTED OF REINFORCED CONCRETE.

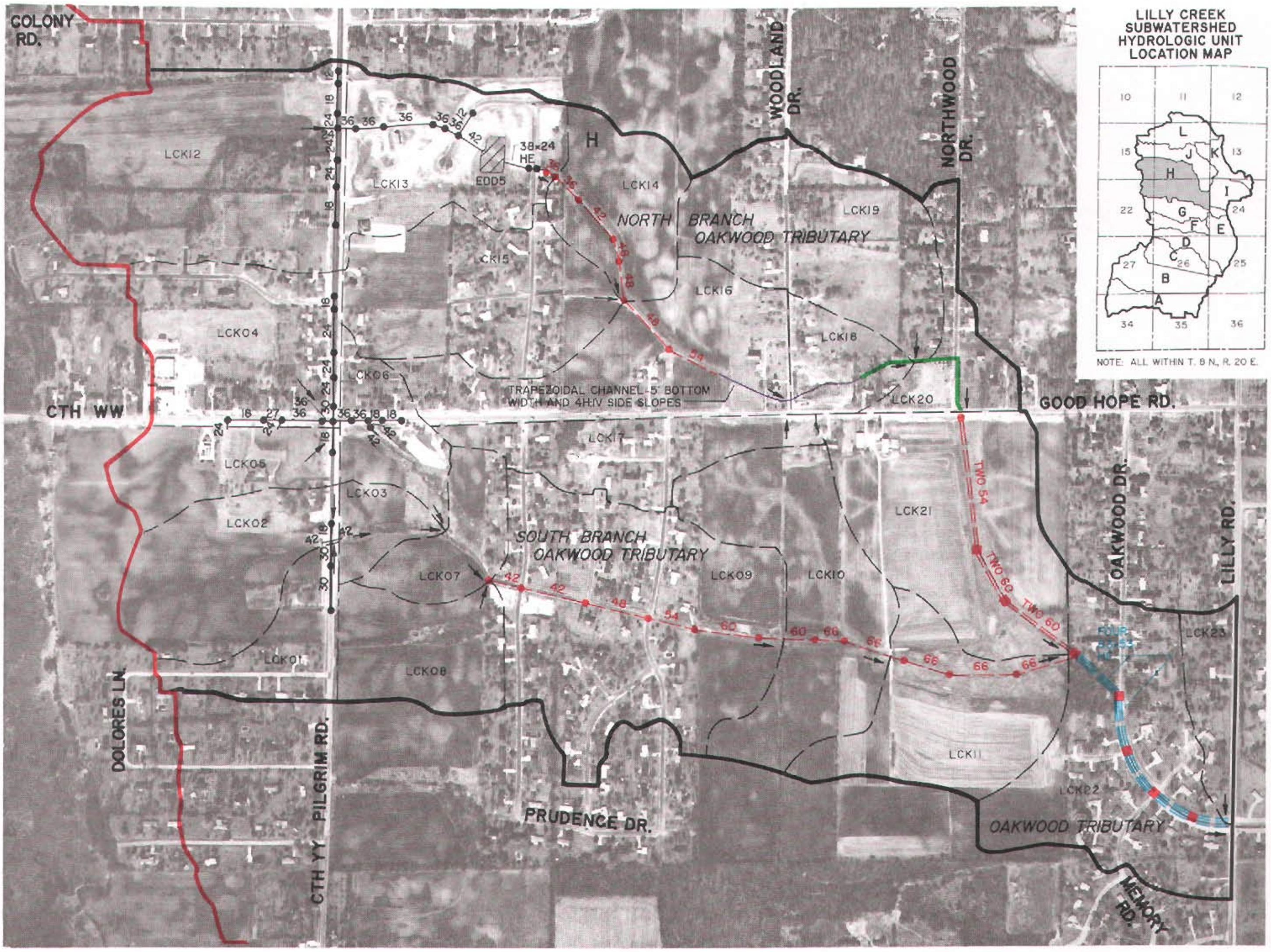
ALL MODIFIED CHANNEL REACHES ALONG NAMED TRIBUTARIES WOULD BE PROVIDED WITH A ONE-FOOT DEEP, TWO-FOOT WIDE, RIPRAP-LINED LOW-FLOW CHANNEL.

FOLLOWING INTEGRATION WITH THE RECOMMENDED NONPOINT SOURCE POLLUTION CONTROL PLAN, THIS ALTERNATIVE WOULD INCLUDE SINGLE-PURPOSE WET DETENTION BASINS AT THE LOCATIONS SHOWN ON MAPS 12 THROUGH 15.

Map 9 (continued)

STORMWATER DRAINAGE ALTERNATIVE PLAN NO. 1: STORM SEWER CONVEYANCE WITH SELECTED OPEN CHANNEL CONVEYANCE AND EXISTING DETENTION STORAGE

HYDROLOGIC UNIT H



LEGEND

🟥	SUBWATERSHED BOUNDARY
▬	HYDROLOGIC UNIT BOUNDARY UNDER EXISTING DRAINAGE CONDITIONS
H	HYDROLOGIC UNIT IDENTIFICATION
— - —	SUBBASIN BOUNDARY
LCK	SUBBASIN IDENTIFICATION
- - - -	CATCHMENT AREA BOUNDARY
LCK16	CATCHMENT AREA IDENTIFICATION
→	CATCHMENT AREA OUTLET UNDER EXISTING DRAINAGE CONDITIONS
42	EXISTING STORM SEWER (SIZE IN INCHES)
●	EXISTING MANHOLE
▨	EXISTING MAN-MADE DRY DETENTION BASIN
🟩	MAINTAIN EXISTING NATURAL CHANNEL AND PROVIDE RIPRAP ALONG STREAMBANKS AND STREAMBED
36	PROPOSED STORM SEWER (SIZE IN INCHES)
●	PROPOSED NEW OR REPLACEMENT MANHOLE
■	PROPOSED JUNCTION BOX
—	PROPOSED TURF-LINED OPEN CHANNEL
— —	PROPOSED CHANNEL ENCLOSURE
HE	HORIZONTAL ELLIPTICAL REINFORCED CONCRETE PIPE

NOTE: PIPES ARE CONSTRUCTED OF REINFORCED CONCRETE.

ALL MODIFIED CHANNEL REACHES ALONG NAMED TRIBUTARIES WOULD BE PROVIDED WITH A ONE-FOOT DEEP, TWO-FOOT WIDE, RIPRAP-LINED LOW-FLOW CHANNEL.

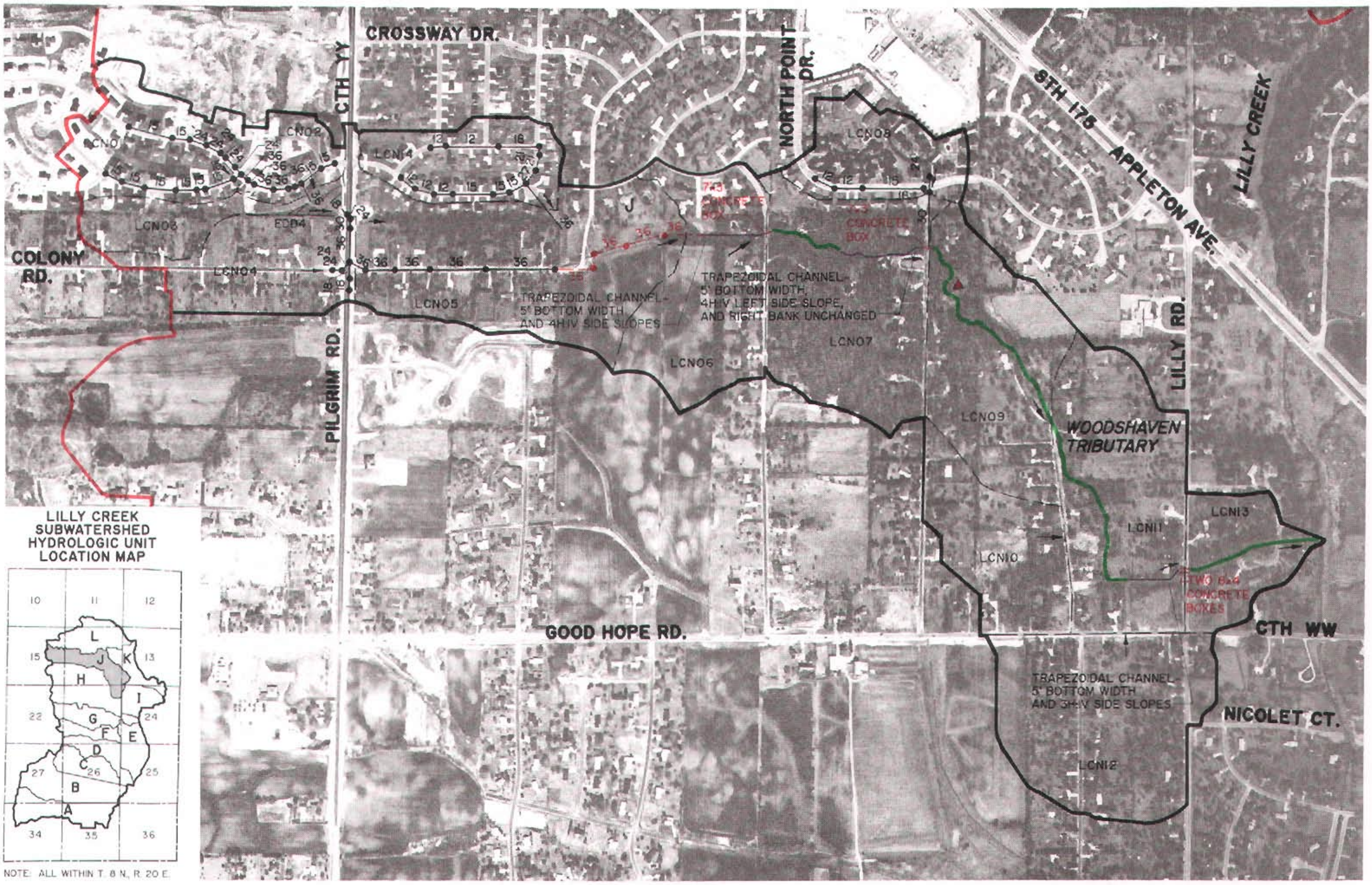
FOLLOWING INTEGRATION WITH THE RECOMMENDED NONPOINT SOURCE POLLUTION CONTROL PLAN, THIS ALTERNATIVE WOULD INCLUDE SINGLE-PURPOSE WET DETENTION BASINS AT THE LOCATIONS SHOWN ON MAPS 12 THROUGH 15.

GRAPHIC SCALE
0 200 400 800 FEET
DATE OF PHOTOGRAPHY: MARCH 1990

Map 9 (continued)

STORMWATER DRAINAGE ALTERNATIVE PLAN NO. 1: STORM SEWER CONVEYANCE WITH SELECTED OPEN CHANNEL CONVEYANCE AND EXISTING DETENTION STORAGE

HYDROLOGIC UNIT J



LEGEND

- SUBWATERSHED BOUNDARY
- HYDROLOGIC UNIT BOUNDARY UNDER EXISTING DRAINAGE CONDITIONS
- J HYDROLOGIC UNIT IDENTIFICATION
- SUBBASIN BOUNDARY
- LCN SUBBASIN IDENTIFICATION
- CATCHMENT AREA BOUNDARY
- LCN09 CATCHMENT AREA IDENTIFICATION
- CATCHMENT AREA OUTLET UNDER EXISTING DRAINAGE CONDITIONS
- 42 EXISTING STORM SEWER (SIZE IN INCHES)
- ● EXISTING MANHOLE
- ▨ EXISTING MAN-MADE DRY DETENTION BASIN
- MAINTAIN EXISTING CHANNEL AND PROVIDE RIPRAP ALONG STREAMBANKS AND STREAMBED
- 36 PROPOSED STORM SEWER (SIZE IN INCHES)
- PROPOSED REPLACEMENT CULVERT
- ● PROPOSED NEW OR REPLACEMENT MANHOLE
- PROPOSED TURF-LINED OPEN CHANNEL
- ▲ PROPOSED STRUCTURE FLOODPROOFING

NOTE: PIPES ARE CONSTRUCTED OF REINFORCED CONCRETE.

ALL MODIFIED CHANNEL REACHES ALONG NAMED TRIBUTARIES WOULD BE PROVIDED WITH A ONE-FOOT DEEP, TWO-FOOT WIDE, RIPRAP-LINED LOW-FLOW CHANNEL.

FOLLOWING INTEGRATION WITH THE RECOMMENDED NONPOINT SOURCE POLLUTION CONTROL PLAN, THIS ALTERNATIVE WOULD INCLUDE SINGLE-PURPOSE WET DETENTION BASINS AT THE LOCATIONS SHOWN ON MAPS 12 THROUGH 15.

GRAPHIC SCALE
0 200 400 800 FEET
DATE OF PHOTOGRAPHY: MARCH 1990

Map 9 (continued)

STORMWATER DRAINAGE ALTERNATIVE PLAN NO. 1: STORM SEWER CONVEYANCE WITH SELECTED OPEN CHANNEL CONVEYANCE AND EXISTING DETENTION STORAGE

HYDROLOGIC UNITS I AND K

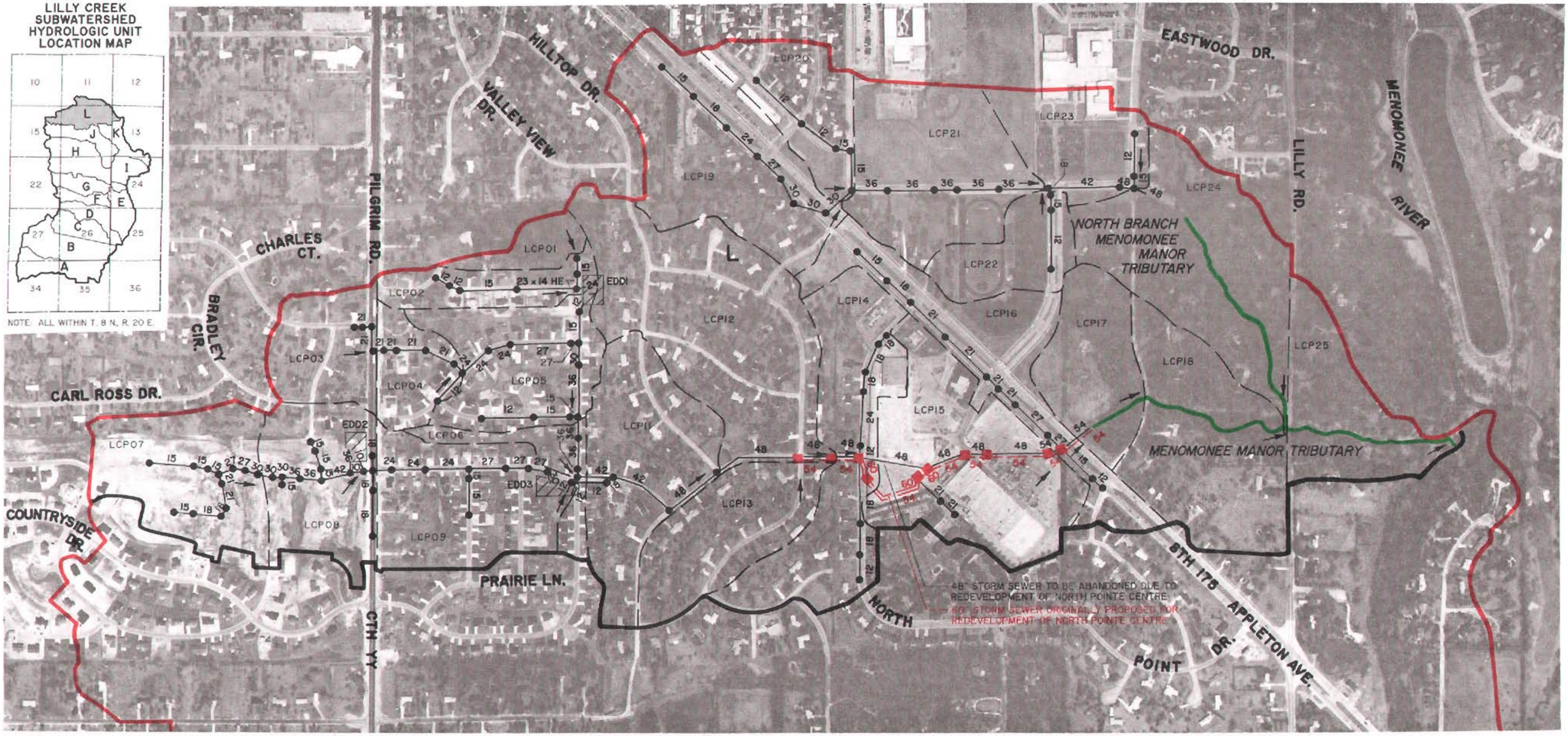
NOTE: PIPES ARE CONSTRUCTED OF REINFORCED CONCRETE UNLESS DESIGNATED AS ABOVE.

FOLLOWING INTEGRATION WITH THE RECOMMENDED NONPOINT SOURCE POLLUTION CONTROL PLAN, THIS ALTERNATIVE WOULD INCLUDE SINGLE-PURPOSE WET DETENTION BASINS AT THE LOCATIONS SHOWN ON MAPS 12 THROUGH 15.

Map 9 (continued)

STORMWATER DRAINAGE ALTERNATIVE PLAN NO. 1: STORM SEWER CONVEYANCE WITH SELECTED OPEN CHANNEL CONVEYANCE AND EXISTING DETENTION STORAGE

HYDROLOGIC UNIT L



LEGEND

🟥	SUBWATERSHED BOUNDARY	21	EXISTING STORM SEWER (SIZE IN INCHES)	🟥	PROPOSED JUNCTION BOX		
▬	HYDROLOGIC UNIT BOUNDARY UNDER EXISTING DRAINAGE CONDITIONS	●	EXISTING MANHOLE	HE	HORIZONTAL ELLIPTICAL REINFORCED CONCRETE PIPE		
L	HYDROLOGIC UNIT IDENTIFICATION	▨	EXISTING MAN-MADE DRY DETENTION BASIN	NOTE:	PIPES ARE CONSTRUCTED OF REINFORCED CONCRETE.		
— —	SUBBASIN BOUNDARY	🟩	MAINTAIN EXISTING CHANNEL AND PROVIDE RIPRAP ALONG STREAMBANKS AND STREAMBED		FOLLOWING INTEGRATION WITH THE RECOMMENDED NONPOINT SOURCE POLLUTION CONTROL PLAN, THIS ALTERNATIVE WOULD INCLUDE SINGLE-PURPOSE WET DETENTION BASINS AT THE LOCATIONS SHOWN ON MAPS 12 THROUGH 15.		
LCP	SUBBASIN IDENTIFICATION		PROPOSED STORM SEWER (SIZE IN INCHES)				
----	CATCHMENT AREA BOUNDARY		PROPOSED NEW OR REPLACEMENT MANHOLE				
LCP22	CATCHMENT AREA IDENTIFICATION						
→	CATCHMENT AREA OUTLET UNDER EXISTING DRAINAGE CONDITIONS						

GRAPHIC SCALE
0 200 400 800 FEET
DATE OF PHOTOGRAPHY: MARCH 1990

Source: SEWRPC.

Table 34

STORMWATER DRAINAGE ALTERNATIVE PLAN NO. 1: COMPONENTS AND COSTS OF THE STORM SEWER CONVEYANCE WITH SELECTED OPEN CHANNEL CONVEYANCE AND EXISTING DETENTION STORAGE

Hydrologic Unit	Project and Component Description[a]	Estimated Cost	
		Capital[b]	Annual Operation and Maintenance[c]
A	Silver Spring Tributary		
	1. 380 feet of 12-inch storm sewer	$ 16,000	$ 200
	2. 1,790 feet of 15-inch storm sewer	82,000	700
	3. 825 feet of 18-inch storm sewer	42,000	300
	4. 2,280 feet of 21-inch storm sewer	132,000	900
	5. 380 feet of 27-inch storm sewer	26,000	200
	6. 1,010 feet of 30-inch storm sewer	78,000	400
	7. 1,730 feet of 36-inch storm sewer	161,000	300
	8. 3,720 feet total of twin 10-foot x 3-foot concrete box culvert	1,858,000	700
	9. 1,830 feet of 10-foot x 3-foot concrete box culvert	914,000	300
	10. 680 feet of 53-inch x 34-inch concrete HE storm sewer	92,000	100
	Subtotal	$ 3,401,000	$ 4,100
B	Phillips Tributary		
	1. 510 feet of 18-inch storm sewer	$ 26,000	$ 200
	2. 230 feet of 24-inch storm sewer	14,000	100
	3. 460 feet of 27-inch storm sewer	32,000	200
	4. 150 feet of 30-inch storm sewer	12,000	100
	5. 860 feet of 36-inch storm sewer	80,000	200
	6. 830 feet of 42-inch storm sewer	93,000	200
	7. Replace existing culverts at Enterprise Drive with three 43-foot-long, 8-foot x 4-foot concrete box culverts	71,000	0
	8. Replace existing culverts at Pilgrim Road with two 40-foot-long, 8-foot x 4-foot concrete box culverts	44,000	0
	9. Remove 10 private pedestrian bridges; replace five bridges	107,000	0
	10. 765 feet of 53-inch x 34-inch concrete HE storm sewer	103,000	100
	11. 760 feet total of double 65-inch x 40-inch RCPA storm sewer	145,000	100
	12. 550 feet of 73-inch x 45-inch RCPA storm sewer	127,000	100
	13. Construct 3,300-foot-long trapezoidal, turf-lined channel with a 10-foot bottom width, a 3H:1V right bank slope, and a 1-foot-deep, 2-foot-wide, riprap-lined low-flow channel	172,000	1,400
	14. Construct 765-foot-long trapezoidal, turf-lined channel with a 5-foot bottom width and 4H:1V side slopes	12,000	300
	15. Floodproof one house	5,000	- -
	16. Riprap along 0.14 mile of existing stream	27,000	300
	Subtotal	$ 1,070,000	$ 3,300
C	Bowling Green Tributary		
	1. 855 feet of 12-inch storm sewer	$ 36,000	$ 300
	2. 110 feet of 15-inch storm sewer	5,000	0
	3. 290 feet of 18-inch storm sewer	15,000	100
	4. 1,780 feet of 21-inch storm sewer	116,000	700
	5. 795 feet of 24-inch storm sewer	58,000	300
	6. 3,185 feet of 30-inch storm sewer	261,000	1,300
	7. 1,775 feet of 36-inch storm sewer	186,000	300
	8. 185 feet of 42-inch storm sewer	23,000	0
	9. 2,155 feet of 48-inch storm sewer	296,000	400
	10. 1,265 feet of 54-inch storm sewer	196,000	200
	Subtotal	$ 1,192,000	$ 3,600

Table 34 (continued)

Hydrologic Unit	Project and Component Description[a]	Estimated Cost	
		Capital[b]	Annual Operation and Maintenance[c]
D	Area Predominately West of Lilly Creek and North and South of W. Mill Road		
	1. 440 feet of 12-inch storm sewer	$ 22,000	$ 200
	2. 430 feet of 15-inch storm sewer	23,000	200
	3. 1,255 feet of 18-inch storm sewer	64,000	500
	4. 1,005 feet of 21-inch storm sewer	62,000	400
	5. 1,780 feet of 24-inch storm sewer	111,000	700
	6. 885 feet of 27-inch storm sewer	63,000	400
	7. 475 feet of 30-inch storm sewer	42,000	200
	8. 2,170 feet of 36-inch storm sewer	229,000	400
	9. 540 feet of 36-inch x 23-inch RCPA storm sewer	52,000	200
	10. 2,215 feet of 44-inch x 27-inch RCPA storm sewer	239,000	400
	11. 1,305 feet total of 51-inch x 31-inch RCPA storm sewer	176,000	200
	12. 730 feet of 58-inch x 36-inch RCPA storm sewer	131,000	100
	13. 60 feet of 38-inch x 24-inch concrete HE storm sewer	6,000	0
	14. 135-foot-long trapezoidal, turf-lined channel with a 5-foot bottom width, 4H:1V side slopes and 1-foot depth	2,000	100
	15. 440-foot-long trapezoidal, turf-lined channel with 5-foot bottom width, 4H:1V side slopes and 2-foot average depth	7,000	200
	16. 250-foot-long trapezoidal, turf-lined channel with 5-foot bottom width, 4H:1V side slopes and 1.25-foot average depth	3,000	100
	Subtotal	$ 1,232,000	$ 4,300
E	Area East of Lilly Creek and North and South of W. Mill Road		
	1. 420 feet of 15-inch storm sewer	$ 19,000	$ 200
	2. 950 feet of 18-inch storm sewer	49,000	400
	3. 860 feet of 21-inch storm sewer	50,000	300
	4. 835 feet of 24-inch storm sewer	52,000	300
	5. 750 feet of 27-inch storm sewer	52,000	300
	6. 1,805 feet of 30-inch storm sewer	139,000	700
	7. 200 feet of 42-inch storm sewer	22,000	0
	8. 330 feet of 45-inch x 29-inch HE storm sewer	36,000	100
	9. 160 feet of 53-inch x 34-inch concrete HE storm sewer	22,000	0
	10. 1,595 feet of 60-inch x 38-inch concrete HE storm sewer	265,000	300
	11. 2,150 feet total of twin 68-inch x 43-inch concrete HE storm sewer	409,000	400
	Subtotal	$ 1,115,000	$ 3,000
F	Lincoln Lane Tributary		
	1. 655 feet of 18-inch storm sewer	$ 34,000	$ 300
	2. 710 feet of 27-inch storm sewer	49,000	300
	3. 330 feet of 30-inch storm sewer	25,000	100
	4. 1,120 feet of 36-inch storm sewer	104,000	200
	5. 2,445 feet of 42-inch storm sewer	288,000	500
	6. 315 feet of 54-inch storm sewer	49,000	100
	Subtotal	$ 549,000	$ 1,500
G	Jerry Lane Tributary		
	1. 290 feet of 18-inch storm sewer	$15,000	$100
	2. 200 feet of 27-inch storm sewer	14,000	100
	3. 410 feet of 30-inch storm sewer	32,000	200
	4. 1,375 feet of 54-inch relief sewer parallel to existing storm sewer	236,000	300
	5. Retain existing open channel through isolated natural area	- -	- -
	6. 920-foot-long trapezoidal, turf-lined channel with 3-foot bottom width, 3H:1V side slopes, and a 1-foot deep, 2-foot wide riprap-lined low-flow channel	43,000	400
	7. Riprap along 0.53 mile of existing stream	100,000	1,100
	Subtotal	$440,000	$2,200

Table 34 (continued)

Hydrologic Unit	Project and Component Description[a]	Estimated Cost	
		Capital[b]	Annual Operation and Maintenance[c]
H	Oakwood Tributary		
	Main Stem		
	1. 7,960 feet total of four 83 x 53-inch concrete HE pipe	$ 2,020,000	$ 1,500
	South Branch		
	2. 570 feet of 42-inch storm sewer	64,000	100
	3. 440 feet of 48-inch storm sewer	54,000	100
	4. 250 feet of 54-inch storm sewer	39,000	0
	5. 680 feet of 60-inch storm sewer	124,000	100
	6. 1,550 feet of 66-inch storm sewer	348,000	300
	North Branch		
	7. 340 feet of 36-inch storm sewer	32,000	100
	8. 400 feet of 42-inch storm sewer	45,000	100
	9. 665 feet of 48-inch storm sewer	90,000	100
	10. 2,020 feet of 54-inch storm sewer	314,000	400
	11. 1,740 feet of 60-inch storm sewer	317,000	300
	12. 1,160-foot-long trapezoidal, turf-lined channel with 5-foot bottom width, 4H:1V side slopes, and a 1-foot deep, 2-foot wide riprap-lined low-flow channel	59,000	500
	13. Riprap along 0.16 mile of existing stream	30,000	300
	Subtotal	$ 3,536,000	$ 3,900
I	Area East of Lilly Road and South of W. Good Hope Road		
	1. 560 feet of 30-inch storm sewer	$ 43,000	$ 200
	2. 630 feet of 36-inch storm sewer	59,000	100
	3. 360 feet of 42-inch storm sewer	40,000	100
	Subtotal	$ 142,000	$ 400
J	Woodshaven Tributary		
	1. 1,045 feet of 36-inch storm sewer	$ 97,000	$ 200
	2. Replace existing 7.5-foot-wide x 3.2-foot-high CMPA culvert at Lilly Road with 72 feet total of twin 8-foot-wide x 4-foot-high concrete box culvert	40,000	0
	3. 350-foot-long turf-lined channel with 5-foot bottom width, 3H:1V side slopes, and a 1-foot-deep, 2-foot-wide riprap-lined low-flow channel	39,000	200
	4. Floodproof one house	5,000	- -
	5. Replace existing 3.6-foot-wide x 2.4-foot-high CMPA culvert at Northwood Drive with a 36-foot-long, 7-foot-wide x 3-foot-high concrete box culvert	17,000	0
	6. 530-foot-long turf-lined channel with 5-foot bottom width, 4H:1V side slopes, and a 1-foot-deep, 2-foot-wide riprap-lined low-flow channel	15,000	300
	7. Replace existing 3.5-foot-wide x 2.5-foot-high CMPA culvert at Woodland Drive with a 40-foot-long, 7-foot-wide x 3-foot-high concrete box culvert	19,000	0
	8. 440-foot-long trapezoidal, turf-lined channel with 5-foot bottom width, 4H:1V side slopes, and a 1-foot-deep, 2-foot-wide, riprap-lined low-flow channel	27,000	200
	9. Riprap along 0.79 mile of existing stream	150,000	1,700
	Subtotal	$ 409,000	$ 2,600

Table 34 (continued)

Hydrologic Unit	Project and Component Description[a]	Estimated Cost	
		Capital[b]	Annual Operation and Maintenance[c]
K	Area along W. Appleton Avenue and East and West of Lilly Creek		
	1. 1,315 feet of 36-inch storm sewer	$ 122,000	$ 200
	2. 115 feet of 58-inch-wide x 36-inch-high RCPA storm sewer	19,000	0
	3. Replace existing 36-inch storm sewer at Appleton Avenue with 566 feet of 42-inch storm sewer	71,000	100
	4. Replace existing 30-inch storm sewer at Appleton Avenue with 94 feet of 36-inch storm sewer	10,000	0
	5. Replace existing 27-inch storm sewer at Appleton Avenue with 142 feet of 30-inch storm sewer	12,000	100
	6. Floodproof one house	5,000	- -
	Subtotal	$ 239,000	$ 400
L	Menomonee Manor Tributary		
	1. 1,986 feet of 54-inch relief sewer parallel to existing and committed storm sewers	$ 341,000	$ 400
	2. Riprap along 0.78 mile of existing stream	148,000	1,700
	Subtotal	$ 489,000	$ 2,100
Total		$13,814,000	$31,400

NOTE: The following abbreviations were used in this table:

CMPA - Corrugated Metal Pipe Arch
HE - Horizontal Elliptical
RCPA - Reinforced Concrete Pipe Arch

[a] *All new and replacement sewers are concrete pipe.*

[b] *Capital costs include 35 percent for engineering, administration, and contingencies.*

[c] *Costs were reported as zero when the project proposed replacement of a component with a component which has similar operation and maintenance costs, or when the annual operation and maintenance cost was estimated to be less than $50.*

Source: SEWRPC.

with existing storm sewers. New circular reinforced concrete storm sewers range in diameter from 12 inches to 66 inches. Horizontal elliptical (HE) storm sewer sizes range from 45 inches by 29 inches to 83 inches by 53 inches. Reinforced concrete pipe arch (RCPA) storm sewer sizes range from 36 inches by 23 inches to 73 inches by 45 inches. The alternative also includes 1,590 lineal feet of replacement storm sewer in areas of existing development. Replacement circular storm sewers range in size from 27 inches to 42 inches. A 53-inch by 34-inch horizontal elliptical replacement storm sewer is also proposed.

In addition to the above-noted storm sewers, this alternative includes 5,680 lineal feet of channel enclosure. The total length of pipe required for the enclosure, including double pipes, is 13,510 lineal feet. This channel enclosure is distinguished from the proposed storm sewers in that it is designed to convey the entire 100-year recurrence interval storm runoff without any additional conveyance system. The enclosure consists of single and dual 10-foot-wide by three-foot-high box culverts.

A total of about 2,510 feet of new grass-lined open channels would be provided at outlets of storm sewers. In addition, modifications including widening and deepening would be made along 5,810 feet of existing open channels. Modifications of "natural" streams would include a riprap-lined low-flow channel. As shown on Map 9, this alternative would also maintain 2.2 miles of stream channel tributary to Lilly Creek. Riprap erosion protection would be provided on the bed and banks of these existing streams. This alternative would utilize four existing natural detention basins located in wetland areas and five existing man-made dry detention basins.

Under this alternative, three buildings located along tributaries to Lilly Creek would be floodproofed.

A total of 10 existing road crossings and pedestrian bridges would be replaced, while five pedestrian bridges would be removed and not replaced. Also, one new road crossing would be constructed in an area where a preliminary street layout was available.

It was found that this stormwater drainage alternative could be implemented without significantly modifying the Lilly Creek channel; however, the alternative only addresses the provision of adequate major and minor system drainage facilities for the areas tributary to Lilly Creek. A detailed analysis of alternatives to address flood control in areas adjoining Lilly Creek itself is presented in a subsequent section of this chapter.

Stormwater Drainage Alternative Plan No. 2: Open Channel Conveyance with Selected Storm Sewer Conveyance and Existing Detention Storage

This alternative plan primarily involves the provision of engineered open channels, roadside swales, and new storm sewers to abate existing stormwater runoff problems and to effectively serve planned new urban development in the subwatershed. Where possible, the existing stream channels of Lilly Creek and its tributaries are maintained. Map 10 shows the approximate location and alignment of the engineered open channels, roadside swales, and new storm sewers under the alternative. Table 35 presents the salient characteristics and estimated cost of the engineered open channels, roadside swales, and new storm sewers comprising this alternative plan.

The alternative plan includes 21,300 feet of roadside swales in areas of planned residential development. The plan also includes 6,390 feet of new grass-lined open channels and 620 feet of new riprap-lined open channels. Modifications including deepening, widening and clearing would also be made along 15,600 feet of existing open channel. Modifications of "natural" streams would include a riprap-lined low-flow channel. The use of grassed roadside swales and open channels was avoided in industrial areas, where there is the potential for groundwater contamination due to the infiltration of polluted runoff through the swale sides and bottom. The use of curb and gutter and storm sewers in areas of new industrial development is consistent with current policies of the Village.

The alternative also calls for 24,800 lineal feet of new storm sewers in areas of planned development and 13,900 lineal feet of new storm sewers in areas of existing development. New circular reinforced concrete sewers range in diameter from 12 inches to 66 inches. Reinforced concrete pipe arch storm sewer sizes range from 58 inches by 36 inches to 73 inches by 45 inches. The alternative also includes 800 feet of replacement storm sewer in areas of existing development. Replacement circular storm sewer sizes range from 36 inches to 42 inches.

A total of 19 existing road and driveway crossings and pedestrian bridges would be replaced, while seven pedestrian bridges and driveway crossings would be removed but not replaced. Also, six new road crossings would be constructed in areas where preliminary street layouts were available.

As shown on Map 10, this alternative would also maintain 4.3 miles of stream channel tributary to Lilly Creek. Riprap erosion protection would be provided on the bed and banks of those existing streams. This alternative would utilize four existing natural detention basins located in wetland areas and five existing man-made dry detention basins.

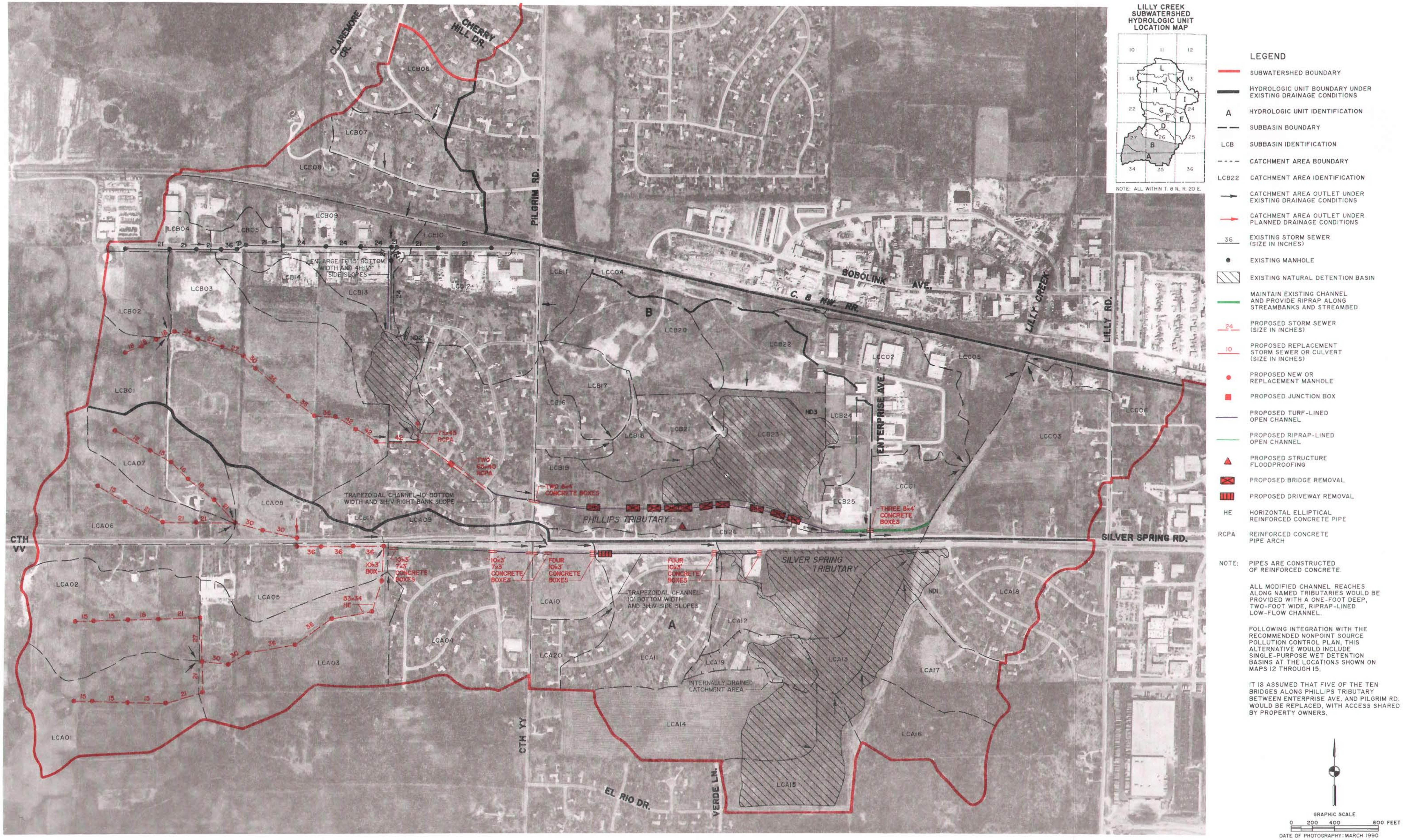
Under this alternative, seven buildings located along tributaries to Lilly Creek would be floodproofed.

It was found that this stormwater drainage alternative could be implemented without significantly modifying the Lilly Creek channel; however, the alternative addresses only the

Map 10

STORMWATER DRAINAGE ALTERNATIVE PLAN NO.2: OPEN CHANNEL CONVEYANCE WITH SELECTED STORM SEWER CONVEYANCE AND EXISTING DETENTION STORAGE

HYDROLOGIC UNITS A AND B



NOTE: PIPES ARE CONSTRUCTED OF REINFORCED CONCRETE.

ALL MODIFIED CHANNEL REACHES ALONG NAMED TRIBUTARIES WOULD BE PROVIDED WITH A ONE-FOOT DEEP, TWO-FOOT WIDE, RIPRAP-LINED LOW-FLOW CHANNEL.

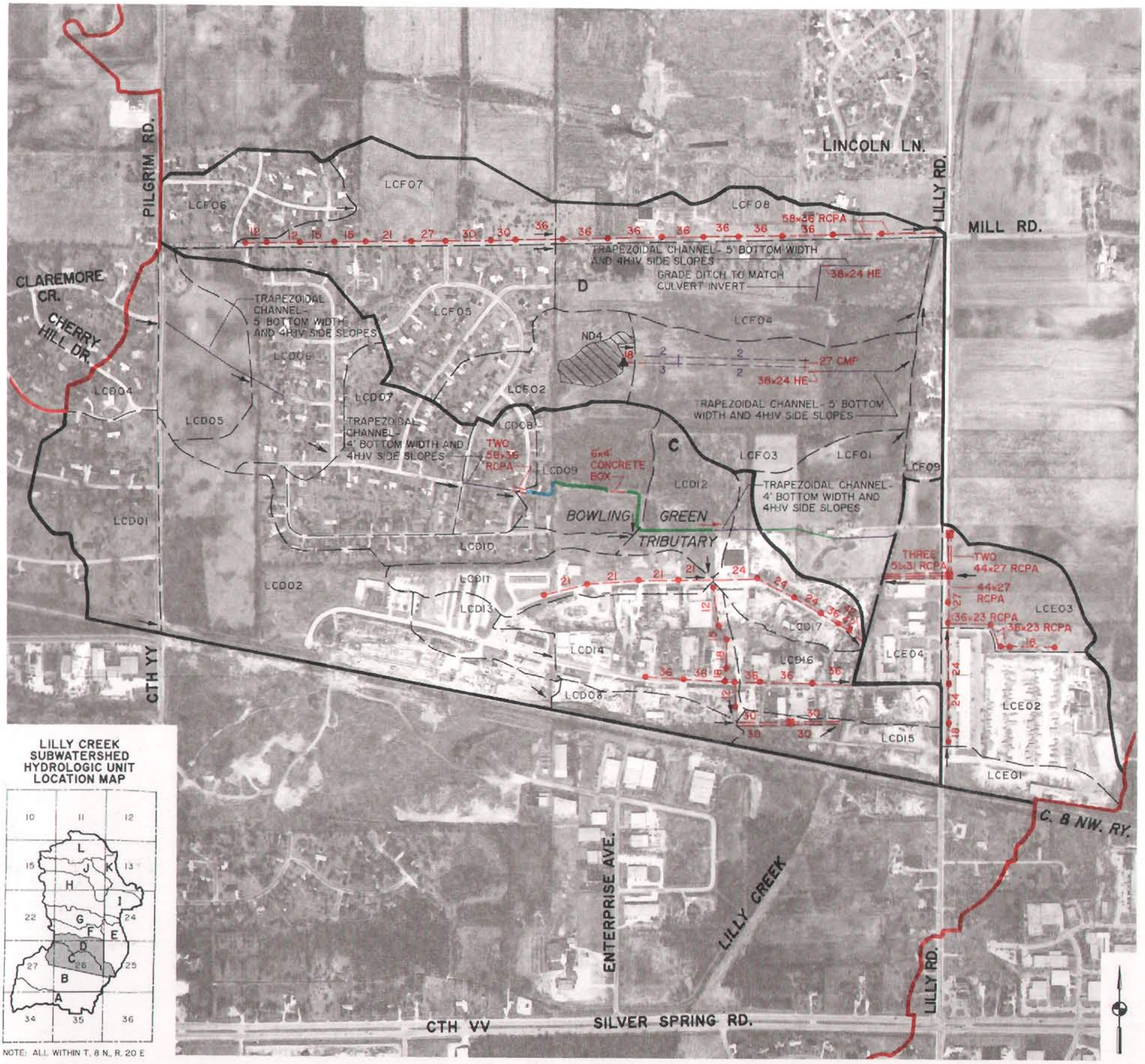
FOLLOWING INTEGRATION WITH THE RECOMMENDED NONPOINT SOURCE POLLUTION CONTROL PLAN, THIS ALTERNATIVE WOULD INCLUDE SINGLE-PURPOSE WET DETENTION BASINS AT THE LOCATIONS SHOWN ON MAPS 12 THROUGH 15.

IT IS ASSUMED THAT FIVE OF THE TEN BRIDGES ALONG PHILLIPS TRIBUTARY BETWEEN ENTERPRISE AVE. AND PILGRIM RD. WOULD BE REPLACED, WITH ACCESS SHARED BY PROPERTY OWNERS.

Map 10 (continued)

STORMWATER DRAINAGE ALTERNATIVE PLAN NO.2: OPEN CHANNEL CONVEYANCE WITH SELECTED STORM SEWER CONVEYANCE AND EXISTING DETENTION STORAGE

HYDROLOGIC UNITS C AND D



LEGEND

🟥	SUBWATERSHED BOUNDARY
▬	HYDROLOGIC UNIT BOUNDARY UNDER EXISTING DRAINAGE CONDITIONS
D	HYDROLOGIC UNIT IDENTIFICATION
— —	SUBBASIN BOUNDARY
LCF	SUBBASIN IDENTIFICATION
- - - -	CATCHMENT AREA BOUNDARY
LCF03	CATCHMENT AREA IDENTIFICATION
→	CATCHMENT AREA OUTLET UNDER EXISTING DRAINAGE CONDITIONS
→	CATCHMENT AREA OUTLET UNDER PLANNED DRAINAGE CONDITIONS
▲	PROPOSED OUTLET STRUCTURE

▨	EXISTING NATURAL DETENTION BASIN
🟩	MAINTAIN EXISTING CHANNEL AND PROVIDE RIPRAP ALONG STREAMBANKS AND STREAMBED
🟦	CHANNEL CLEANING AND DEBRUSHING WITH RIPRAP PROVIDED ALONG STREAMBANKS AND STREAMBED
36	PROPOSED STORM SEWER (SIZE IN INCHES)
24	PROPOSED REPLACEMENT STORM SEWER OR CULVERT (SIZE IN INCHES)
●	PROPOSED NEW OR REPLACEMENT MANHOLE
■	PROPOSED JUNCTION BOX

—	PROPOSED TURF-LINED OPEN CHANNEL
—	PROPOSED RIPRAP-LINED OPEN CHANNEL
2	PROPOSED ROADSIDE SWALE (DEPTH IN FEET)
HE	HORIZONTAL ELLIPTICAL REINFORCED CONCRETE PIPE
RCPA	REINFORCED CONCRETE PIPE ARCH
CMP	CORRUGATED METAL PIPE

NOTE: PIPES ARE CONSTRUCTED OF REINFORCED CONCRETE UNLESS DESIGNATED AS ABOVE.

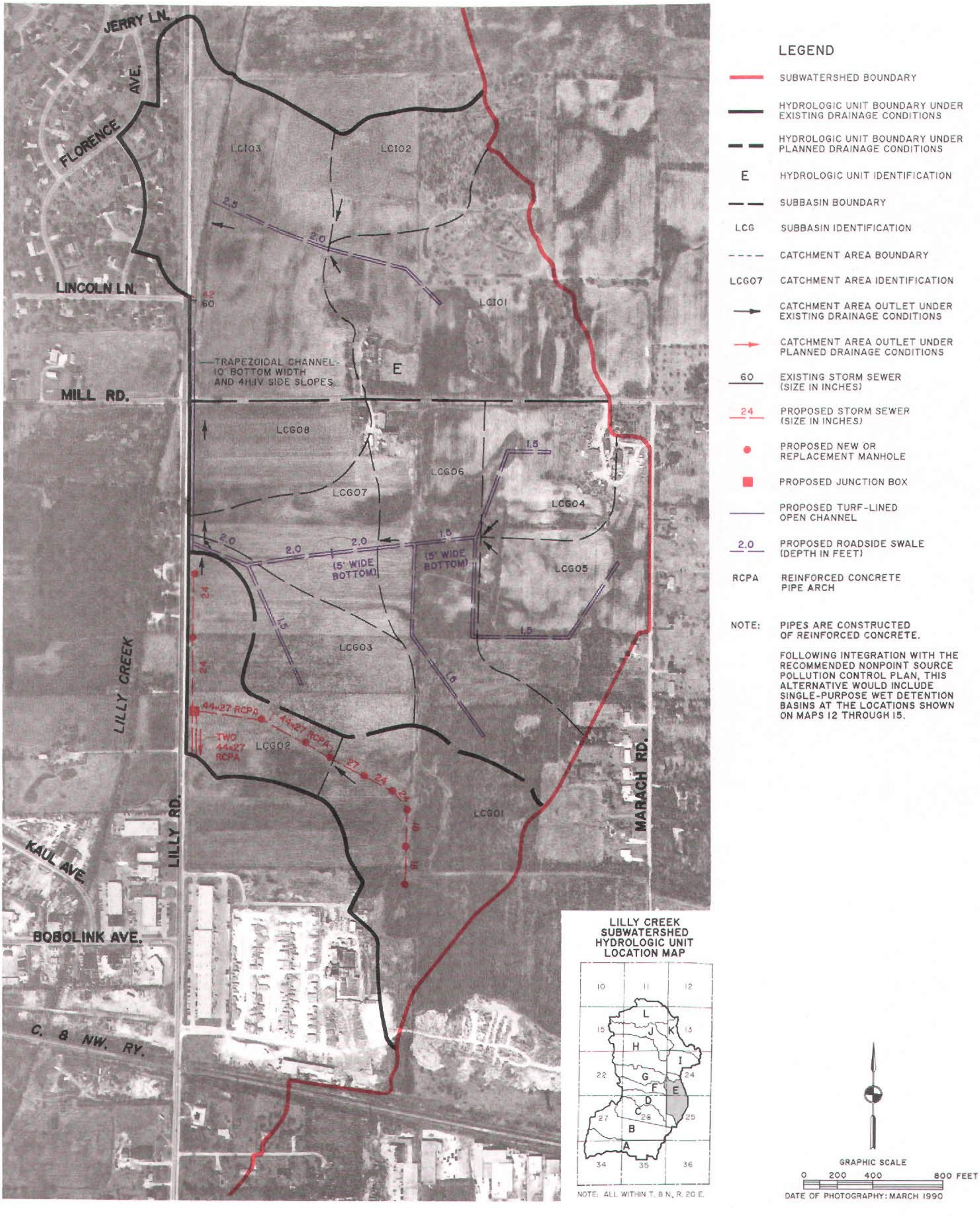
ALL MODIFIED CHANNEL REACHES ALONG NAMED TRIBUTARIES WOULD BE PROVIDED WITH A ONE-FOOT DEEP, TWO-FOOT WIDE, RIPRAP-LINED LOW-FLOW CHANNEL.

FOLLOWING INTEGRATION WITH THE RECOMMENDED NONPOINT SOURCE POLLUTION CONTROL PLAN, THIS ALTERNATIVE WOULD INCLUDE SINGLE-PURPOSE WET DETENTION BASINS AT THE LOCATIONS SHOWN ON MAPS 12 THROUGH 15.

GRAPHIC SCALE
0 200 400 800 FEET
DATE OF PHOTOGRAPHY: MARCH 1990

Map 10 (continued)

STORMWATER DRAINAGE ALTERNATIVE PLAN NO.2: OPEN CHANNEL CONVEYANCE WITH SELECTED STORM SEWER CONVEYANCE AND EXISTING DETENTION STORAGE HYDROLOGIC UNIT E



LEGEND

—	SUBWATERSHED BOUNDARY
—	HYDROLOGIC UNIT BOUNDARY UNDER EXISTING DRAINAGE CONDITIONS
— —	HYDROLOGIC UNIT BOUNDARY UNDER PLANNED DRAINAGE CONDITIONS
E	HYDROLOGIC UNIT IDENTIFICATION
— —	SUBBASIN BOUNDARY
LCG	SUBBASIN IDENTIFICATION
- - - -	CATCHMENT AREA BOUNDARY
LCG07	CATCHMENT AREA IDENTIFICATION
→	CATCHMENT AREA OUTLET UNDER EXISTING DRAINAGE CONDITIONS
→	CATCHMENT AREA OUTLET UNDER PLANNED DRAINAGE CONDITIONS
60	EXISTING STORM SEWER (SIZE IN INCHES)
24	PROPOSED STORM SEWER (SIZE IN INCHES)
●	PROPOSED NEW OR REPLACEMENT MANHOLE
■	PROPOSED JUNCTION BOX
—	PROPOSED TURF-LINED OPEN CHANNEL
2.0	PROPOSED ROADSIDE SWALE (DEPTH IN FEET)
RCPA	REINFORCED CONCRETE PIPE ARCH

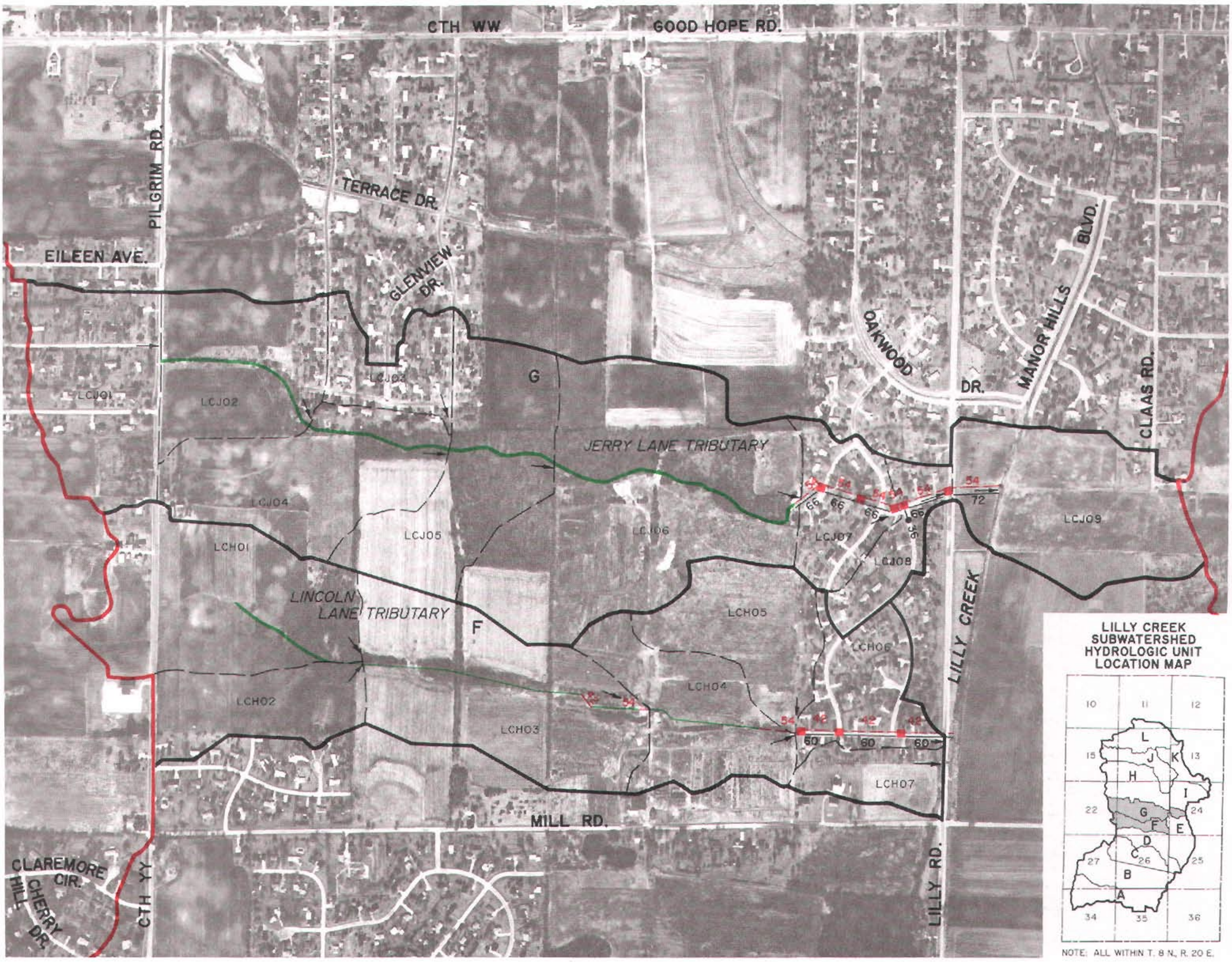
NOTE: PIPES ARE CONSTRUCTED OF REINFORCED CONCRETE.

FOLLOWING INTEGRATION WITH THE RECOMMENDED NONPOINT SOURCE POLLUTION CONTROL PLAN, THIS ALTERNATIVE WOULD INCLUDE SINGLE-PURPOSE WET DETENTION BASINS AT THE LOCATIONS SHOWN ON MAPS 12 THROUGH 15.

Map 10 (continued)

STORMWATER DRAINAGE ALTERNATIVE PLAN NO.2: OPEN CHANNEL CONVEYANCE WITH SELECTED STORM SEWER CONVEYANCE AND EXISTING DETENTION STORAGE

HYDROLOGIC UNITS F AND G



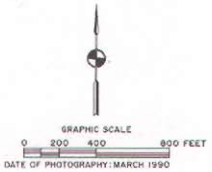
LEGEND

—	SUBWATERSHED BOUNDARY
—	HYDROLOGIC UNIT BOUNDARY UNDER EXISTING DRAINAGE CONDITIONS
G	HYDROLOGIC UNIT IDENTIFICATION
— —	SUBBASIN BOUNDARY
LCF	SUBBASIN IDENTIFICATION
- - - -	CATCHMENT AREA BOUNDARY
LCF07	CATCHMENT AREA IDENTIFICATION
→	CATCHMENT AREA OUTLET UNDER EXISTING DRAINAGE CONDITIONS
60	EXISTING STORM SEWER (SIZE IN INCHES)
●	EXISTING MANHOLE
—	MAINTAIN EXISTING CHANNEL AND PROVIDE RIPRAP ALONG STREAMBANKS AND STREAMBED
54	PROPOSED STORM SEWER (SIZE IN INCHES)
42	PROPOSED CULVERT (SIZE IN INCHES)
■	PROPOSED JUNCTION BOX
—	PROPOSED RIPRAP-LINED OPEN CHANNEL

NOTE: PIPES ARE CONSTRUCTED OF REINFORCED CONCRETE.

ALL MODIFIED CHANNEL REACHES ALONG NAMED TRIBUTARIES WOULD BE PROVIDED WITH A ONE-FOOT DEEP, TWO-FOOT WIDE, RIPRAP-LINED LOW-FLOW CHANNEL.

FOLLOWING INTEGRATION WITH THE RECOMMENDED NONPOINT SOURCE POLLUTION CONTROL PLAN, THIS ALTERNATIVE WOULD INCLUDE SINGLE-PURPOSE WET DETENTION BASINS AT THE LOCATIONS SHOWN ON MAPS 12 THROUGH 15.

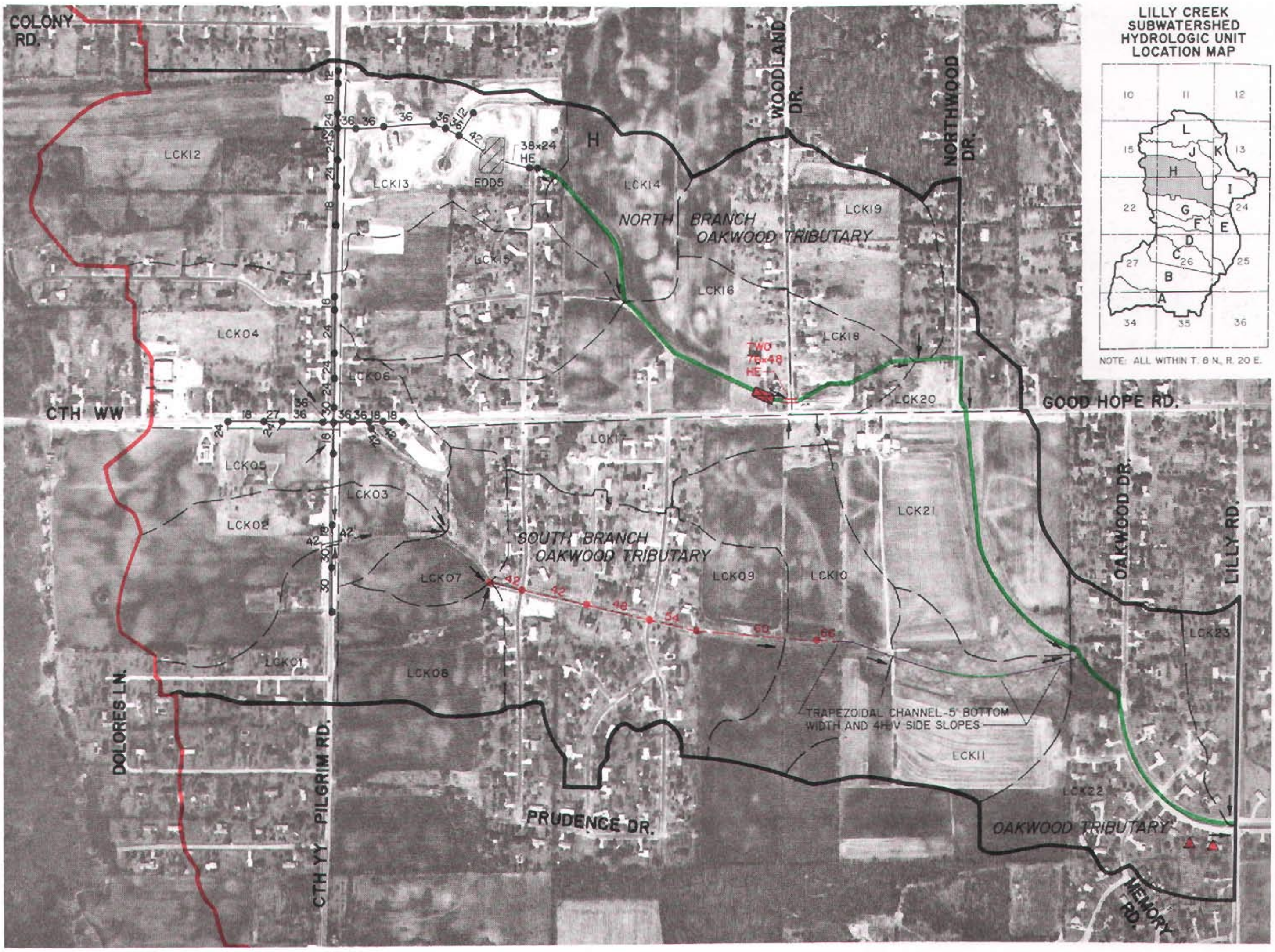


DATE OF PHOTOGRAPHY: MARCH 1990

Map 10 (continued)

STORMWATER DRAINAGE ALTERNATIVE PLAN NO.2: OPEN CHANNEL CONVEYANCE WITH SELECTED STORM SEWER CONVEYANCE AND EXISTING DETENTION STORAGE

HYDROLOGIC UNIT H



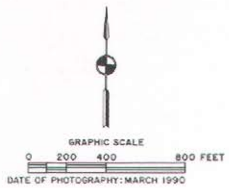
LEGEND

—	SUBWATERSHED BOUNDARY
—	HYDROLOGIC UNIT BOUNDARY UNDER EXISTING DRAINAGE CONDITIONS
H	HYDROLOGIC UNIT IDENTIFICATION
— —	SUBBASIN BOUNDARY
LCK	SUBBASIN IDENTIFICATION
- - - -	CATCHMENT AREA BOUNDARY
LCK16	CATCHMENT AREA IDENTIFICATION
→	CATCHMENT AREA OUTLET UNDER EXISTING DRAINAGE CONDITIONS
36	EXISTING STORM SEWER (SIZE IN INCHES)
●	EXISTING MANHOLE
▨	EXISTING MAN-MADE DRY DETENTION BASIN
—	MAINTAIN EXISTING CHANNEL AND PROVIDE RIPRAP ALONG STREAMBANKS AND STREAMBED
42	PROPOSED STORM SEWER (SIZE IN INCHES)
76	PROPOSED REPLACEMENT STORM SEWER OR CULVERT (SIZE IN INCHES)
●	PROPOSED NEW OR REPLACEMENT MANHOLE
—	PROPOSED TURF-LINED OPEN CHANNEL
—	PROPOSED RIPRAP-LINED OPEN CHANNEL
▲	PROPOSED STRUCTURE FLOODPROOFING
☒	PROPOSED BRIDGE REMOVAL
HE	HORIZONTAL ELLIPTICAL REINFORCED CONCRETE PIPE

NOTE: PIPES ARE CONSTRUCTED OF REINFORCED CONCRETE.

ALL MODIFIED CHANNEL REACHES ALONG NAMED TRIBUTARIES WOULD BE PROVIDED WITH A ONE-FOOT DEEP, TWO-FOOT WIDE, RIPRAP-LINED LOW-FLOW CHANNEL.

FOLLOWING INTEGRATION WITH THE RECOMMENDED NONPOINT SOURCE POLLUTION CONTROL PLAN, THIS ALTERNATIVE WOULD INCLUDE SINGLE-PURPOSE WET DETENTION BASINS AT THE LOCATIONS SHOWN ON MAPS 12 THROUGH 15.

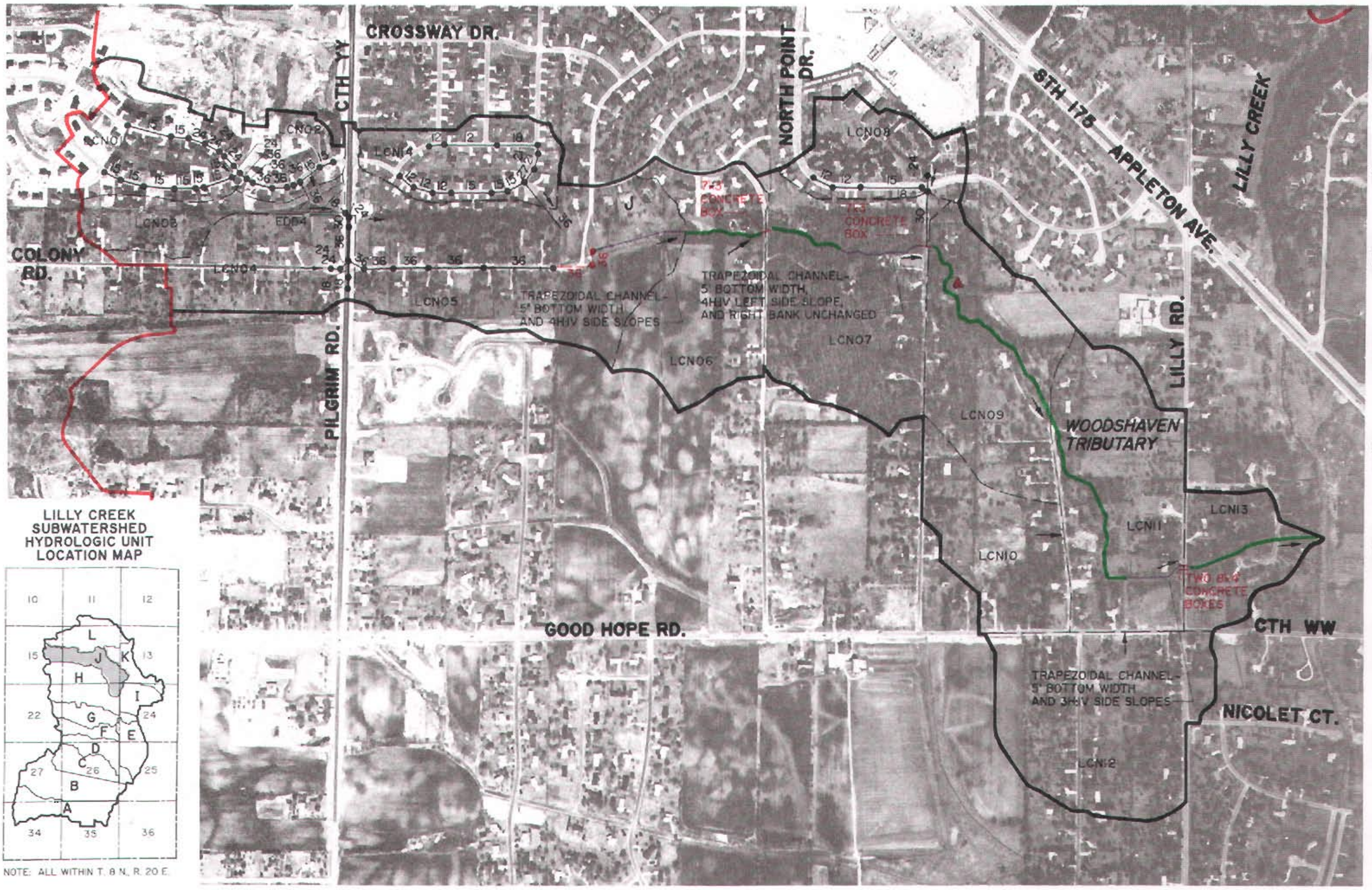


DATE OF PHOTOGRAPHY: MARCH 1990

Map 10 (continued)

STORMWATER DRAINAGE ALTERNATIVE PLAN NO.2: OPEN CHANNEL CONVEYANCE WITH SELECTED STORM SEWER CONVEYANCE AND EXISTING DETENTION STORAGE

HYDROLOGIC UNIT J



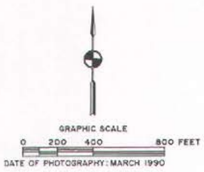
LEGEND

—	SUBWATERSHED BOUNDARY
—	HYDROLOGIC UNIT BOUNDARY UNDER EXISTING DRAINAGE CONDITIONS
J	HYDROLOGIC UNIT IDENTIFICATION
— —	SUBBASIN BOUNDARY
LCN	SUBBASIN IDENTIFICATION
- - - -	CATCHMENT AREA BOUNDARY
LCN09	CATCHMENT AREA IDENTIFICATION
→	CATCHMENT AREA OUTLET UNDER EXISTING DRAINAGE CONDITIONS
42	EXISTING STORM SEWER (SIZE IN INCHES)
●	EXISTING MANHOLE
▨	EXISTING MAN-MADE DRY DETENTION BASIN
—	MAINTAIN EXISTING CHANNEL AND PROVIDE RIPRAP ALONG STREAMBANKS AND STREAMBED
36	PROPOSED STORM SEWER (SIZE IN INCHES)
—	PROPOSED REPLACEMENT CULVERT
●	PROPOSED NEW OR REPLACEMENT MANHOLE
—	PROPOSED TURF-LINED OPEN CHANNEL
▲	PROPOSED STRUCTURE FLOODPROOFING

NOTE: PIPES ARE CONSTRUCTED OF REINFORCED CONCRETE.

ALL MODIFIED CHANNEL REACHES ALONG NAMED TRIBUTARIES WOULD BE PROVIDED WITH A ONE-FOOT DEEP, TWO-FOOT WIDE, RIPRAP-LINED LOW-FLOW CHANNEL.

FOLLOWING INTEGRATION WITH THE RECOMMENDED NONPOINT SOURCE POLLUTION CONTROL PLAN, THIS ALTERNATIVE WOULD INCLUDE SINGLE-PURPOSE WET DETENTION BASINS AT THE LOCATIONS SHOWN ON MAPS 12 THROUGH 15.

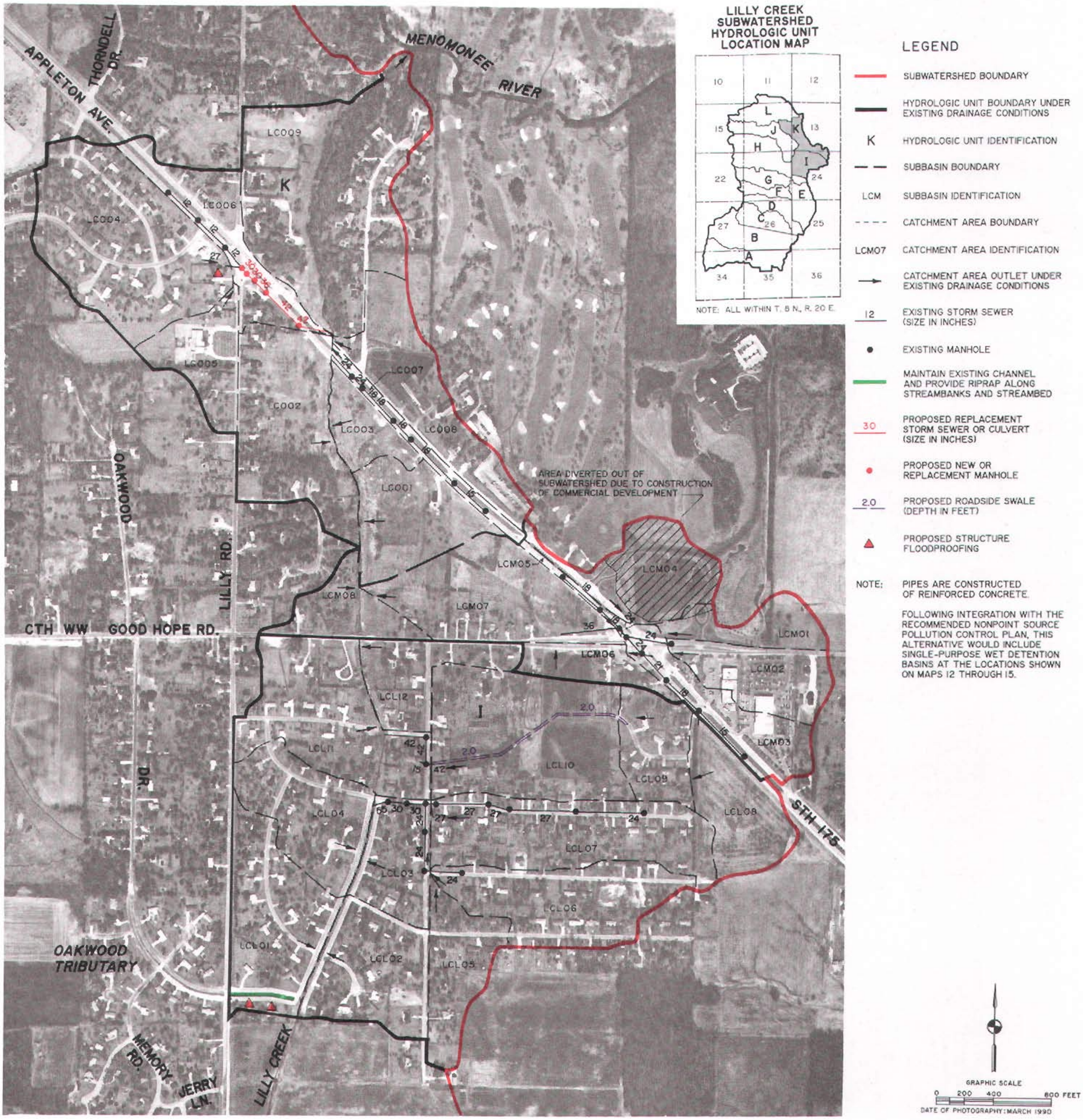


DATE OF PHOTOGRAPHY: MARCH 1990

Map 10 (continued)

STORMWATER DRAINAGE ALTERNATIVE PLAN NO.2: OPEN CHANNEL CONVEYANCE WITH SELECTED STORM SEWER CONVEYANCE AND EXISTING DETENTION STORAGE

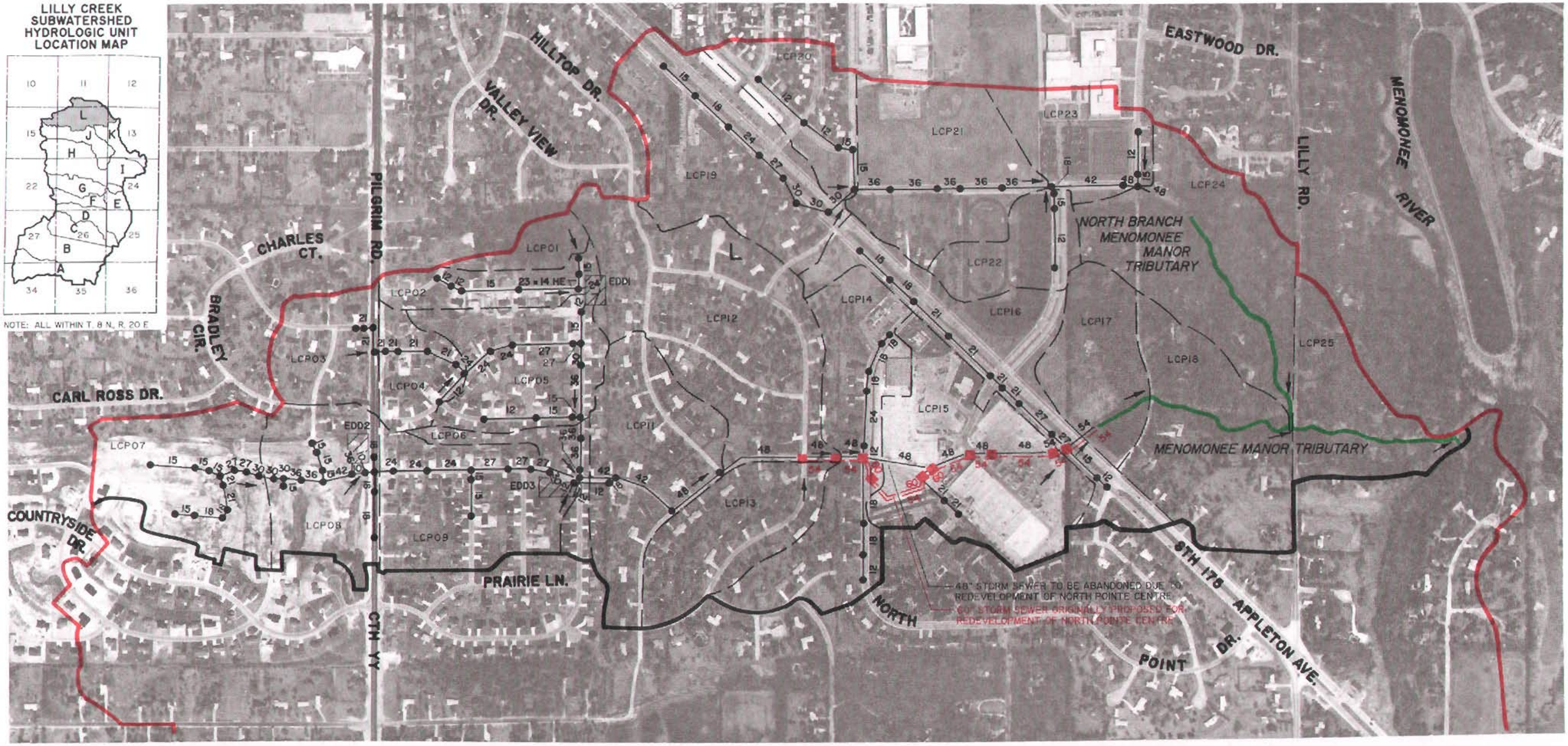
HYDROLOGIC UNITS I AND K



Map 10 (continued)

STORMWATER DRAINAGE ALTERNATIVE PLAN NO.2: OPEN CHANNEL CONVEYANCE WITH SELECTED STORM SEWER CONVEYANCE AND EXISTING DETENTION STORAGE

HYDROLOGIC UNIT L



LEGEND

🟥	SUBWATERSHED BOUNDARY
▬	HYDROLOGIC UNIT BOUNDARY UNDER EXISTING DRAINAGE CONDITIONS
L	HYDROLOGIC UNIT IDENTIFICATION
— —	SUBBASIN BOUNDARY
LCP	SUBBASIN IDENTIFICATION
- - - -	CATCHMENT AREA BOUNDARY
LCP22	CATCHMENT AREA IDENTIFICATION
→	CATCHMENT AREA OUTLET UNDER EXISTING DRAINAGE CONDITIONS
21	EXISTING STORM SEWER (SIZE IN INCHES)
●	EXISTING MANHOLE
▨	EXISTING MAN-MADE DRY DETENTION BASIN
🟩	MAINTAIN EXISTING CHANNEL AND PROVIDE RIPRAP ALONG STREAMBANKS AND STREAMBED
54	PROPOSED STORM SEWER (SIZE IN INCHES)
●	PROPOSED NEW OR REPLACEMENT MANHOLE
■	PROPOSED JUNCTION BOX
HE	HORIZONTAL ELLIPTICAL REINFORCED CONCRETE PIPE
NOTE:	PIPES ARE CONSTRUCTED OF REINFORCED CONCRETE.
	FOLLOWING INTEGRATION WITH THE RECOMMENDED NONPOINT SOURCE POLLUTION CONTROL PLAN, THIS ALTERNATIVE WOULD INCLUDE SINGLE-PURPOSE WET DETENTION BASINS AT THE LOCATIONS SHOWN ON MAPS 12 THROUGH 15.

GRAPHIC SCALE

0 200 400 800 FEET

DATE OF PHOTOGRAPHY: MARCH 1990

Source: SEWRPC.

Table 35

COMPONENTS AND COSTS OF STORMWATER DRAINAGE ALTERNATIVE NO. 2: OPEN CHANNEL CONVEYANCE WITH SELECTED STORM SEWER CONVEYANCE AND EXISTING DETENTION STORAGE

Hydrologic Unit	Project and Component Description[a]	Estimated Cost	
		Capital[b]	Annual Operation and Maintenance[c]
A	Silver Spring Tributary		
	1. 380 feet of 12-inch storm sewer	$ 16,000	$ 200
	2. 1,790 feet of 15-inch storm sewer	82,000	700
	3. 825 feet of 18-inch storm sewer	42,000	300
	4. 2,280 feet of 21-inch storm sewer	132,000	900
	5. 380 feet of 27-inch storm sewer	26,000	200
	6. 1,010 feet of 30-inch storm sewer	78,000	400
	7. 1,730 feet of 36-inch storm sewer	161,000	300
	8. 300 feet of 10-foot x 3-foot concrete box culvert	150,000	100
	9. 680 feet of 53-inch x 34-inch concrete HE storm sewer	92,000	100
	10. 3,430-foot long, turf-lined channel with a 10-foot bottom width and 3H:1V side slopes. Include a 1-foot-deep, 2-foot-wide, riprap-lined low-flow channel	162,000	1,400
	11. Replace existing culverts at Fire Station with four 50-foot-long, 10-foot-wide x 3-foot high concrete box culverts	105,000	0
	12. Replace existing culverts at Badger Drive with four 36-foot-long, 10-foot-wide x 3-foot-high concrete box culverts	76,000	0
	13. Replace existing culverts at Butternut Drive with four 36-foot-long, 10-foot-wide x 3-foot-high concrete box culverts	76,000	0
	14. Replace existing culverts private drive with a 42-foot-long, 10-foot-wide x 3-foot-high concrete box culvert and a 42-foot-long, 7-foot-wide x 3-foot-high concrete box culvert	42,000	0
	15. Replace existing culverts at Pilgrim Road with an 83-foot-long, 10-foot-wide x 3-foot-high concrete box culvert and an 83-foot-long, 7-foot-wide x 3-foot-high concrete box culvert	83,000	0
	16. Replace existing culverts at Bette Drive with a 36-foot-long, 10-foot-wide x 3-foot-high concrete box culvert and a 36-foot-long, 7-foot-wide x 3-foot-high concrete box culvert	36,000	0
	17. Replace existing culverts at private drive with a 28-foot-long, 10-foot-wide x 3-foot-high concrete box culvert and a 28-foot-long, 7-foot-wide x 3-foot-high concrete box culvert	28,000	0
	18. Remove private drive	1,000	- -
	Subtotal	$1,388,000	$ 4,600
B	Phillips Tributary		
	1. Replace existing culverts at Enterprise Drive with three 43-foot-long, 8-foot-wide x 4-foot-high concrete box culverts	$ 71,000	$ 0
	2. Replace existing culverts at Pilgrim Road with two 40-foot-long, 8-foot-wide x 4-foot-high concrete box culverts	44,000	0
	3. Remove 10 private pedestrian bridges. Replace five bridges	107,000	0
	4. Construct 3,300-foot-long trapezoidal, turf-lined channel with 10-foot bottom width and 3H:1V right bank slope. Include a 1-foot-deep, 2-foot-wide, riprap-lined low-flow channel	172,000	1,400
	5. Floodproof one house	5,000	0
	6. 510 feet of 18-inch storm sewer	26,000	200
	7. 230 feet of 24-inch storm sewer	14,000	100
	8. 460 feet of 27-inch storm sewer	32,000	200
	9. 150 feet of 30-inch storm sewer	12,000	100
	10. 860 feet of 36-inch storm sewer	80,000	200
	11. 830 feet of 42-inch storm sewer	93,000	200
	12. 760 feet total of double 65-inch x 40-inch RCPA storm sewer	145,000	100
	13. 550 feet of 73-inch x 45-inch RCPA storm sewer	127,000	100
	14. Enlarge existing 765-foot-long channel to a 15-foot bottom width with 4H:1V side slopes	56,000	300
	15. Riprap erosion protection along existing stream	27,000	300
	Subtotal	$1,011,000	$ 3,200

Table 35 (continued)

Hydrologic Unit	Project and Component Description[a]	Estimated Cost	
		Capital[b]	Annual Operation and Maintenance[c]
C	Bowling Green Tributary		
	1. 520 feet of 12-inch storm sewer	$ 22,000	$ 200
	2. 110 feet of 15-inch storm sewer	5,000	0
	3. 290 feet of 18-inch storm sewer	15,000	100
	4. 1,205 feet of 21-inch storm sewer	82,000	500
	5. 795 feet of 24-inch storm sewer	58,000	300
	6. 1,460 feet of 30-inch storm sewer	112,000	600
	7. 1,510 feet of 36-inch storm sewer	159,000	300
	8. 185 feet of 42-inch storm sewer	23,000	0
	9. Channel cleaning and debrushing	2,000	600
	10. Construct 1,245-foot-long (620 feet to be riprapped and 625 feet to be turf-lined) trapezoidal, diversion channel with 4-foot bottom width and 4H:1V side slopes	109,000	500
	11. 40 feet of 6-foot-wide x 4-foot-high concrete box culvert	20,000	0
	12. Replace existing culvert at Wampum Drive with two 58-inch x 36-inch RCPA storm sewers	12,000	0
	13. 340-foot-long trapezoidal, turf-lined channel with 4-foot bottom width and 4H:1V side slopes	3,000	200
	14. 1,060-foot-long trapezoidal, turf-lined channel with 4-foot bottom width and 4H:1V side slopes	11,000	500
	15. Riprap along 0.32 mile of existing stream	61,000	700
	Subtotal	$ 694,000	$ 4,500
D	Area Predominantly West of Lilly Creek and North and South of W. Mill Road		
	1. 440 feet of 12-inch storm sewer	$ 22,000	$ 200
	2. 430 feet of 15-inch storm sewer	23,000	200
	3. 1,015 feet of 18-inch storm sewer	52,000	200
	4. 325 feet of 21-inch storm sewer	22,000	100
	5. 1,780 feet of 24-inch storm sewer	111,000	700
	6. 560 feet of 27-inch storm sewer	41,000	200
	7. 475 feet of 30-inch storm sewer	42,000	200
	8. 2,170 feet of 36-inch storm sewer	229,000	400
	9. 730 feet of 58-inch x 36-inch RCPA storm sewer	131,000	100
	10. 540 feet of 36-inch x 23-inch RCPA storm sewer	52,000	200
	11. 2,215 feet of 44-inch x 27-inch RCPA storm sewer	239,000	400
	12. 1,305 feet total of 51-inch x 31-inch RCPA storm sewer	176,000	200
	13. 120 feet of 38-inch x 24-inch HE culvert	12,000	0
	14. 60 feet of 27-inch CMP culvert	3,000	0
	15. 1,980 feet of 2-foot-deep roadside swale with driveway culverts	42,000	1,200
	16. 240 feet of 3-foot-deep roadside swale with driveway culverts	7,000	200
	17. 540-foot-long trapezoidal, turf-lined channel with a 5-foot bottom width and 4H:1V side slopes	14,000	200
	18. 250-foot-long trapezoidal, turf-lined channel with a 5-foot bottom width and 4H:1V side slopes	3,000	100
	Subtotal	$1,221,000	$ 4,800
E	Area East of Lilly Creek and North and South of W. Mill Road		
	1. 8,690 feet of 1.5-foot-deep roadside swale with driveway culverts	$ 157,000	$ 4,900
	2. 4,210 feet of 2-foot-deep roadside swale with driveway culverts	88,000	2,600
	3. 200 feet of 2.5-foot-deep roadside swale with driveway culverts	5,000	100
	4. 660 feet of 1.5-foot-deep open channel with a 5-foot bottom width	7,000	400
	5. 850 feet of 2-foot-deep open channel with a 5-foot bottom width	13,000	500
	6. 1,325 feet of 2.5-foot-deep open channel with a 10-foot bottom width	29,000	600
	Subtotal	$ 299,000	$ 9,100

Table 35 (continued)

Hydrologic Unit	Project and Component Description[a]	Estimated Cost	
		Capital[b]	Annual Operation and Maintenance[c]
F	Lincoln Lane Tributary		
	1. 1,045 feet of 42-inch storm sewer	$ 131,000	$ 200
	2. 315 feet of 54-inch storm sewer	49,000	100
	3. 2,850-foot-long trapezoidal, riprap-lined channel with 5-foot bottom width and 4H:1V side slopes (include one 60-foot-long, 54-inch reinforced concrete culvert pipe and one 100-foot-long, 48-inch reinforced concrete culvert pipe)	461,000	1,100
	4. Riprap along 0.14 mile of existing stream	27,000	300
	Subtotal	$ 668,000	$ 1,700
G	Jerry Lane Tributary		
	1. 1,375 feet of 54-inch storm sewer parallel to existing storm sewer from Washington Avenue to Lilly Creek	$ 236,000	$ 300
	2. Riprap along 0.87 mile of existing stream	165,000	1,800
	Subtotal	$ 401,000	$ 2,100
H	Oakwood Tributary		
	Main Stem		
	1. Floodproof four houses	$ 19,000	$ 0
	South Branch		
	2. 1,445-foot-long trapezoidal channel (640 feet to be riprapped and 805 feet to be turf-lined) with 5-foot bottom width and 4H:1V side slopes	102,000	500
	3. 570 feet of 42-inch storm sewer	64,000	100
	4. 400 feet of 48-inch storm sewer	54,000	100
	5. 250 feet of 54-inch storm sewer	39,000	0
	6. 680 feet of 60-inch storm sewer	124,000	100
	7. 160 feet of 66-inch storm sewer	35,000	0
	North Branch		
	8. Replace two existing 6.1-foot-wide x 4.6-foot-high CMPA culverts at Woodland Drive with two 40-foot-long, 76-inch-wide x 48-inch-high HE culvert pipes	20,000	0
	9. Remove existing private bridge	1,000	- -
	10. Riprap along 0.96 mile of existing stream	194,000	2,100
	Subtotal	$ 652,000	$ 2,900
I	Area East of Lilly Creek and South of W. Good Hope Road		
	1. 3,100 feet of 2-foot-deep roadside swale with driveway culverts	$ 65,000	$ 1,900
	Subtotal	$ 65,000	$ 1,900
J	Woodshaven Tributary		
	1. Replace existing 7.5-foot-wide x 3.2-foot-high CMPA culvert at Lilly Road with 72 feet total of twin 8-foot-wide x 4-foot-high concrete box culvert	$ 40,000	$ 0
	2. Construct 350-foot-long turf-lined channel with 5-foot bottom width and 3H:1V side slopes	39,000	200
	3. Floodproof one house	5,000	0
	4. Replace existing 3.6-foot-wide x 2.4-foot-high culvert at Northwood Drive with a 36-foot-long, 7-foot-wide x 3-foot-high concrete box culvert	17,000	0
	5. 530-foot-long trapezoidal, turf-lined channel with 5-foot bottom width and 4H:1V side slopes	10,000	300
	6. Replace existing 3.5-foot-wide x 2.5-foot-high CMPA culvert at Woodland Drive with a 40-foot-long, 7-foot-wide x 3-foot-high concrete box culvert	19,000	0
	7. 40 feet of 7-foot-wide x 3-foot-high concrete box culvert	18,000	0
	8. Construct 740-foot-long turf-lined channel with 5-foot bottom width and 4H:1V side slopes	34,000	300
	9. 340 feet of 36-inch storm sewer	32,000	100
	10. Riprap along 0.88 mile of existing stream	167,000	1,900
	Subtotal	$ 381,000	$ 2,800

Table 35 (continued)

Hydrologic Unit	Project and Component Description[a]	Estimated Cost Capital[b]	Estimated Cost Annual Operation and Maintenance[c]
K	Area along W. Appleton Avenue and East and West of Lilly Creek		
	1. Replace existing 36-inch storm sewer in Appleton Avenue with 566 feet of 42-inch storm sewer	$ 71,000	$ 100
	2. Replace existing 30-inch storm sewer in Appleton Avenue with 94 feet of 36-inch storm sewer	10,000	0
	3. Replace existing 27-inch storm sewer in Appleton Avenue with 142 feet of 30-inch storm sewer	12,000	100
	4. Floodproof one house	5,000	0
	Subtotal	$ 98,000	$ 200
L	Menomonee Manor Tributary		
	1. 1,986 feet of 54-inch relief sewer parallel to existing storm sewer	$ 341,000	$ 400
	2. Riprap along 0.78 mile of existing stream	148,000	1,700
	Subtotal	$ 489,000	$ 2,100
Total		$7,367,000[d]	$39,900

NOTE: The following abbreviations were used in this table:

CMP - Corrugated Metal Pipe
CMPA - Corrugated Metal Pipe Arch
HE - Horizontal Elliptical
RCPA - Reinforced Concrete Pipe Arch

[a] *All new and replacement sewers are concrete pipe.*

[b] *Capital costs include 35 percent for engineering, administration, and contingencies.*

[c] *Costs were noted to be zero when the project proposed replacement of a component with a component which has similar operation and maintenance costs, or when the annual operation and maintenance cost was estimated to be less than $50.*

[d] *Based on current fair market values, the incremental value of land in the 100-year recurrence interval floodplain along tributary streams which is preserved in the floodplain under this alternative, but eliminated under Alternative No. 1, is approximately $25,000. That $25,000 is not included in the total capital cost.*

Source: SEWRPC.

provision of adequate major and minor system drainage facilities for the areas tributary to Lilly Creek. A detailed analysis of alternatives to address flood control in areas adjoining Lilly Creek itself is presented in a subsequent section of this chapter.

Stormwater Drainage Alternative Plan No. 3: Maximum Detention Storage with a Combination of Open Channel and Storm Sewer Conveyance

This alternative plan primarily involves the provision of detention storage along with new storm sewers and engineered open channels to abate existing stormwater runoff problems and to effectively serve planned new urban development in the subwatershed. Where possible, the existing stream channels of Lilly Creek and its tributaries are maintained.

The proposed detention basins were assumed to be supplemented by a mixture of the storm sewer and open channel conveyance facilities proposed under Alternative Plan Nos. 1 and 2. Those facilities were selected for inclusion in this alternative plan on the basis of development trends in the subwatershed and of considerations related to drainage, nonpoint source pollution control, existing and planned land uses, and preservation of aquatic and terrestrial habitat. The use of grassed roadside swales was avoided in industrial areas, where there is the potential for groundwater contamination due to the infiltration of polluted runoff through the swale sides and bottom. As set forth in the subsequent section of this chapter which evaluates nonpoint source pollution control alternatives, the use of roadside swales in areas of planned medium-density residential development would provide only a marginal additional degree of control of pollutants. The hydrologic analyses conducted for this study showed that use of roadside swales would not significantly reduce flood flows or volumes in the 10- to 100-year recurrence interval range compared with flows generated with storm sewer conveyance. The use of swales along major drainageways in areas of planned medium-density residential development may cause potential hydraulic problems associated with closely spaced driveway culverts and potential maintenance problems in comparison to storm sewer conveyance. On balance, the use of grassed roadside swales along major drainageways in most areas of medium-density residential development was considered to present more negative than positive aspects; therefore, such swales were excluded from most areas of new medium-density residential development under this alternative. The use of this approach for the main trunk sewer and drainageway system in the planned medium-density residential areas does not preclude the use of roadside swales in the tributary area conveyance system but rather allows the flexibility of providing either roadside swale or storm sewer conveyance in the tributary areas.

Reductions in peak stormwater flows caused by the proposed detention basins were accounted for in the sizing of the conveyance components included in this alternative.

Map 11 shows the approximate location and alignment of the measures proposed under the alternative. Table 36 presents the salient characteristics and estimated costs of the new detention basins, storm sewers, culverts, roadside swales, and open channels comprising this alternative plan.

This alternative would maximize the use of detention storage within the subwatershed. In addition to the four existing natural detention basins and the five existing man-made dry detention basins in the subwatershed, the alternative proposes the construction of 15 dual-purpose wet detention basins for the control of both water quantity and quality and three single-purpose dry detention basins for control of water quantity. Thirteen of the 15 dual-purpose basins would be designed to control peak flows during storms with recurrence intervals ranging from two through 100 years. Two of the dual-purpose basins would control peak flows only during storms with recurrence intervals up to two years. Those basins are intended to control downstream erosion during frequently occurring storms. Under 100-year recurrence interval storm and planned land use conditions, the individual dual-purpose basin pond areas would range from about one acre up to 12.9 acres and the total volume of runoff stored above the permanent pond level would vary from 3.2 to 48 acre-feet. For the two dual-purpose basins designed for two-year storm control, the maximum pond areas would be 1.5 and 2.5 acres and the total volume of runoff stored above permanent pond level during a two-year storm would be 3.6 and 7.9 acre-feet. For the single-purpose dry detention basins under 100-year recurrence interval storm and planned land use conditions the individual pond areas would range from 0.5 to 5.0 acres and the total volume of runoff stored in individual basins would range from 1.7 acre-feet to 17.8 acre-feet. It is possible that the construction of some of the detention basins called for under this alternative would involve excavation or dike construction in wetland areas. Such activities may require permits from the Wisconsin Department of Natural Resources and the U. S. Army Corps of Engineers. Pertinent data regarding detention basin volumes and pond areas are given in Table 37.

In addition to detention storage, the alternative calls for 35,500 lineal feet of new storm sewers in areas of planned development and 10,500 lineal feet of new storm sewers in areas of existing development which are currently served by open channels or roadside swales. New circular reinforced concrete storm sewers range in diameter from 12 inches to 66 inches. Horizontal elliptical storm sewers range in size from 45 inches by 29 inches to 53 inches by 34 inches. Reinforced concrete pipe arch storm sewer sizes range from 36 inches by 23 inches to 58 inches by 36 inches. The alternative also includes 800

Map 11

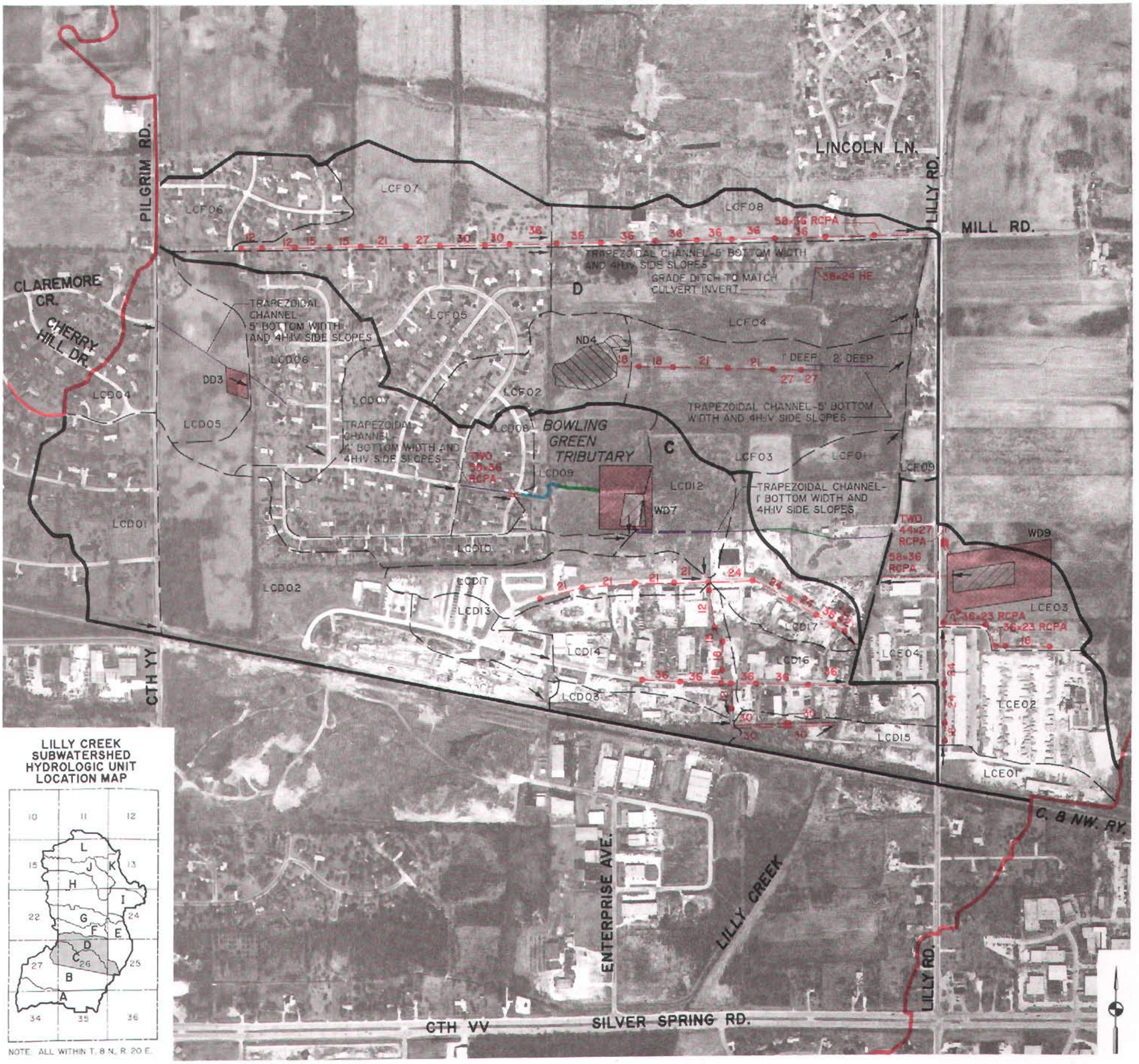
STORMWATER DRAINAGE ALTERNATIVE PLAN NO. 3: MAXIMUM DETENTION STORAGE WITH A COMBINATION OF OPEN CHANNEL AND STORM SEWER CONVEYANCE

HYDROLOGIC UNITS A AND B

Map 11 (continued)

STORMWATER DRAINAGE ALTERNATIVE PLAN NO. 3: MAXIMUM DETENTION STORAGE WITH A COMBINATION OF OPEN CHANNEL AND STORM SEWER CONVEYANCE

HYDROLOGIC UNITS C AND D



LEGEND

🟥	SUBWATERSHED BOUNDARY
▬	HYDROLOGIC UNIT BOUNDARY UNDER EXISTING DRAINAGE CONDITIONS
D	HYDROLOGIC UNIT IDENTIFICATION
— —	SUBBASIN BOUNDARY
LCF	SUBBASIN IDENTIFICATION
- - - -	CATCHMENT AREA BOUNDARY
LCF03	CATCHMENT AREA IDENTIFICATION
→	CATCHMENT AREA OUTLET UNDER EXISTING DRAINAGE CONDITIONS
▨	EXISTING NATURAL DETENTION BASIN
— —	MAINTAIN EXISTING CHANNEL
—	MAINTAIN EXISTING NATURAL CHANNEL AND PROVIDE RIPRAP ALONG STREAMBANKS AND STREAMBED
—	CHANNEL CLEANING AND DEBRUSHING WITH RIPRAP PROVIDED ALONG STREAMBANKS AND STREAMBED
▨	PERMANENT POND AREA OF PROPOSED WET DETENTION BASIN
▣	MAXIMUM POND AREA DURING THE 100-YEAR STORM UNDER PLANNED LAND USE AND PLANNED DRAINAGE CONDITIONS FOR A PROPOSED DUAL-PURPOSE DETENTION BASIN
▩	MAXIMUM POND AREA DURING THE 100-YEAR STORM UNDER PLANNED LAND USE AND PLANNED DRAINAGE CONDITIONS FOR A PROPOSED DRY DETENTION BASIN
36	PROPOSED STORM SEWER (SIZE IN INCHES)
58x36	PROPOSED REPLACEMENT STORM SEWER OR CULVERT (SIZE IN INCHES)
●	PROPOSED NEW OR REPLACEMENT MANHOLE
■	PROPOSED JUNCTION BOX
	PROPOSED TURF-LINED OPEN CHANNEL
—	PROPOSED RIPRAP-LINED OPEN CHANNEL
HE	HORIZONTAL ELLIPTICAL REINFORCED CONCRETE PIPE
RCPA	REINFORCED CONCRETE PIPE ARCH
NOTE:	PIPES ARE CONSTRUCTED OF REINFORCED CONCRETE.

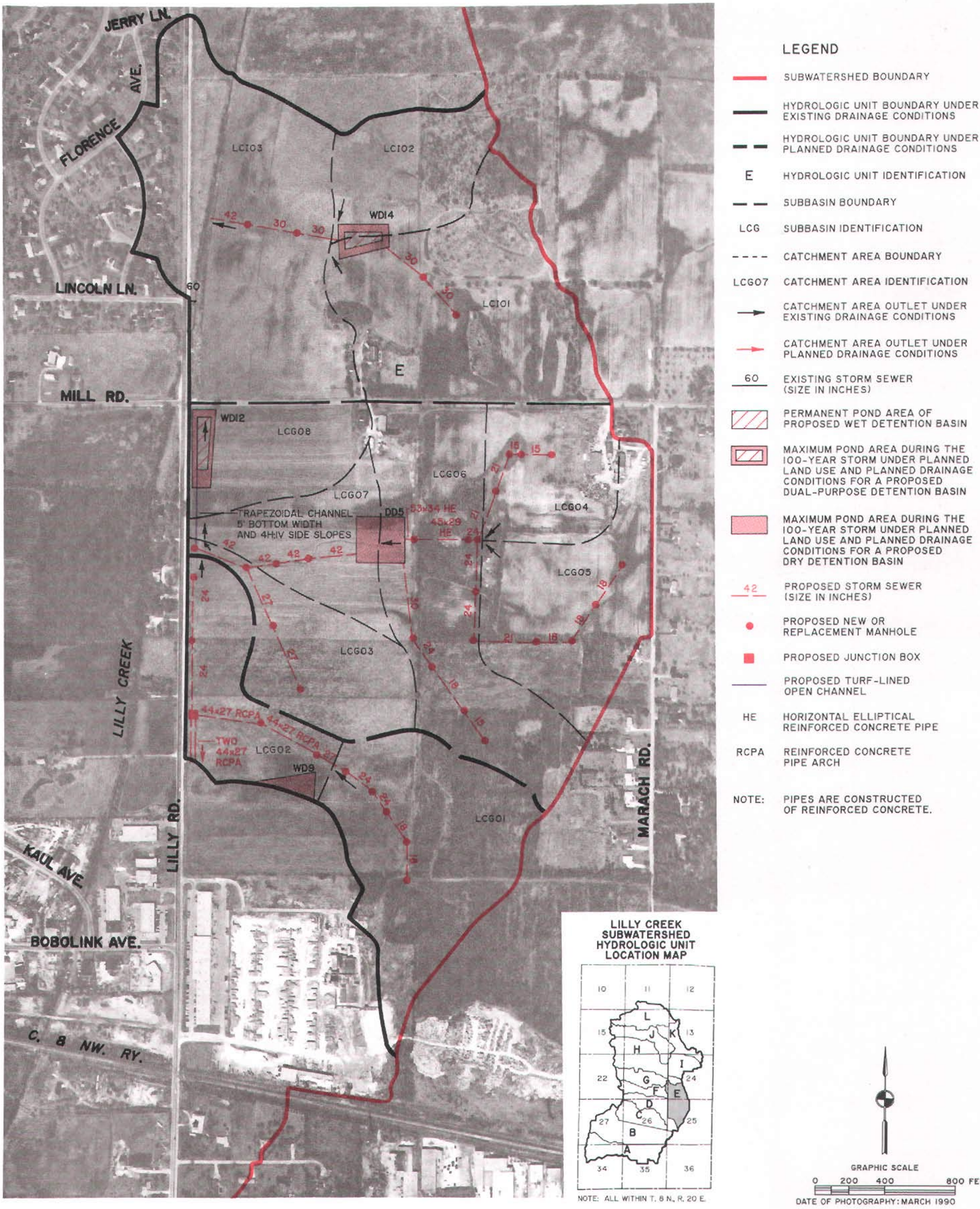
ALL MODIFIED CHANNEL REACHES ALONG NAMED TRIBUTARIES WOULD BE PROVIDED WITH A ONE-FOOT DEEP, TWO-FOOT WIDE, RIPRAP-LINED LOW-FLOW CHANNEL.

GRAPHIC SCALE
0 200 400 800 FEET
DATE OF PHOTOGRAPHY: MARCH 1990

Map 11 (continued)

STORMWATER DRAINAGE ALTERNATIVE PLAN NO. 3: MAXIMUM DETENTION STORAGE WITH A COMBINATION OF OPEN CHANNEL AND STORM SEWER CONVEYANCE

HYDROLOGIC UNIT E



Map 11 (continued)

STORMWATER DRAINAGE ALTERNATIVE PLAN NO. 3: MAXIMUM DETENTION STORAGE WITH A COMBINATION OF OPEN CHANNEL AND STORM SEWER CONVEYANCE

HYDROLOGIC UNITS F AND G

LEGEND

🟥 line	SUBWATERSHED BOUNDARY
━	HYDROLOGIC UNIT BOUNDARY UNDER EXISTING DRAINAGE CONDITIONS
G	HYDROLOGIC UNIT IDENTIFICATION
— —	SUBBASIN BOUNDARY
LCF	SUBBASIN IDENTIFICATION
- - - -	CATCHMENT AREA BOUNDARY
LCF07	CATCHMENT AREA IDENTIFICATION
→	CATCHMENT AREA OUTLET UNDER EXISTING DRAINAGE CONDITIONS
60	EXISTING STORM SEWER (SIZE IN INCHES)
●	EXISTING MANHOLE
🟩 line	MAINTAIN EXISTING NATURAL CHANNEL AND PROVIDE RIPRAP ALONG STREAMBANKS AND STREAMBED
▨	PERMANENT POND AREA OF PROPOSED WET DETENTION BASIN
▨	MAXIMUM POND AREA DURING THE 100-YEAR STORM UNDER PLANNED LAND USE AND PLANNED DRAINAGE CONDITIONS FOR A PROPOSED DUAL-PURPOSE DETENTION BASIN
42	PROPOSED STORM SEWER (SIZE IN INCHES)
●	PROPOSED NEW OR REPLACEMENT MANHOLE
——	PROPOSED TURF-LINED OPEN CHANNEL

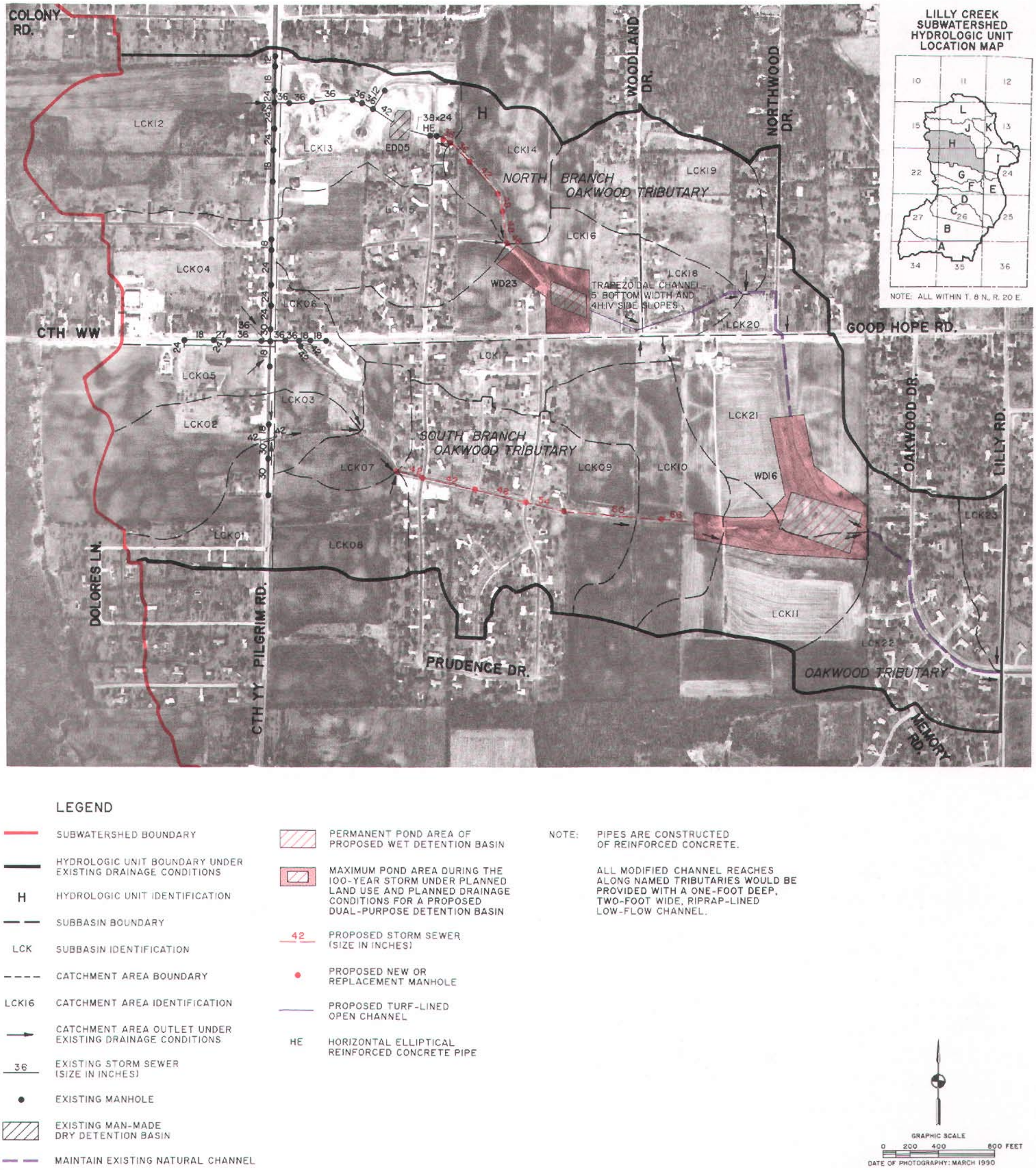
NOTE: PIPES ARE CONSTRUCTED OF REINFORCED CONCRETE.

ALL MODIFIED CHANNEL REACHES ALONG NAMED TRIBUTARIES WOULD BE PROVIDED WITH A ONE-FOOT DEEP, TWO-FOOT WIDE, RIPRAP-LINED LOW-FLOW CHANNEL.

Map 11 (continued)

STORMWATER DRAINAGE ALTERNATIVE PLAN NO. 3: MAXIMUM DETENTION STORAGE WITH A COMBINATION OF OPEN CHANNEL AND STORM SEWER CONVEYANCE

HYDROLOGIC UNIT H



LEGEND

SUBWATERSHED BOUNDARY

HYDROLOGIC UNIT BOUNDARY UNDER EXISTING DRAINAGE CONDITIONS

H HYDROLOGIC UNIT IDENTIFICATION

SUBBASIN BOUNDARY

LCK SUBBASIN IDENTIFICATION

CATCHMENT AREA BOUNDARY

LCK16 CATCHMENT AREA IDENTIFICATION

CATCHMENT AREA OUTLET UNDER EXISTING DRAINAGE CONDITIONS

36 EXISTING STORM SEWER (SIZE IN INCHES)

● EXISTING MANHOLE

▨ EXISTING MAN-MADE DRY DETENTION BASIN

MAINTAIN EXISTING NATURAL CHANNEL

▨ PERMANENT POND AREA OF PROPOSED WET DETENTION BASIN

▨ MAXIMUM POND AREA DURING THE 100-YEAR STORM UNDER PLANNED LAND USE AND PLANNED DRAINAGE CONDITIONS FOR A PROPOSED DUAL-PURPOSE DETENTION BASIN

42 PROPOSED STORM SEWER (SIZE IN INCHES)

● PROPOSED NEW OR REPLACEMENT MANHOLE

PROPOSED TURF-LINED OPEN CHANNEL

HE HORIZONTAL ELLIPTICAL REINFORCED CONCRETE PIPE

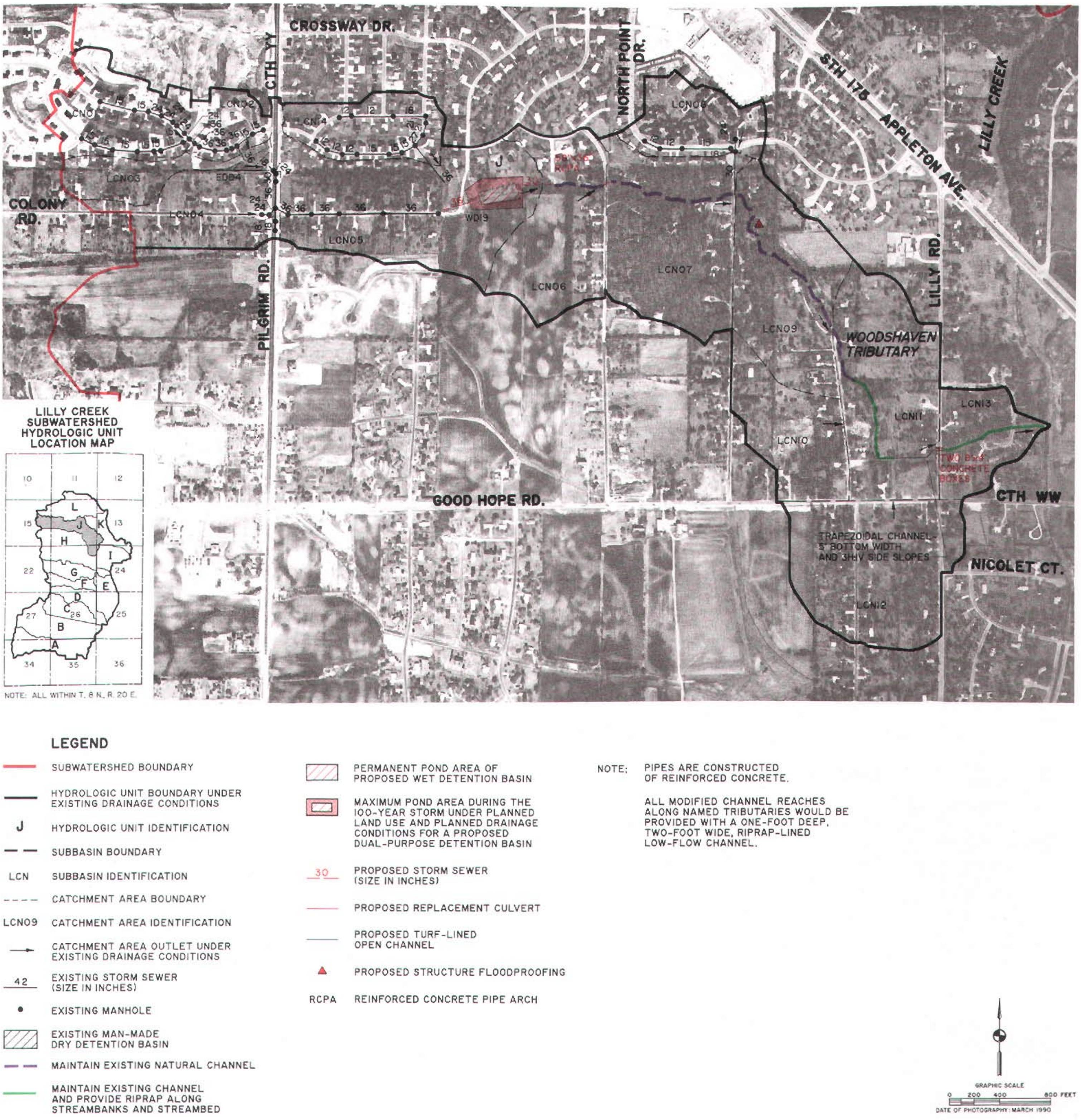
NOTE: PIPES ARE CONSTRUCTED OF REINFORCED CONCRETE.

ALL MODIFIED CHANNEL REACHES ALONG NAMED TRIBUTARIES WOULD BE PROVIDED WITH A ONE-FOOT DEEP, TWO-FOOT WIDE, RIPRAP-LINED LOW-FLOW CHANNEL.

Map 11 (continued)

STORMWATER DRAINAGE ALTERNATIVE PLAN NO. 3: MAXIMUM DETENTION STORAGE WITH A COMBINATION OF OPEN CHANNEL AND STORM SEWER CONVEYANCE

HYDROLOCIC UNIT J



LEGEND

SUBWATERSHED BOUNDARY

HYDROLOGIC UNIT BOUNDARY UNDER EXISTING DRAINAGE CONDITIONS

J HYDROLOGIC UNIT IDENTIFICATION

SUBBASIN BOUNDARY

LCN SUBBASIN IDENTIFICATION

CATCHMENT AREA BOUNDARY

LCN09 CATCHMENT AREA IDENTIFICATION

CATCHMENT AREA OUTLET UNDER EXISTING DRAINAGE CONDITIONS

42 EXISTING STORM SEWER (SIZE IN INCHES)

● EXISTING MANHOLE

▨ EXISTING MAN-MADE DRY DETENTION BASIN

MAINTAIN EXISTING NATURAL CHANNEL

MAINTAIN EXISTING CHANNEL AND PROVIDE RIPRAP ALONG STREAMBANKS AND STREAMBED

▨ PERMANENT POND AREA OF PROPOSED WET DETENTION BASIN

MAXIMUM POND AREA DURING THE 100-YEAR STORM UNDER PLANNED LAND USE AND PLANNED DRAINAGE CONDITIONS FOR A PROPOSED DUAL-PURPOSE DETENTION BASIN

30 PROPOSED STORM SEWER (SIZE IN INCHES)

PROPOSED REPLACEMENT CULVERT

PROPOSED TURF-LINED OPEN CHANNEL

▲ PROPOSED STRUCTURE FLOODPROOFING

RCPA REINFORCED CONCRETE PIPE ARCH

NOTE: PIPES ARE CONSTRUCTED OF REINFORCED CONCRETE.

ALL MODIFIED CHANNEL REACHES ALONG NAMED TRIBUTARIES WOULD BE PROVIDED WITH A ONE-FOOT DEEP, TWO-FOOT WIDE, RIPRAP-LINED LOW-FLOW CHANNEL.

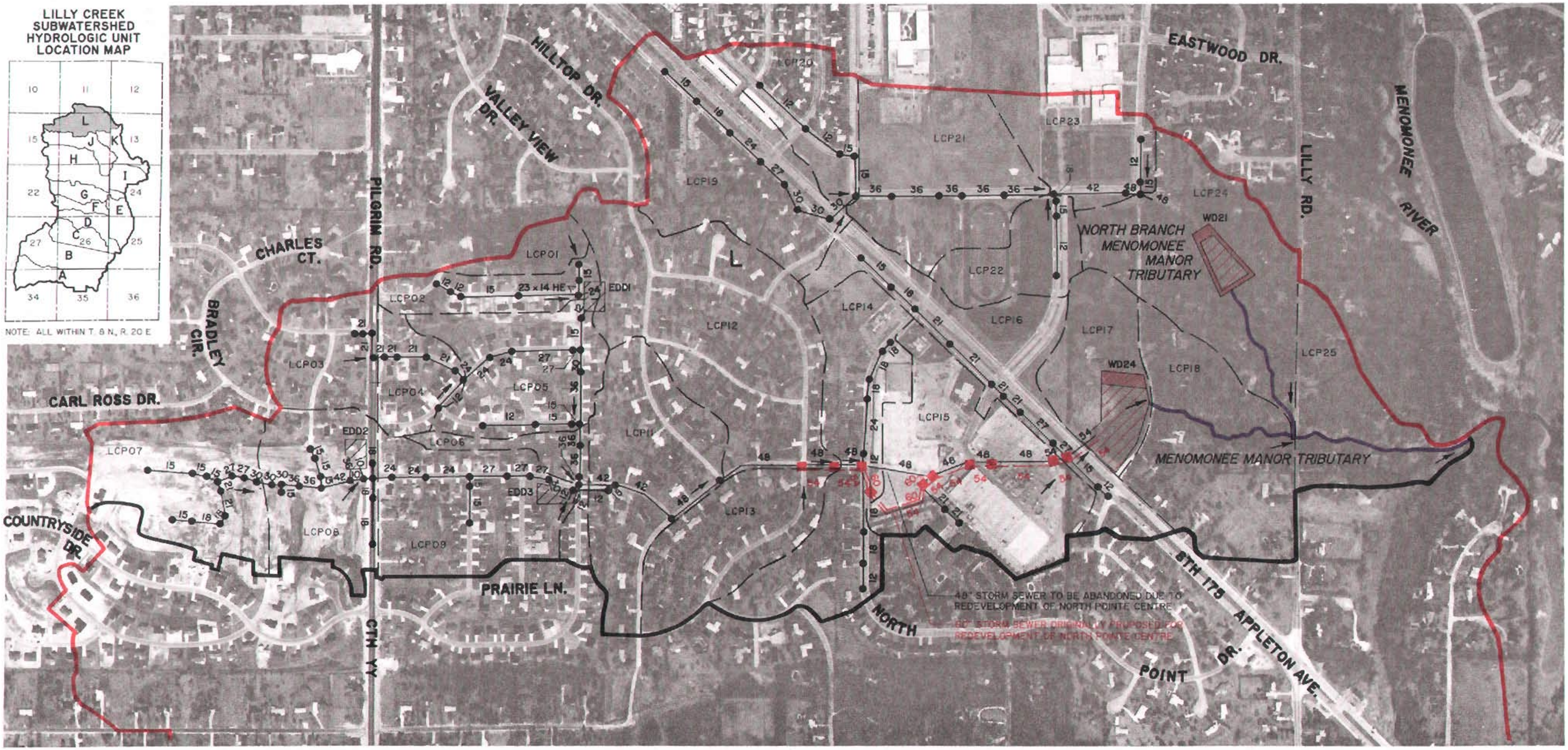
Map 11 (continued)

STORMWATER DRAINAGE ALTERNATIVE PLAN NO. 3: MAXIMUM DETENTION STORAGE WITH A COMBINATION OF OPEN CHANNEL AND STORM SEWER CONVEYANCE

HYDROLOGIC UNITS I AND K

STORMWATER DRAINAGE ALTERNATIVE PLAN NO. 3: MAXIMUM DETENTION STORAGE WITH A COMBINATION OF OPEN CHANNEL AND STORM SEWER CONVEYANCE

HYDROLOGIC UNIT L



LEGEND

🟥	SUBWATERSHED BOUNDARY
▬	HYDROLOGIC UNIT BOUNDARY UNDER EXISTING DRAINAGE CONDITIONS
L	HYDROLOGIC UNIT IDENTIFICATION
— —	SUBBASIN BOUNDARY
LCP	SUBBASIN IDENTIFICATION
- - - -	CATCHMENT AREA BOUNDARY
LCP22	CATCHMENT AREA IDENTIFICATION
→	CATCHMENT AREA OUTLET UNDER EXISTING DRAINAGE CONDITIONS
21	EXISTING STORM SEWER (SIZE IN INCHES)
●	EXISTING MANHOLE
▨	EXISTING MAN-MADE DRY DETENTION BASIN
▬	MAINTAIN EXISTING CHANNEL
▨	PERMANENT POND AREA OF PROPOSED WET DETENTION BASIN
▨	MAXIMUM POND AREA DURING THE TWO-YEAR STORM UNDER PLANNED LAND USE AND PLANNED DRAINAGE CONDITIONS FOR A PROPOSED DUAL-PURPOSE DETENTION BASIN
54	PROPOSED STORM SEWER (SIZE IN INCHES)
●	PROPOSED NEW OR REPLACEMENT MANHOLE
■	PROPOSED JUNCTION BOX
HE	HORIZONTAL ELLIPTICAL REINFORCED CONCRETE PIPE

NOTE: PIPES ARE CONSTRUCTED OF REINFORCED CONCRETE.

RED SHADED AREA FOR BASINS WD21 AND WD24 REPRESENTS THE MAXIMUM ADDITIONAL POND AREA FOR A TWO-YEAR, 24-HOUR STORM.

Source: SEWRPC.

Table 36

STORMWATER DRAINAGE ALTERNATIVE PLAN NO. 3: MAXIMUM DETENTION STORAGE WITH A COMBINATION OF OPEN CHANNEL AND STORM SEWER CONVEYANCE

Hydrologic Unit	Project and Component Description[b]	Estimated Cost[a]	
		Capital[c]	Annual Operation and Maintenance[d]
A	Silver Spring Tributary		
	1. 380 feet of 12-inch storm sewer	$ 16,000	$ 200
	2. 1,790 feet of 15-inch storm sewer	82,000	700
	3. 825 feet of 18-inch storm sewer	42,000	300
	4. 2,280 feet of 21-inch storm sewer	132,000	900
	5. 380 feet of 27-inch storm sewer	26,000	200
	6. 1,010 feet of 30-inch storm sewer	78,000	400
	7. 1,280 feet of 36-inch storm sewer	119,000	200
	8. 345 feet of 53-inch x 34-inch concrete HE storm sewer	47,000	100
	9. 3,430-foot-long, turf-lined channel with a 5-foot bottom width and 3H:1V side slopes, and a 1-foot-deep, 2-foot-wide riprap-lined low-flow channel	116,000	1,400
	10. Replace existing culverts at Fire Station with two 50-foot-long, 10-foot-wide x 3-foot-high concrete box culverts	53,000	0
	11. Replace existing culverts at Badger Drive with two 36-foot-long, 10-foot-wide x 3-foot-high concrete box culverts	38,000	0
	12. Replace existing culverts at Butternut Drive with two 36-foot-long, 10-foot-wide x 3-foot-high concrete box culverts	38,000	0
	13. Replace existing culverts at private drive with a 42-foot-long, 10-foot-wide x 3-foot-high concrete box culvert	22,000	0
	14. Replace existing culverts at Pilgrim Road with an 83-foot-long, 10-foot-wide x 3-foot-high concrete box culvert	44,000	0
	15. Replace existing culverts at Bette Drive with a 36-foot-long, 7-foot-wide x 3-foot-high concrete box culvert	17,000	0
	16. Replace existing culverts at Private Drive with a 28-foot-long, 7-foot-wide x 3-foot-high concrete box culvert	13,000	0
	17. Remove private drive	1,000	- -
	18. Detention basin WD1 flood control storage volume of 27.1 acre-feet	302,000	6,900
	Subtotal	$1,186,000	$11,300
B	Phillips Tributary		
	1. 510 feet of 18-inch storm sewer	$ 26,000	$ 200
	2. 370 feet of 21-inch storm sewer	21,000	100
	3. 230 feet of 24-inch storm sewer	14,000	100
	4. 440 feet of 27-inch storm sewer	31,000	200
	5. 150 feet of 30-inch storm sewer	12,000	100
	6. 700 feet of 36-inch storm sewer	65,000	100
	7. Replace existing culverts at Pilgrim Road with three 40-foot-long, 8-foot-wide x 4-foot-high concrete box culverts	66,000	0
	8. Construct 475-foot-long trapezoidal turf-lined channel with a 10-foot bottom width, a 3H:1V side slopes, and a 1-foot-deep, 2-foot-wide riprap-lined low-flow channel	29,000	200
	9. Floodproof one house	5,000	0
	10. Detention basin WD2 flood control storage volume of 19.2 acre-feet	260,000	5,700
	11. Detention basin WD4 flood control storage volume of 4.3 acre-feet	120,000	1,700
	12. Detention basin DD1 Flood control storage volume of 15.9 acre-feet	260,000	6,900
	Subtotal	$ 909,000	$15,300

Table 36 (continued)

Hydrologic Unit	Project and Component Description[b]	Estimated Cost[a]	
		Capital[c]	Annual Operation and Maintenance[d]
C	Bowling Green Tributary		
	1. 520 feet of 12-inch storm sewer	$ 22,000	$ 200
	2. 110 feet of 15-inch storm sewer	5,000	0
	3. 290 feet of 18-inch storm sewer	15,000	100
	4. 1,205 feet of 21-inch storm sewer	84,000	500
	5. 795 feet of 24-inch storm sewer	58,000	300
	6. 1,460 feet of 30-inch storm sewer	112,000	600
	7. 1,510 feet of 36-inch storm sewer	159,000	300
	8. 185 feet of 42-inch storm sewer	23,000	0
	9. 1,245-foot-long (620 feet to be riprapped and 625 feet to be turf-lined) trapezoidal, turf-lined channel with 1-foot bottom width and 4H:1V side slopes	98,000	500
	10. Channel cleaning and debrushing	1,000	200
	11. 84 feet total of 58-inch-wide x 36-inch-high RCPA	12,000	0
	12. 340-foot-long trapezoidal, turf-lined channel with 4-foot bottom width and 4H:1V side slopes	3,000	200
	13. 860-foot-long trapezoidal, turf-lined channel with 5-foot bottom width and 4H:1V side slopes	9,000	400
	14. Riprap erosion protection along 0.12 mile of existing stream	23,000	300
	15. Detention basin WD7 flood control storage volume of 11.0 acre-feet	187,000	4,200
	16. Detention basin DD3 flood control storage volume of 1.7 acre-feet	40,000	1,600
	Subtotal	$ 847,000	$ 9,400
D	Area Predominantly West of Lilly Creek and North and South of W. Mill Road		
	1. 440 feet of 12-inch storm sewer	$ 22,000	$ 200
	2. 430 feet of 15-inch storm sewer	23,000	200
	3. 1,255 feet of 18-inch storm sewer	64,000	500
	4. 1,005 feet of 21-inch storm sewer	62,000	400
	5. 1,780 feet of 24-inch storm sewer	111,000	700
	6. 885 feet of 27-inch storm sewer	63,000	400
	7. 475 feet of 30-inch storm sewer	42,000	200
	8. 2,170 feet of 36-inch storm sewer	229,000	400
	9. 540 feet of 36-inch x 23-inch RCPA storm sewer	52,000	200
	10. 1,815 feet of 44-inch x 27-inch RCPA storm sewer	196,000	300
	11. 1,165 feet of 58-inch x 36-inch RCPA storm sewer	203,000	200
	12. 60 feet of 38-inch x 24-inch concrete HE storm sewer	6,000	0
	13. 135-foot-long trapezoidal, turf-lined channel with 5-foot bottom width, 4H:1V side slopes, and 1-foot average depth	2,000	100
	14. 440-foot-long trapezoidal, turf-lined channel with 5-foot bottom width, 4H:1V side slopes, and 2-foot average depth	7,000	200
	15. 250-foot-long trapezoidal turf-lined channel with 5-foot bottom width, 4H:1V side slopes, and 1.25-foot average depth	3,000	100
	16. Detention basin WD9 flood control storage volume of 13.9 acre-feet	215,000	4,800
	Subtotal	$1,300,000	$ 8,800
E	Area East of Lilly Creek and North and South of W. Mill Road		
	1. 420 feet of 15-inch storm sewer	$ 19,000	$ 200
	2. 950 feet of 18-inch storm sewer	49,000	400
	3. 860 feet of 21-inch storm sewer	50,000	300
	4. 835 feet of 24-inch storm sewer	52,000	300
	5. 750 feet of 27-inch storm sewer	52,000	300
	6. 1,415 feet of 30-inch storm sewer	109,000	600

Table 36 (continued)

Hydrologic Unit	Project and Component Description[b]	Estimated Cost[a]	
		Capital[c]	Annual Operation and Maintenance[d]
E	Area East of Lilly Creek and North and South of W. Mill Road (continued)		
	7. 1,295 feet of 42-inch storm sewer	$ 145,000	$ 200
	8. 330 feet of 45-inch x 29-inch HE storm sewer	36,000	100
	9. 40 feet of 53-inch x 34-inch concrete HE storm sewer	5,000	0
	10. 435-foot-long trapezoidal, turf-lined channel with 5-foot bottom width, and 4H:1V side slopes	30,000	200
	11. Detention basin WD12 flood control storage volume of 7.1 acre-feet	117,000	2,700
	12. Detention basin WD14 flood control storage volume of 3.2 acre-feet	49,000	1,300
	13. Detention basin DD5 flood control storage volume of 5.9 acre-feet	109,000	3,200
	Subtotal	$ 822,000	$ 9,800
F	Lincoln Lane Tributary		
	1. 655 feet of 18-inch storm sewer	$ 34,000	$ 300
	2. 710 feet of 27-inch storm sewer	49,000	300
	3. 330 feet of 30-inch storm sewer	25,000	100
	4. 1,120 feet of 36-inch storm sewer	104,000	200
	5. 1,240 feet of 42-inch storm sewer	139,000	200
	6. Detention basin WD13 flood control storage volume of 8.1 acre-feet	136,000	3,100
	Subtotal	$ 487,000	$ 4,200
G	Jerry Lane Tributary		
	1. 290 feet of 18-inch storm sewer	$15,000	$100
	2. 200 feet of 27-inch storm sewer	14,000	100
	3. 90 feet of 30-inch storm sewer	7,000	0
	4. Retain existing channel through isolated natural area. Riprap erosion protection along 0.53 mile of stream	100,000	1,100
	5. 920-foot-long trapezoidal, turf-lined channel with 3-foot bottom width, 3H:1V side slopes, and a 1-foot-deep, 2-foot-wide, riprap-lined, low-flow channel	43,000	400
	6. Detention basin WD15 flood control storage volume of 13.1 acre-feet	304,000	4,600
	7. Detention basin WD22 flood control storage volume of 4.8 acre-feet	80,000	1,800
	Subtotal	$ 563,000	$ 8,100
H	Oakwood Tributary		
	South Branch		
	1. 340 feet of 36-inch storm sewer	$ 32,000	$ 100
	2. 970 feet of 42-inch storm sewer	109,000	200
	3. 755 feet of 48-inch storm sewer	97,000	200
	4. 250 feet of 54-inch storm sewer	39,000	0
	5. 680 feet of 60-inch storm sewer	124,000	100
	6. 160 feet of 66-inch storm sewer	35,000	0
	North Branch		
	7. 930-foot-long trapezoidal, turf-lined channel with 5-foot bottom width, 4H:1V side slopes, and 1-foot-deep, 2-foot-wide, riprap-lined low-flow channel	56,000	500
	8. Detention basin WD23 flood control storage volume of 17.8 acre-feet	248,000	3,600
	North and South Branches		
	9. Detention *basin* WD16 flood control storage volume of 48.2 acre-feet	524,000	9,900
	Subtotal	$1,264,000	$14,600

Table 36 (continued)

Hydrologic Unit	Project and Component Description[b]	Estimated Cost[a]	
		Capital[c]	Annual Operation and Maintenance[d]
I	Area East of Lilly Road and South of W. Good Hope Road		
	1. 3,100 feet of 2-foot-deep roadside swale with driveway culverts	$ 65,000	$ 1,900
	Subtotal	$ 65,000	$ 1,900
J	Woodshaven Tributary		
	1. 72 feet total of double 8-foot-wide x 4-foot-high concrete box culvert	$ 40,000	$ 0
	2. 40 feet of 58-inch-wide x 36-inch-high RCPA culvert	7,000	0
	3. 245 feet of 30-inch storm sewer	19,000	100
	4. 340 feet of 36-inch storm sewer	32,000	100
	5. 350-foot-long trapezoidal, turf-lined channel with 5-foot bottom width, 3H:1V side slopes, and a 1-foot-deep, 2-foot-wide, riprap-lined low-flow channel	39,000	200
	6. Riprap along 0.25 mile of existing stream	24,000	500
	7. Floodproof one house	5,000	0
	8. Detention basin WD19 flood control storage volume of 12.1 acre-feet	193,000	4,500
	Subtotal	$ 359,000	$ 5,400
K	Area along W. Appleton Avenue and East and West of Lilly Creek		
	1. Replace existing 36-inch storm sewer in Appleton Avenue with 566 feet of 42-inch storm sewer	$ 71,000	$ 100
	2. Replace existing 30-inch storm sewer in Appleton Avenue with 94 feet of 36-inch storm sewer	10,000	0
	3. Replace existing 27-inch storm sewer in Appleton Avenue with 142 feet of 30-inch storm sewer	12,000	100
	4. Floodproof one house	5,000	0
	5. 115 feet of 58-inch-wide x 36-inch-high RCPA storm sewer	19,000	0
	6. 1,315 feet of 36-inch storm sewer	122,000	300
	Subtotal	$ 239,000	$ 500
L	Menomonee Manor Tributary		
	1. 1,986 feet of 54-inch storm sewer parallel to existing and committed storm sewers	$ 341,000	$ 400
	2. Riprap erosion protection along 170 feet of stream	6,000	100
	3. Detention basin WD21 two-year storm control storage volume of 3.6 acre-feet	61,000	1,300
	4. Detention basin WD24 two-year storm control storage volume of 7.9 acre-feet	120,000	4,500
	Subtotal	$ 528,000	$6,300
Total		$8,569,000	$95,600

NOTE: The following abbreviations were used in this table:

HE - Horizontal Elliptical
RCPA - Reinforced Concrete Pipe Arch

[a] *Capital and operation and maintenance costs for dual-purpose water quantity and quality detention basins only reflect that portion of the total cost which is assignable to the water quantity control function, including two-year storm control.*

[b] *All new and replacement sewers are concrete pipe.*

[c] *Capital costs include 35 percent for engineering, administration, and contingencies.*

[d] *Costs were noted to be zero when the project proposed replacement of a component with a component which has similar operation and maintenance costs, or when the annual operation and maintenance cost was estimated to be less than $50.*

Source: SEWRPC.

Table 37

STORMWATER DRAINAGE ALTERNATIVE PLAN NO. 3: DUAL-PURPOSE DETENTION BASIN AND DRY DETENTION BASIN POND AREAS AND VOLUMES

Basin Designation	Permanent Pond Area (acres)	Flood Control Pond Area (acres)[a]	Permanent Pond Volume (acre-feet)	Flood Control Pond Volume[a] (acre-feet)	Total Maximum Pond Volume (acre-feet)
WD1	2.9	5.4	14.6	27.1	41.7
WD2	1.5	4.3	7.4	19.2	26.6
WD4	0.5	1.6	2.7	4.3	7.0
WD7	0.8	3.6	3.8	11.0	14.8
WD9	1.5	6.5	7.3	13.9	21.2
WD12	0.9	1.4	4.3	7.1	11.4
WD13	1.1	1.8	5.4	8.1	13.5
WD14	0.4	1.0	1.8	3.2	5.0
WD15	1.4	2.3	7.2	13.1	20.3
WD16	3.7	12.9	18.6	48.2	66.8
WD19	0.8	2.7	4.0	12.1	16.1
WD21	1.1	1.5	5.2	3.6	8.8
WD22	0.4	1.9	2.2	4.8	7.0
WD23	1.2	5.0	5.8	17.8	23.6
WD24	2.1	2.5	10.5	7.9	18.4
DD1	- -	2.8	- -	15.9	15.9
DD3	- -	0.5	- -	1.7	1.7
DD5	- -	1.7	- -	5.9	5.9
Total	20.3	59.4	100.8	224.9	325.7

[a] *All flood control areas and volumes are for a 100-year recurrence interval, 24-hour storm under planned land use and drainage conditions, except for the areas and volumes for WD21 and WD24. These two basins are designed to control a two-year recurrence interval, 24-hour storm under planned land use and drainage conditions.*

Source: SEWRPC.

lineal feet of replacement storm sewer in areas of existing development. Replacement circular storm sewer sizes range from 27 inches to 42 inches.

The alternative includes 3,100 feet of roadside swales in areas of planned residential development. A total of about 3,100 feet of new grass-lined open channels would also be provided. In addition, modifications including widening and/or deepening would be made along 10,300 feet of existing open channels and a 1,245-foot-long diversion channel would be constructed. Modifications to existing "natural" streams would include a riprap-lined low-flow channel. As shown on Map 11, this alternative would also maintain 4.0 miles of stream channel tributary to Lilly Creek.

Under this alternative, three buildings located along tributaries to Lilly Creek would be floodproofed.

A total of 10 existing road crossings and pedestrian bridges would be replaced, while five pedestrian bridges would be removed and not replaced. Also, one new road crossing would be constructed.

Through the inclusion of extended detention storage of frequently occurring flows and flood control storage at large flows, this alternative would provide substantial protection from streambank erosion along Lilly Creek and its tributaries. In those reaches where the provision of extended detention storage is not feasible, riprap erosion protection is recommended for the existing streambed and streambanks.

It was found that this stormwater drainage alternative could be implemented without significantly modifying the Lilly Creek channel; however, the alternative addresses only the provision of adequate major and minor system drainage facilities for the areas tributary to Lilly Creek. A detailed analysis of alternatives to address flood control in areas adjoining Lilly Creek itself is presented in a subsequent section of this chapter.

EVALUATION OF ALTERNATIVE STORMWATER DRAINAGE PLANS

The preceding sections described the three alternative stormwater drainage system plans considered for the Lilly Creek subwatershed. The information presented was intended to provide a basis for a comparative evaluation of the alternative plans. Each alternative was designed to resolve the identified existing drainage problems, as well as to serve anticipated future development within the subwatershed under planned land use conditions. The advantages and disadvantages of each alternative are summarized in Table 38.

For each hydrologic unit within the planning area, Table 39 compares the capital costs, the annual operation and maintenance costs, and the present value of the cost of each alternative. A comparison of the ability of each alternative plan to meet the recommended stormwater management objectives and supporting standards is provided in Table 40 for those objectives and standards which differ between the plans in level of achievement.

Stormwater Drainage Alternative Plan No. 1: Storm Sewer Conveyance with Selected Open Channel Conveyance and Existing Detention Storage

Under this alternative plan, storm sewers and open channels, supplemented by existing natural and man-made detention storage areas, would convey stormwater runoff to receiving surface watercourses in the Lilly Creek subwatershed. The alternative would entail a capital cost of about $13.81 million and an incremental average annual operation and maintenance cost of about $31,400 and an equivalent annual cost of about $908,000.

For the planning area as a whole, this alternative has the highest capital and equivalent annual costs of the three alternatives considered; however, the annual operation and maintenance cost of the storm sewer alternative is the lowest. In Hydrologic Unit L, an area with a large amount of existing development, the components of the storm sewer conveyance alternative are also proposed under the open channel alternative. The capital cost and the operation and maintenance cost of the storm sewer alternative for Hydrologic Unit L are somewhat less than the corresponding costs of the maximum detention alternative for that hydrologic unit.

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

Also, this alternative would require less floodproofing of buildings along streams tributary to Lilly Creek than would the open channel alternative. Minimizing the number of buildings to be floodproofed increases the implementability of the floodproofing component of the alternative.

The major disadvantage of the storm sewer conveyance alternative plan is the high capital cost. Another significant disadvantage is that in some areas downstream peak discharges may be expected to be higher than existing discharges and also significantly higher than discharges under the maximum detention alternative. Those higher peak discharges would necessitate more extensive and costly flood control measures along Lilly Creek. Other disadvantages include a relatively low level of nonpoint source pollution removal, the lack of any multipurpose benefits, and the loss of some marginal aquatic and riparian habitat through channel enclosure.

Most of the agreed-upon stormwater management objectives could be met by the storm sewer conveyance alternative plan, although a lower level of aquatic and terrestrial habitat protection would be provided than under the open channel or maximum-detention alternatives. This alternative could achieve the established water quality objective through integration with the

Table 38

SUMMARY OF PRINCIPAL COMPONENTS AND ADVANTAGES AND DISADVANTAGES OF ALTERNATIVE STORMWATER DRAINAGE PLANS FOR THE LILLY CREEK SUBWATERSHED

Alternative	Principal Components	Capital	Annual Operation and Maintenance	Equivalent Annual Cost[a]	Advantages	Disadvantages
No. 1—Storm Sewer Conveyance with Selected Open Channel Conveyance and Existing Detention Storage	56,000 feet of storm sewer areas of planned development 16,300 feet of new storm sewer in areas of existing development 1,590 feet of replacement storm sewer in areas of existing development 5,680 feet of channel enclosure (pipe length is 13,510 feet) 8,320 feet of engineered open channel Utilization of existing natural and man-made detention basins 2.2 miles of existing tributary streams	$13,814,000	$31,400	$908,000	Stormwater drainage components are acceptable and well known to the public; minimal operation and maintenance is required. Use of existing natural and man-made detention and retention basins limits peak discharges and flow volumes downstream from several areas of planned development. Retains some existing stream length. Reduces the need for structure floodproofing in comparison to Alternative No. 2	Peak discharges and flow volumes are increased downstream from some areas of planned development; some public officials and citizens may oppose high capital cost; channel enclosure reduces available aquatic and riparian habitat
No. 2—Open Channel Conveyance with Selected Storm Sewer Conveyance and Existing Detention Storage	21,300 feet of roadside swales in areas of planned development 22,600 feet of engineered channel 24,800 feet of storm sewer in areas of planned development 13,900 feet of new storm sewers in areas of existing development 800 feet of new storm sewers in areas of existing development Utilization of existing natural and man-made detention basins 4.3 miles of existing tributary streams	$ 7,367,000	$39,900	$507,000	Storm-sewer drainage components are acceptable and well known to the public. Roadside swales are currently used in areas of suburban- and low-density residential development and their use in planned areas of such development should be acceptable to the public and to public officials. Use of existing natural and man-made detention basins limits peak discharges and flow volumes downstream from several areas of planned development. Roadside swales have some effect on reducing nonpoint source pollutant loadings. Maintains existing streams to the greatest degree possible	Peak discharges and flow volumes are increased downstream from some areas of planned development; roadside swales in areas of new medium-density residential development may not be acceptable to the public or to public officials because of hydraulic and maintenance considerations and, in some cases, the need for obtaining drainage easements beyond standard street rights-of-way. Requirement of floodproofing of seven structures along tributary streams may pose problems regarding implementation
No.3—Maximum Detention Storage with a Combination of Open Channel and Storm Sewer Conveyance	Fifteen new dual-purpose wet detention basins Three new single-purpose dry detention basins Utilization of existing natural detention basins 3,100 feet of roadside swales 14,600 feet of engineered open channel 35,500 feet of storm sewer in areas of planned development 10,500 feet of new storm sewers in areas of existing development 800 feet of replacement storm sewer in areas of existing development	$ 8,569,000	$95,600	$640,000	Minimizes future increases in peak discharges and areas of inundation along Lilly Creek and its tributaries; reduces the required size and resultant cost of some downstream conveyance systems; reduces the potential for increased streambank erosion and streambed scour under planned conditions; minimizes the need for structural erosion protection along streambanks; basins for water quantity control can be readily supplemented with wet detention basins which would achieve a high level of reduction in pollutant loadings from nonpoint sources; reduces the need for structure floodproofing in comparison to Alternative No. 2. Floodproofing requirements are similar to Alternative No. 1	Maintenance requirements are substantial; land requirements are considerably greater than under the conveyance alternatives; some public officials and citizens may oppose ponded water in urban areas

[a] *Equivalent annual cost computations assume a 50-year life and 6 percent annual interest.*

Source: SEWRPC.

Table 39

COSTS OF ALTERNATIVE STORMWATER DRAINAGE PLANS FOR THE LILLY CREEK SUBWATERSHED IN THE VILLAGE OF MENOMONEE FALLS

Hydrologic Unit	Estimated Cost—Plan Year 2010 Land Use Conditions								
	Stormwater Drainage Alternative No. 1 Storm Sewer Conveyance with Selected Open Channel Conveyance and Existing Detention Storage			Stormwater Drainage Alternative No. 2 Open Channel Conveyance with Selected Storm Sewer Conveyance and Existing Detention Storage			Stormwater Drainage Alternative No. 3 Maximum Detention Storage with a Combination of Open Channel and Storm Sewer Conveyance		
	Capital	Annual Operation and Maintenance	Equivalent Annual Cost[a]	Capital	Annual Operation and Maintenance	Equivalent Annual Cost[a]	Capital	Annual Operation and Maintenance	Equivalent Annual Cost[a]
A	$ 3,401,000	$ 4,100	$220,000	$1,388,000	$ 4,600	$ 93,000	$1,186,000	$11,300	$ 87,000
B	1,070,000	3,300	71,000	1,011,000	3,200	67,000	909,000	15,300	73,000
C	1,192,000	3,600	79,000	694,000	4,500	49,000	889,000	9,800	63,000
D	1,232,000[b]	4,300	83,000	1,221,000[b]	4,800	82,000	1,300,000[b]	8,800	91,000
E	1,115,000	3,000	74,000	299,000	9,100	28,000	822,000	9,800	62,000
F	549,000	1,500	36,000	668,000	1,700	44,000	487,000	4,200	35,000
G	440,000	2,200	30,000	401,000	2,100	28,000	563,000	8,100	44,000
H	3,536,000	3,900	228,000	652,000	2,900	44,000	1,264,000	14,600	95,000
I	142,000	400	9,000	65,000	1,900	6,000	65,000	1,900	6,000
J	409,000	2,600	29,000	381,000	2,800	27,000	359,000	5,400	28,000
K	239,000	400	16,000	98,000	200	6,000	239,000	500	16,000
L	489,000	2,100	33,000	489,000	2,100	33,000	528,000	6,300	40,000
Total	$13,814,000	$31,400	$908,000	$7,367,000	$39,900	$507,000	$8,569,000	$95,600	$640,000

[a] *Equivalent annual cost computations assume 50-year life and 6 percent annual interest.*

[b] *Includes subbasins LCG01 and LCG02 which are in Hydrologic Unit E under existing drainage conditions.*

Source: SEWRPC.

preferred nonpoint source pollution control alternative, which is set forth in a subsequent section of this chapter. Based on the cost analyses and other considerations, it was concluded that the storm sewer conveyance alternative plan facility components should be considered further for Hydrologic Unit L in the synthesis of a preliminary recommended stormwater drainage plan.

Stormwater Drainage Alternative Plan No. 2: Open Channel Conveyance with Selected Storm Sewer Conveyance and Existing Detention Storage

Under this alternative plan, storm sewers, roadside swales, and open channels, supplemented by existing natural and man-made detention storage areas, would convey stormwater runoff to receiving surface watercourses. The alternative would entail a capital cost of about $7.37 million, an average annual operation and maintenance cost increase of about $39,900, and have an equivalent annual cost of $507,000.

For the planning area as a whole, the open channel alternative has the lowest capital cost and the second lowest operation and maintenance costs of the three alternatives. In Hydrologic Units B, C, D, E, G, H, I, J, and K, the present value cost of the open channel alternative is lower than, or equal to, the present value cost of the other alternatives.

When compared to the other alternative system plans, the advantages of the open channel alternative plan include low capital cost, probable acceptance by local officials and citizens when applied in areas of suburban- and low-density residential development, a reduction in nonpoint source pollutant loadings due to infiltration and filtering, and some reduction in streambank erosion and streambed scour due to a decrease in runoff volumes and peak flow rates during storms occurring more frequently.

A significant disadvantage of the alternative is that, in some areas, downstream peak discharges during large storms may be expected to

Table 40

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

Stormwater Management Objective[a]	Supporting Standards	Alternative No. 1 Storm Sewer Conveyance with Selected Open Channel Conveyance and Existing Detention Storage	Alternative No. 2 Open Channel Conveyance with Selected Storm Sewer Conveyance and Existing Detention Storage	Alternative No. 3 Maximum Detention Storage with a Combination of Open Channel and Storm Sewer Conveyance
Objective No. 2 The development of an integrated stormwater management and flood control system which will effectively serve existing and planned land uses and will promote implementation of the adopted land use plan	1. Stormwater drainage systems should be designed assuming that the layout of collector and land access streets for proposed urban development and redevelopment will be carefully adjusted to the topography in order to minimize grading and drainage problems, to utilize to the fullest extent practicable the natural drainage and storage capabilities of the site; and to provide the most economical installation of a gravity flow drainage system. Generally, drainage systems should be designed to complement a street layout wherein collector streets follow valley lines and land access streets cross contour lines at right angles	Partially met; some natural storage capabilities may be lost	Partially met; some natural storage capabilities may be lost	Met
	4. Stormwater management system shall be designed to minimize the creation of new drainage or flooding problems, or the intensification of existing problems, at both upstream and downstream locations	Can be met but may require additional downstream drainage facilities to address potential flooding problems because of increased flows	Can be met but may require additional downstream drainage facilities to address potential flooding problems because of increased flows	Met
Objective No. 4 The development of a stormwater management and flood control system which will abate nonpoint source water pollution and help achieve the recommended water use objectives and supporting water quality standards for surface water bodies	1. Stormwater management and flood control facilities should not impede the achievement of existing water use objectives and supporting water quality standards for lakes, streams, and wetlands, nor degrade existing habitat conditions for fish and aquatic life	Partially met through combination with alternative nonpoint source control measures	Partially met through combination with alternative nonpoint source control measures	Substantially met through combination with alternative nonpoint source control measures and through provision of extended detention storage for control of streambed erosion and streambank scour
Objective No. 6 The development of a stormwater management system which will efficiently and effectively meet all of the other stated objectives at the lowest practicable cost	1. The sum of stormwater management system capital investment and operation and maintenance costs should be minimized	Partially met; this alternative has the lowest total present value cost for one of the 12 hydrologic units	Partially met; this alternative has the lowest total present value cost for nine of the 12 hydrologic units	Partially met; this alternative has the lowest total present value cost for two of the 12 hydrologic units
	2. Maximum feasible use should be made of all existing stormwater management components, as well as the natural storm drainage system. The latter should be supplemented with engineered facilities only as necessary to serve the anticipated stormwater management needs generated by existing and proposed land use development and redevelopment	Partially met; would not use all components of natural drainage system	Partially met; would not use all components of natural drainage system, but would use more than Alternative No. 1	Partially met; would not use all components of natural drainage system, but would use more than Alternative No. 1
	4. To the maximum extent practicable, the location and alignment of new storm sewers and engineered channels and storage facilities should coincide with existing public rights-of-way to minimize land acquisition or easement costs	Can be met	Can be met, but roadside swale construction could require obtaining easements outside the standard street right-of-way	Partially met; 17 detention basins would be located on property which is currently privately owned
	5. Stormwater storage facilities, consisting of retention facilities and of both centralized and onsite detention facilities, should, where hydraulically feasible and economically sound, be considered as a means of reducing the size and resultant costs of the required stormwater conveyance facilities immediately downstream of these storage sites	Partially met through utilization of natural detention basins in existing wetlands	Partially met through utilization of natural detention basins in existing wetlands	Met
Objective No. 7 The development of a stormwater management and flood control system which will maintain or enhance existing terrestrial, riparian, and aquatic biological communities, including fish and wildlife	2. Stormwater management and flood control facilities should be designed to control sedimentation in receiving streams and to prevent the loss of fish and aquatic life habitat through streambank erosion and streambed scour	Partially met through utilization of existing natural and manmade detention storage, through combination with alternative nonpoint source control measures, and through proposed riprap erosion protection along bed and banks	Partially met through utilization of existing natural and manmade detention storage, through combination with alternative nonpoint source control measures, and through proposed riprap erosion protection along bed and banks	Met through utilization of existing natural detention storage, through the provision of extended detention storage, through combination with alternative nonpoint source control measures, and through limited riprap erosion protection along bed and banks

[a] *The stormwater management objectives and supporting standards are set forth in Table 21 in Chapter IV of this report. This table compares only those objectives and supporting standards which differed in the degree to which they are met by the alternatives.*

Source: SEWRPC.

be higher than existing discharges and higher than discharges under the maximum detention alternative. Those higher peak discharges would necessitate more extensive and costly flood control measures along Lilly Creek. Other disadvantages include potential safety hazards, relatively high maintenance costs, the lack of any multipurpose benefits, and difficulties in adapting such a system to areas of medium- and high-density development where right-of-way is limited and driveway culverts would be closely spaced. Also, because this alternative would require the greatest amount of floodproofing of buildings along streams tributary to Lilly Creek, the likelihood of full implementation of the floodproofing component is decreased.

Most of the agreed-upon stormwater management objectives could be met by this alternative. Based on the cost analyses and other considerations, it was concluded that open channel alternative components should be considered further for Hydrologic Units B, C, D, E, G, H, I, J, and K, in the synthesis of a preliminary recommended stormwater drainage plan.

Stormwater Drainage Alternative Plan No. 3: Maximum Detention Storage with a Combination of Open Channel and Storm Sewer Conveyance

This alternative plan provides for the construction of 18 new detention basins and the maintenance of four existing natural detention basins and five existing man-made dry basins. The alternative would entail a capital cost of about $8.57 million, an annual operation and maintenance cost increase of about $95,600, and an equivalent annual cost of $640,000.

For the subwatershed, the capital and equivalent annual costs of this alternative are greater than the open channel alternative and less than the storm sewer alternative. The annual operation and maintenance cost is approximately 2.4 times that of the open channel conveyance alternative, and about 3.0 times that of the storm sewer alternative. Combining the maximum detention capital and operation and maintenance costs yields an equivalent annual cost about 30 percent less than that of the storm sewer alternative and about 26 percent more than the open channel alternative. The maximum detention alternative has the lowest equivalent annual cost of the four alternatives for Hydrologic Units A and F.

The advantages of the maximum detention alternative include the significant reduction of peak rates of discharge and control of the more frequent storms which determine stream channel size and shape through streambank erosion and streambed scour. Also, this alternative would require less floodproofing of buildings along tributaries to Lilly Creek. Limiting the number of buildings increases the likelihood that the floodproofing component of the plan would be fully implemented.

Disadvantages of the maximum detention alternative include the increased land area required for the proposed detention facilities, and, in some cases, higher costs in comparison to the conveyance alternatives. However, those higher costs would be offset to some degree by the need for less extensive and less costly flood control measures along Lilly Creek with the proposed detention basins in place.

Most stormwater management objectives could be met by the maximum detention alternative plan. Based on the cost analyses and other considerations, it was concluded that components of this alternative should be considered further for all hydrologic units in the preparation of a recommended plan.

Selection of the Preliminary Recommended Alternative Stormwater Drainage Plan for the Lilly Creek Subwatershed

Drainage Alternative Plan No. 3: Maximum Detention Storage with a Combination of Open Channel and Storm Sewer Conveyance, was selected as the preferred stormwater drainage system plan, subject to further refinement and revision during the evaluation of flood control alternatives. Although Drainage Alternative Plan No. 3 is somewhat more costly than Alternative No. 2: Open Channel Conveyance with Selected Storm Sewer Conveyance and Existing Detention Storage, Alternative No. 3 fully satisfies more of the adopted objectives and standards, as indicated in Table 40, than does Alternative No. 2. Selection of Alternative No. 3 as the preliminary recommended drainage plan allows the most flexibility in the design of a combined stormwater management and flood control system which will best meet the stated objectives and standards of this plan.

Alternative No. 3 represents a logical progression in the planning process whereby the most applicable components of Alternative Nos. 1 and 2 are combined with the maximum level of detention storage which it is practicable to achieve within the subwatershed. Because Alternative No. 3 is able to expand upon Alternative Nos. 1 and 2 through the inclusion of additional detention storage, it is intrinsically able to provide additional flood control and streambank and habitat protection benefits not possible with the other two alternatives. In the section of this chapter which presents the description and evaluation of flood control alternatives, the level of detention storage called for in Drainage Alternative No. 3 is refined to identify those basins which have the most significant impact on flood flows in Lilly Creek and to eliminate, or reconfigure, those basins which are ineffective in providing flood control benefits along the Lilly Creek main stem or are too costly in comparison to the flood control benefits to be realized from their construction.

ALTERNATIVE WATER QUALITY MANAGEMENT PLANS

Introduction

Alternative water quality management plans for the control of nonpoint source pollutants were developed and evaluated to achieve the water quality objectives presented in Chapter IV of this report wherever practicable. The alternative measures considered represent a refinement of the more generalized recommendations presented in the regional water quality management plan for southeastern Wisconsin. Furthermore, the measures considered are consistent with the Menomonee River watershed nonpoint source control plan.[4] To the maximum extent practicable, the water quality management measures considered are also coordinated and combined with the drainage recommendations made here so as to provide multiple water quantity and water quality benefits and to minimize costs. This section describes alternative water quality management plans, estimates pollutant loadings to the surface waters under each of these alternatives, and presents the estimated cost of each alternative.

Each of the potentially available water quality management measures provides unique benefits with respect to the plan objectives. Yet, each measure also has limitations resulting from the physical constraints imposed by the watershed. The recommended water quality management plan will be selected on the basis of the desired reduction in pollutant loadings, the cost-effectiveness of the measures, the availability of suitable sites, and compatibility with the aforementioned stormwater drainage recommendations. Based upon the results of recent studies of urban nonpoint source pollution, it was concluded that four general types of control measures could be expected to be effective and could potentially have application in the Lilly Creek subwatershed. These types are: 1) wet detention basins, 2) grassed swales in areas of suburban-, low-, and medium-density urban development, 3) increased street sweeping of commercial and industrial streets, and 4) construction site erosion control measures.

Infiltration facilities, such as infiltration trenches and basins, porous pavement, and onsite seepage pits, remove waterborne pollutants by capturing surface water runoff and filtering it through the soil or other substrate material. Such facilities have been found to be highly effective in certain urban areas where the soils and drainage system are suitable and there are no significant sources of toxic pollutants which could contaminate underlying groundwater resources. Within the Lilly Creek subwatershed, however, infiltration facilities were not found to be a viable alternative because about 95 percent of the watershed is covered by poorly drained or very poorly drained soils. Under these soil conditions, infiltration rates would be relatively low, and, because runoff would be higher with only limited stormwater actually infiltrating, the removal of pollutants through infiltration into the soil would be limited.

Four alternative nonpoint source pollution control plans were developed for the Lilly Creek subwatershed. Each of these alternative plans is described below.

Water Quality Alternative Plan No. 1

Alternative No. 1 includes the implementation of construction erosion control measures and the installation of 23 wet detention basins. The construction of erosion control measures would be required under the provisions of a recommended construction erosion control ordinance.

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[4] *Wisconsin Department of Natural Resources and Wisconsin Department of Agriculture, Trade and Consumer Protection, A Nonpoint Source Control Plan for the Menomonee River Priority Watershed Project, draft, 1990.*

The ordinance would define the land-disturbing activities subject to control, set forth standards and criteria for erosion control, describe permit application and administrative procedures, and identify enforcement and appeal procedures. A model ordinance for construction erosion control was developed by the Wisconsin League of Municipalities and the Wisconsin Department of Natural Resources, and is set forth in Wisconsin Construction Site Best Management Practice Handbook, 1989. Construction site erosion control measures are temporary, have very little impact on stormwater quantity, and are therefore fully compatible with the stormwater drainage and flood control plan elements.

The availability of suitable sites is a constraint on the use of wet detention basins. Sites were considered suitable if they contained adequate open land area for the development of a basin, were on a well-defined drainage system, and drained an appropriately sized area which generated significant pollutant loadings. The 23 wet detention basins would contain permanent pools ranging in size from 0.3 to 3.7 acres and in volume from 1.7 to 18.6 acre-feet. The locations of the proposed wet detention basins and their tributary drainage areas are shown on Map 12. Each of the hydrologic units within the watershed would contain at least one wet basin. Hydrologic Unit B would contain five wet basins; the combined Hydrologic Units D and E would contain five wet basins; Hydrologic Units C, G, H, and L would each contain two wet basins; and Hydrologic Units A, F, I, J, and K would each contain one wet basin. All these basins would retain a mean permanent pool depth of about five feet. The basins, which have tributary drainage areas ranging from 31 to 446 acres, would treat runoff from a combined total area of about 2,555 acres, or about 70 percent of the total area of the subwatershed.

More than any other water quality management measure, wet detention basins require careful analysis and planning based upon application of a detailed watershed hydrologic model in order to properly locate and size the ponds and to properly adjust outflow rates. Basins may be expanded in size to reduce peak flow rates from larger storms and thereby reduce the required size of downstream conveyance facilities. Basins may also include extended detention storage for use during more frequent storms to control flows and reduce streambank erosion in downstream reaches. Some 15 of the 23 wet basins would be designed to serve stormwater quantity as well as quality management purposes, while the remaining eight basins would be designed solely for water quality management purposes. It is possible that construction of some of the wet detention basins called for under this alternative would involve excavation or dike construction in wetland areas. Such activities may require permits from the Wisconsin Department of Natural Resources and the U. S. Army Corps of Engineers.

Water Quality Alternative Plan No. 2

Alternative No. 2 includes the wet detention basin and construction erosion control proposals set forth in Alternative No. 1 plus weekly sweeping of streets during the spring, summer, and fall seasons in industrial and commercial areas which are not tributary to one of the wet detention basins. Alternative No. 2 is shown on Map 13. Increased street sweeping was not proposed in the areas tributary to detention basins because the pollutants removed by sweepers, generally the larger particles, would also readily settle out in wet basins. Increased sweeping was also not proposed for residential streets because the pollutant loading analysis modeling results indicated that such sweeping would not remove significant additional amounts of pollutants. The effectiveness of street sweeping would be greatest during spring and fall. Increased cleaning of catch basins and improved leaf collection would be associated with increased street sweeping.

Water Quality Alternative Plan No. 3

Alternative No. 3 includes the wet detention basin, construction erosion control, and street sweeping proposals set forth in Alternative No. 2, plus the installation of grassed roadside swales, rather than storm sewers, to drain all new suburban and new low-density urban residential development.[5] The roadside swales

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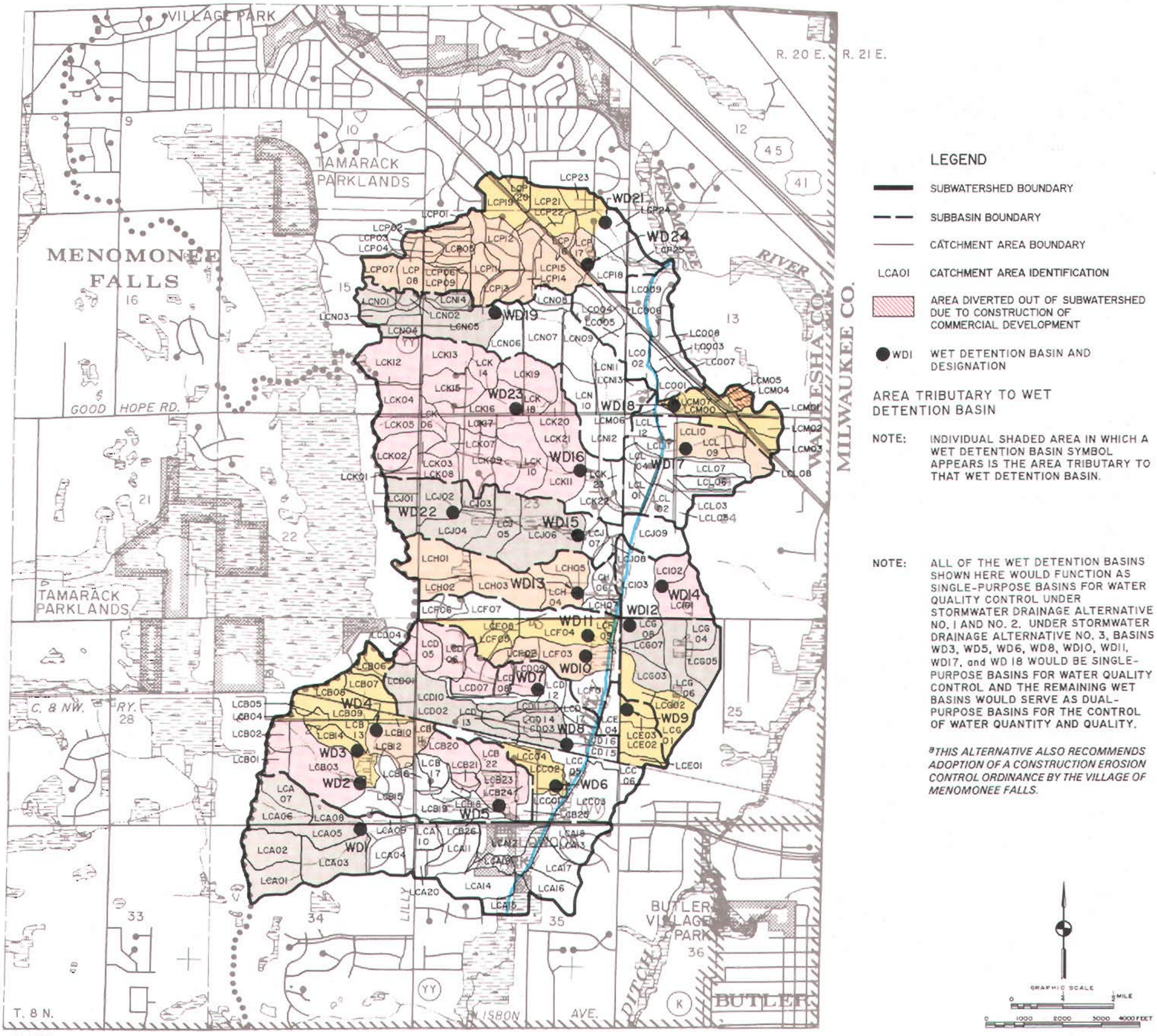
[5] *In this report and in the Village of Menomonee Falls land use plan, suburban residential development is defined as having 1.0- to 5.0-acre lots, low-density urban residential development is defined as having from 20,000-square-foot lots to 1.0-acre lots, and medium-density urban residential development is defined as having from 7,000- to 20,000-square-foot lots.*

Map 12

WATER QUALITY ALTERNATIVE PLAN NO. 1: PROPOSED WET DETENTION BASINS[a]



NOTE: INDIVIDUAL SHADED AREA IN WHICH A WET DETENTION BASIN SYMBOL APPEARS IS THE AREA TRIBUTARY TO THAT WET DETENTION BASIN.

NOTE: ALL OF THE WET DETENTION BASINS SHOWN HERE WOULD FUNCTION AS SINGLE-PURPOSE BASINS FOR WATER QUALITY CONTROL UNDER STORMWATER DRAINAGE ALTERNATIVE NO. 1 AND NO. 2. UNDER STORMWATER DRAINAGE ALTERNATIVE NO. 3, BASINS WD3, WD5, WD6, WD8, WD10, WD11, WD17, and WD 18 WOULD BE SINGLE-PURPOSE BASINS FOR WATER QUALITY CONTROL AND THE REMAINING WET BASINS WOULD SERVE AS DUAL-PURPOSE BASINS FOR THE CONTROL OF WATER QUANTITY AND QUALITY.

[a]THIS ALTERNATIVE ALSO RECOMMENDS ADOPTION OF A CONSTRUCTION EROSION CONTROL ORDINANCE BY THE VILLAGE OF MENOMONEE FALLS.

Source: SEWRPC.

would be used both within and outside the areas tributary to the proposed detention basins. Alternative No. 3 is shown on Map 14.

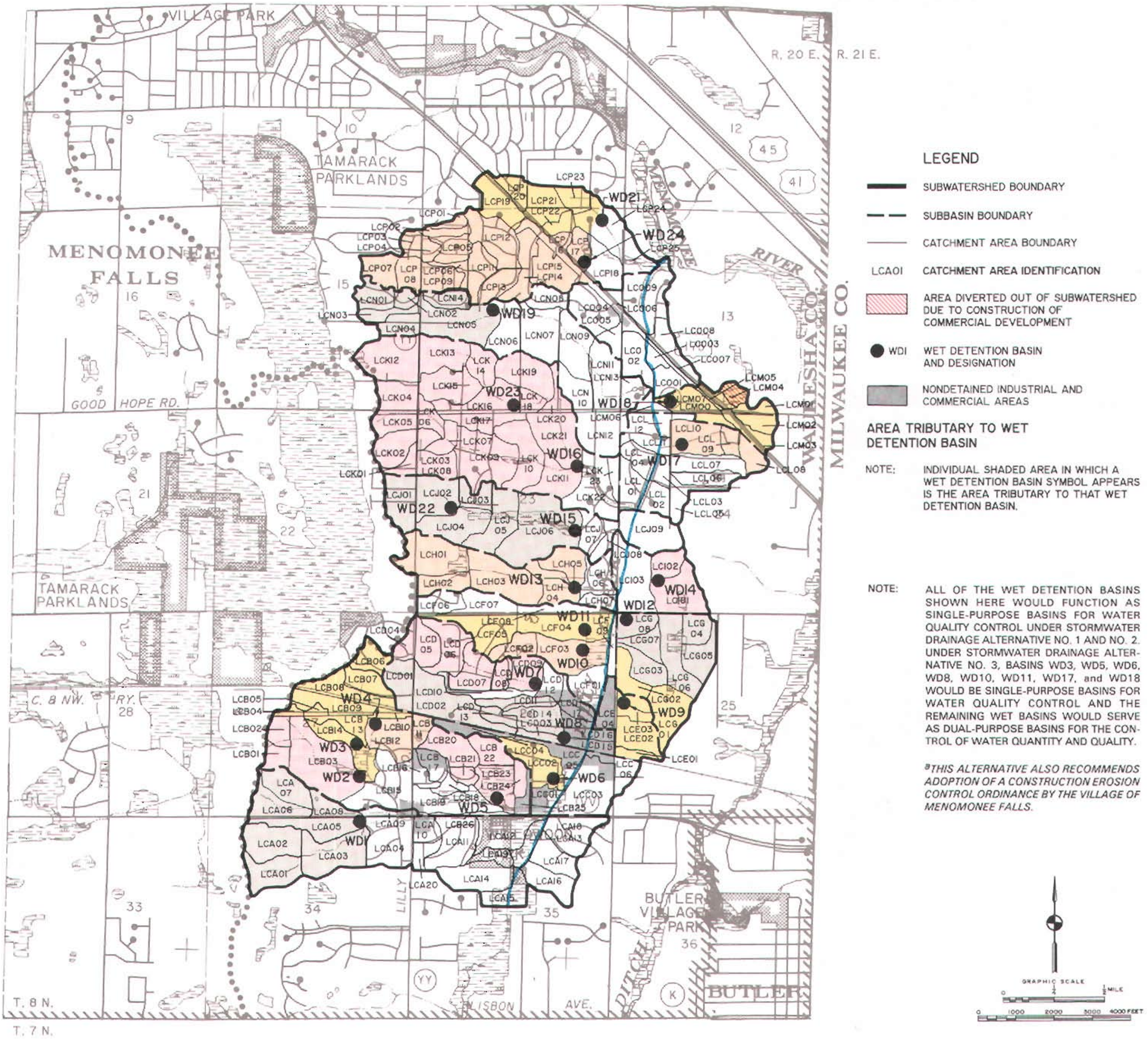
Although the grassed roadside swales would allow some stormwater runoff to infiltrate, this infiltration would not be sufficient during storms with recurrence intervals of 10 to 100 years to allow down-sizing the downstream conveyance facilities.

Water Quality Alternative Plan No. 4

Alternative No. 4 includes the wet detention basin, construction erosion control, and street

Map 13

WATER QUALITY ALTERNATIVE PLAN NO. 2: PROPOSED WET DETENTION BASINS AND STREET SWEEPING IN NONDETAINED INDUSTRIAL AND COMMERCIAL AREAS[a]



NOTE: INDIVIDUAL SHADED AREA IN WHICH A WET DETENTION BASIN SYMBOL APPEARS IS THE AREA TRIBUTARY TO THAT WET DETENTION BASIN.

NOTE: ALL OF THE WET DETENTION BASINS SHOWN HERE WOULD FUNCTION AS SINGLE-PURPOSE BASINS FOR WATER QUALITY CONTROL UNDER STORMWATER DRAINAGE ALTERNATIVE NO. 1 AND NO. 2. UNDER STORMWATER DRAINAGE ALTERNATIVE NO. 3, BASINS WD3, WD5, WD6, WD8, WD10, WD11, WD17, and WD18 WOULD BE SINGLE-PURPOSE BASINS FOR WATER QUALITY CONTROL AND THE REMAINING WET BASINS WOULD SERVE AS DUAL-PURPOSE BASINS FOR THE CONTROL OF WATER QUANTITY AND QUALITY.

[a]THIS ALTERNATIVE ALSO RECOMMENDS ADOPTION OF A CONSTRUCTION EROSION CONTROL ORDINANCE BY THE VILLAGE OF MENOMONEE FALLS.

Source: SEWRPC.

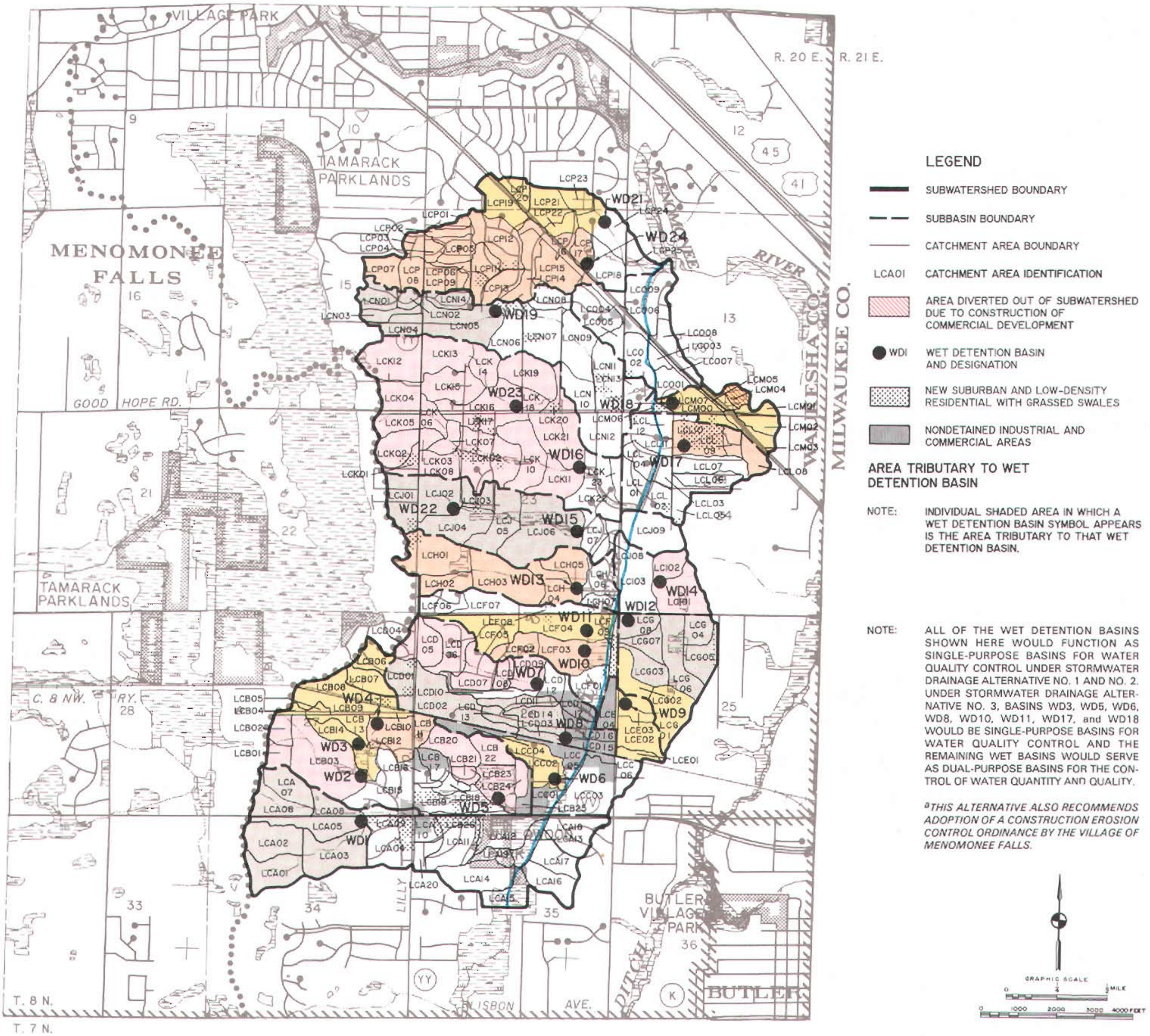
sweeping proposals set forth in Alternative No. 2 plus the installation of grassed roadside swales to drain all new medium-density urban, as well as suburban-density and low-density urban, residential development. Alternative No. 4 is shown on Map 15. Like Alternative No. 3, the grassed swales under Alternative No. 4 would not provide sufficient infiltration of stormwater to allow down-sizing the downstream conveyance facilities.

Evaluation of Water Quality Management Alternatives

The four alternative water quality management

Map 14

WATER QUALITY ALTERNATIVE PLAN NO. 3: PROPOSED WET DETENTION BASINS AND STREET SWEEPING IN NONDETAINED INDUSTRIAL AND COMMERCIAL AREAS AND GRASSED SWALES IN AREAS OF NEW SUBURBAN- AND LOW-DENSITY RESIDENTIAL DEVELOPMENT[a]



NOTE: INDIVIDUAL SHADED AREA IN WHICH A WET DETENTION BASIN SYMBOL APPEARS IS THE AREA TRIBUTARY TO THAT WET DETENTION BASIN.

NOTE: ALL OF THE WET DETENTION BASINS SHOWN HERE WOULD FUNCTION AS SINGLE-PURPOSE BASINS FOR WATER QUALITY CONTROL UNDER STORMWATER DRAINAGE ALTERNATIVE NO. 1 AND NO. 2. UNDER STORMWATER DRAINAGE ALTERNATIVE NO. 3, BASINS WD3, WD5, WD6, WD8, WD10, WD11, WD17, and WD18 WOULD BE SINGLE-PURPOSE BASINS FOR WATER QUALITY CONTROL AND THE REMAINING WET BASINS WOULD SERVE AS DUAL-PURPOSE BASINS FOR THE CONTROL OF WATER QUANTITY AND QUALITY.

[a]THIS ALTERNATIVE ALSO RECOMMENDS ADOPTION OF A CONSTRUCTION EROSION CONTROL ORDINANCE BY THE VILLAGE OF MENOMONEE FALLS.

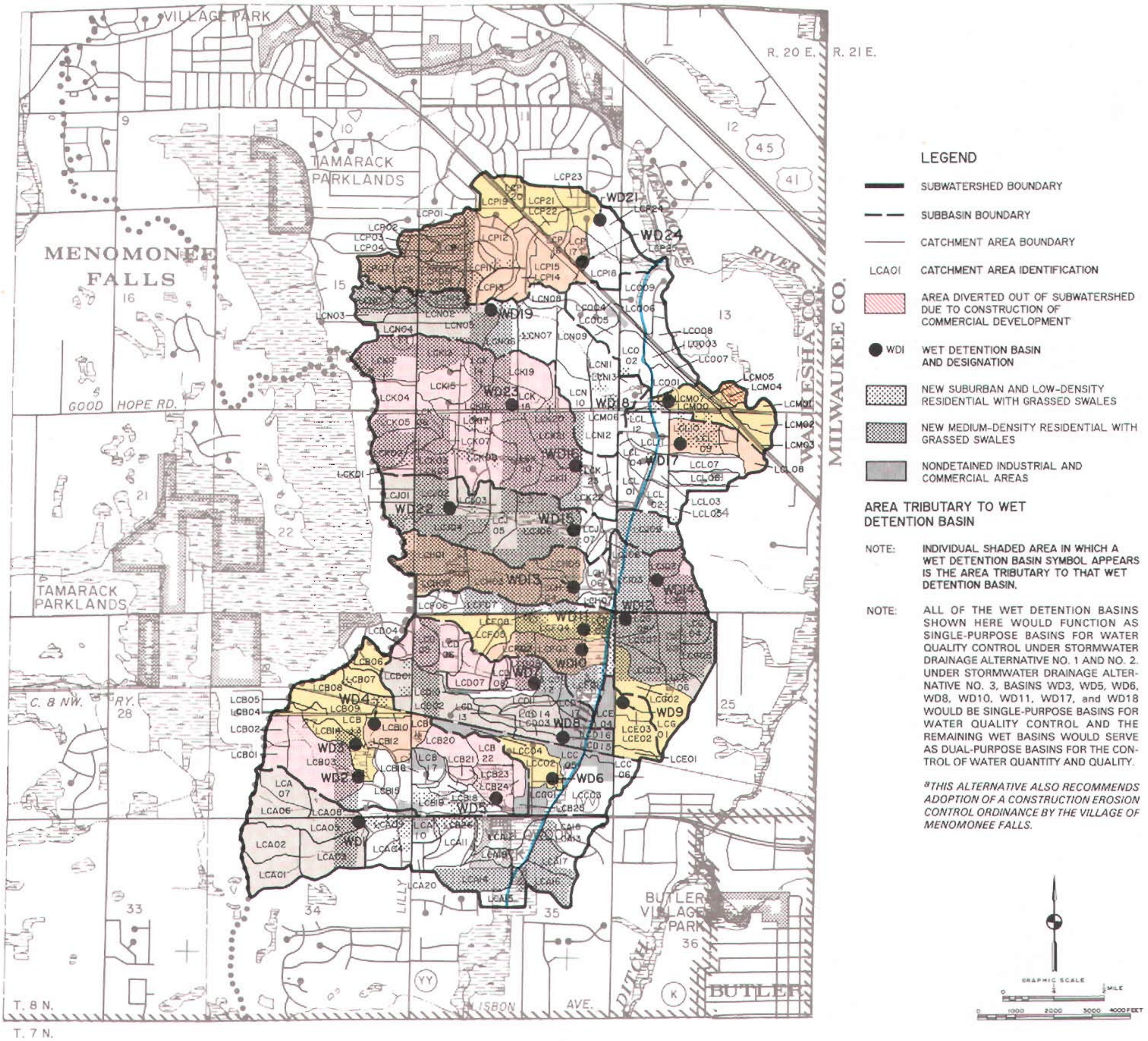
Source: SEWRPC.

plans were evaluated for each hydrologic unit with respect to pollutant removal effectiveness and cost.

Pollutant Removal Effectiveness: The assumed nonpoint source pollutant removal effectiveness of various control measures is set forth in Table 41. The estimated suspended solids, phosphorus, and lead loadings to Lilly Creek from each land use category under each of the alternative plans are presented in Tables 42, 43, and 44, respectively. The existing loadings of these pollutants, as well as estimated loadings under planned ultimate land use conditions if no

Map 15

WATER QUALITY ALTERNATIVE PLAN NO. 4: PROPOSED WET DETENTION BASINS AND STREET SWEEPING IN NONDETAINED INDUSTRIAL AND COMMERCIAL AREAS AND GRASSED SWALES IN AREA OF NEW SUBURBAN-, LOW-, AND MEDIUM-DENSITY RESIDENTIAL DEVELOPMENT[a]



NOTE: INDIVIDUAL SHADED AREA IN WHICH A WET DETENTION BASIN SYMBOL APPEARS IS THE AREA TRIBUTARY TO THAT WET DETENTION BASIN.

NOTE: ALL OF THE WET DETENTION BASINS SHOWN HERE WOULD FUNCTION AS SINGLE-PURPOSE BASINS FOR WATER QUALITY CONTROL UNDER STORMWATER DRAINAGE ALTERNATIVE NO. 1 AND NO. 2. UNDER STORMWATER DRAINAGE ALTERNATIVE NO. 3, BASINS WD3, WD5, WD6, WD8, WD10, WD11, WD17, and WD18 WOULD BE SINGLE-PURPOSE BASINS FOR WATER QUALITY CONTROL AND THE REMAINING WET BASINS WOULD SERVE AS DUAL-PURPOSE BASINS FOR THE CONTROL OF WATER QUANTITY AND QUALITY.

[a]THIS ALTERNATIVE ALSO RECOMMENDS ADOPTION OF A CONSTRUCTION EROSION CONTROL ORDINANCE BY THE VILLAGE OF MENOMONEE FALLS.

Source: SEWRPC.

controls are implemented, are also presented in the tables. All the alternatives would remove relatively large amounts of pollutants. Both construction site erosion control measures and wet detention basins, which have high pollutant removal rates, are included in all alternatives.

All the alternative plans would be highly effective in removing loadings of suspended solids, with the resultant loadings ranging from 67 to 72 percent lower than the existing loadings to Lilly Creek and from 84 percent to 86 percent lower than the planned condition loadings if no

Table 41

NONPOINT SOURCE POLLUTANT REMOVAL EFFECTIVENESS OF VARIOUS CONTROL MEASURES

Control Measure	Percent Reductions in Pollutant Loadings		
	Total Suspended Solids	Total Phosphorus	Lead[a]
Wet Detention Basins	90	50	70
Construction Site Erosion Control	75	75	75
Increased Sweeping of Commercial and Industrial Streets	43	36	54
Grassed Swales			
Suburban-, Low-Density Residential Land Areas	27	25	40
Medium-Density Residential Land Areas	32	31	33

[a]Lead is used as an indicator of the pollutant loadings of metals because lead loading rates and the removal of lead in land management systems has been well characterized.

Source: Wisconsin Department of Natural Resources and SEWRPC.

Table 42

ANNUAL TOTAL SUSPENDED SOLIDS LOADINGS TO LILLY CREEK UNDER ALTERNATIVE WATER QUALITY MANAGEMENT PLANS

Land Use Category	Existing Load (pounds)	Planned No Action		Alternative No. 1		Alternative No. 2		Alternative No. 3		Alternative No. 4	
		Load (pounds)	Percent Change[a]	Load (pounds)	Percent Change[a]	Load (pounds)	Percent Change[a]	Load (pounds)	Percent Change[a]	Load (pounds)	Percent Change[a]
Industrial	136,120	518,730	281.1	139,130	2.2	97,410	-28.4	97,410	-28.4	97,410	-28.4
Commercial	85,200	76,400	-10.3	20,033	-76.5	14,103	-83.4	14,103	-83.4	14,103	-83.4
Governmental and Institutional	15,740	17,360	10.3	7,850	-50.1	7,850	-50.1	7,850	-50.1	7,850	-50.1
Suburban-Density Residential	940	1,000	6.4	880	-6.0	880	-6.0	880	-6.0	880	-6.0
Low-Density Residential	13,050	13,690	4.9	7,420	-43.1	7,420	-43.1	7,290	-44.1	7,290	-44.1
Medium-Density Residential	5,750	245,820	4,175.1	69,396	1,106.9	69,396	1,106.9	69,396	1,106.9	48,584	744.9
Medium-High-Density Residential	8,360	53,130	535.5	38,810	364.2	38,810	364.2	38,810	364.2	38,810	364.2
Construction Sites	1,020,000	1,826,260	79.0	168,790	-83.5	168,790	-83.5	168,790	-83.5	168,790	-83.5
Prime Agriculture	67,100	0	-100.0	0	-100.0	0	-100.0	0	-100.0	0	-100.0
Parks and Recreation	90	60	-33.3	60	-29.0	60	-29.0	60	-29.0	60	-29.0
Water	740	800	8.1	540	-27.4	540	-27.4	540	-27.4	540	-27.4
Woodlands	150	130	-13.3	40	-73.2	40	-73.2	40	-73.2	40	-73.2
Wetlands	490	410	-16.3	230	-52.9	230	-52.9	230	-52.9	230	-52.9
Transportation, Communication, and Utilities	80	70	-12.5	20	-74.2	20	-74.2	20	-74.2	20	-74.2
Other	4,560	50	-98.9	20	-99.5	20	-99.5	20	-99.5	20	-99.5
Total	1,358,370	2,753,910	102.7	453,189	-66.6	405,569	-70.1	405,439	-70.2	384,627	-71.7

[a]The percent change refers to the change relative to the existing loading.

Source: SEWRPC.

controls were implemented. These reductions depend to a large degree upon the effectiveness of construction erosion control measures. Under existing conditions, uncontrolled construction site loadings account for about three-fourths of the total load of solids to Lilly Creek. It was assumed that enactment and proper enforcement of a construction erosion control ordinance would result in about a 75 percent reduction in pollutant loadings from such activities.

With respect to total phosphorus loadings, future loadings under any of the alternative plans would be only slightly lower than the existing

Table 43

ANNUAL TOTAL PHOSPHORUS LOADINGS TO LILLY CREEK UNDER ALTERNATIVE WATER QUALITY MANAGEMENT PLANS

Land Use Category	Existing Load (pounds)	Planned No Action		Alternative No. 1		Alternative No. 2		Alternative No. 3		Alternative No. 4	
		Load (pounds)	Percent Change[a]	Load (pounds)	Percent Change[a]	Load (pounds)	Percent Change[a]	Load (pounds)	Percent Change[a]	Load (pounds)	Percent Change[a]
Industrial	211.4	803.2	279.9	387.9	83.5	343.7	62.6	343.7	62.6	343.7	62.6
Commercial	132.3	118.5	10.4	70.0	-47.1	62.3	-52.9	62.3	-52.9	62.3	-52.9
Governmental and Institutional	41.9	46.3	10.5	33.8	-19.3	30.3	-27.7	30.3	-27.7	30.3	-27.7
Suburban-Density Residential	3.4	3.4	0.0	3.4	0.0	3.4	0.0	3.4	0.0	3.4	0.0
Low-Density Residential	47.5	49.8	4.8	37.0	-22.1	37.0	-22.1	37.0	-22.1	37.0	-22.1
Medium-Density Residential	16.8	625.3	3,622.0	376.7	2,142.3	376.7	2,142.3	376.7	2,143.3	264.9	1,476.8
Medium-High-Density Residential	19.3	122.2	533.2	103.5	436.3	103.5	436.3	103.5	436.3	103.5	436.3
Construction Site	663.0	1,187.1	79.0	195.7	-70.5	195.7	-70.5	195.7	-70.5	195.7	-70.5
Prime Agriculture	128.2	0.0	-100.0	0.0	-100.0	0.0	-100.0	0.0	-100.0	0.0	-100.0
Parks and Recreation	0.9	0.6	-33.3	0.6	-28.8	0.6	-28.8	0.6	-28.8	0.6	-28.8
Water	0.5	0.5	0.0	0.5	0.0	0.5	0.0	0.5	0.0	0.5	0.0
Woodlands	1.5	1.3	-13.3	0.8	-48.5	0.8	-48.5	0.8	-48.5	0.8	-48.5
Wetlands	4.9	4.1	-16.3	3.1	-37.0	3.1	-37.0	3.1	-37.0	3.1	-37.0
Transportation, Communication, and Utilities	0.8	0.8	0.0	0.4	-43.0	0.4	-43.0	0.4	-43.0	0.4	-43.0
Other	45.6	0.5	98.9	0.4	-99.2	0.4	-99.2	0.4	-99.2	0.4	-99.2
Total	1,318.1	2,963.6	124.8	1,301.9	-1.2	1,236.6	-6.2	1,236.6	-6.2	1,124.8	-14.4

[a] *The percent change refers to the change relative to the existing loading.*

Source: SEWRPC.

Table 44

ANNUAL LEAD LOADINGS TO LILLY CREEK UNDER ALTERNATIVE WATER QUALITY MANAGEMENT PLANS

Land Use Category	Existing Load (pounds)	Planned No Action		Alternative No. 1		Alternative No. 2		Alternative No. 3		Alternative No. 4	
		Load (pounds)	Percent Change[a]	Load (pounds)	Percent Change[a]	Load (pounds)	Percent Change[a]	Load (pounds)	Percent Change[a]	Load (pounds)	Percent Change[a]
Industrial	382.30	1,457.78	281.3	627.56	64.2	480.28	25.6	480.28	25.6	480.28	25.6
Commercial	239.29	214.60	-10.3	91.52	-61.8	70.60	-70.5	70.60	-70.5	70.60	-70.5
Governmental and Institutional	16.91	18.77	11.0	11.28	-33.3	9.89	-41.5	9.89	-41.5	9.89	-41.5
Suburban-Density Residential	0.86	0.90	4.7	0.83	-2.4	0.83	-2.4	0.80	-6.2	0.80	-6.2
Low-Density Residential	11.86	12.44	4.9	8.21	-30.8	8.21	-30.8	7.79	-34.3	7.79	-34.3
Medium-Density Residential	6.49	272.00	4,091.1	124.30	1,815.3	124.30	1,815.3	124.30	1,815.3	86.10	1,226.7
Medium-High-Density Residential	15.07	98.19	551.6	78.31	419.6	78.31	419.6	78.31	419.6	78.31	419.6
Construction Sites	3.57	6.39	79.0	0.82	-77.0	0.82	-77.0	0.82	-77.0	0.82	-77.0
Prime Agriculture	1.49	0.00	-100.0	0.0	-100.0	0.0	-100.0	0.0	-100.0	0.0	-100.0
Parks and Recreation	0.03	0.02	-33.3	0.0	-29.0	0.02	-29.0	0.02	-29.0	0.02	-29.0
Water	0.51	0.55	7.8	0.41	-19.6	0.41	-19.6	0.41	-19.6	0.41	-19.6
Woodlands	0.20	0.17	-15.0	0.08	-60.9	0.08	-60.9	0.08	-60.9	0.08	-60.9
Wetlands	0.66	0.55	-16.6	0.36	-45.0	0.36	-45.0	0.36	-45.0	0.36	-45.0
Transportation, Communication, and Utilities	0.10	0.04	-58.6	0.04	-58.6	0.04	-58.6	0.04	-58.6		
Other	6.08	0.07	-98.8	0.04	-99.4	0.04	-99.4	0.04	-99.4	0.04	-99.4
Total	685.43	2,082.53	203.8	943.78	37.7	775.58	13.2	775.13	13.1	736.93	7.5

NOTE: Lead is used as an indicator of the pollutant loadings of metals because that metal and its removal in land management systems has been well characterized.

[a] *The percent change refers to the change relative to the existing loading.*

Source: SEWRPC.

loadings of phosphorus. Alternative No. 1 would result in about a 1 percent reduction in existing phosphorus loadings, while Alternative Plan Nos. 2 and 3 would result in about 6 percent reductions. Alternative Plan No. 4 would provide an estimated 14 percent reduction in existing phosphorus loadings. While none of the alternative plans is expected to achieve a significant reduction in existing loadings, the plans would provide phosphorus loadings which are 56 to 62 percent lower than the planned condition loadings if no controls were implemented.

Table 45

COMPONENT COSTS FOR NONPOINT SOURCE POLLUTION CONTROL MEASURES

Hydrologic Unit	Construction Site Erosion Control			Wet Detention Basins		
	Description	Capital	Annual Operation and Maintenance	Description	Capital	Annual Operation and Maintenance
A	13.2 acres per year	$ 396,000	$1,000	WD1—2.92-acre pond	$ 242,900	$ 6,600
B	14.2 acres per year	426,000	1,100	WD2—1.48-acre pond	136,900	3,800
				WD3—1.36-acre pond	126,200	3,600
				WD4—0.54-acre pond	101,000	1,900
				WD5—1.10-acre pond	160,000	3,100
				WD6—0.59-acre pond	87,800	2,100
C	4.6 acres per year	138,000	300	WD7—0.76-acre pond	73,700	2,400
				WD8—1.62-acre pond	164,400	4,000
D and E[a]	16.6 acres per year	498,000	1,200	WD9—1.46-acre pond	132,500	3,800
				WD10—0.38-acre pond	45,400	1,600
				WD11—0.51-acre pond	56,500	1,900
				WD12—0.86-acre pond	131,300	2,600
				WD14—0.36-acre pond	43,500	1,600
F	6.7 acres per year	201,000	500	WD13—1.08-acre pond	107,000	3,000
G	7.5 acres per year	225,000	600	WD15—1.44-acre pond	190,500	3,700
				WD22—0.45-acre pond	49,100	1,800
H	13.8 acres per year	414,000	1,000	WD16—3.73-acre pond	305,700	8,000
				WD23—1.17-acre pond	110,500	3,200
I	1.2 acres per year	36,000	100	WD17—0.34-acre pond	43,600	1,600
J	4.9 acres per year	147,000	400	WD19—0.80-acre pond	78,900	2,500
K	2.6 acres per year	78,000	200	WD18—0.81-acre pond	109,400	2,500
L	6.1 acres per year	183,000	500	WD21—1.09-acre pond	128,400	3,000
				WD24—2.10-acre pond	211,000	5,200
Total	91.4 acres per year	$2,742,000	$6,900	23 ponds	$2,836,200	$73,500

Because of the extensive amount of urban development which is expected to occur within the Lilly Creek watershed, metal loadings under all the alternative plans are expected to be higher than the existing loadings. The expected increase in lead loadings would approximate 7 percent for Alternative No. 4, 13 percent for Alternative Plan Nos. 2 and 3, and 38 percent for Alternative Plan No. 1. The lead loadings under the alternative plans would be expected to be 55 to 65 percent lower than the planned condition loadings if no controls were implemented.

Two of the detention basins, a 0.4-acre basin within Hydrologic Unit G on the Jerry Lane Tributary and a 1.2-acre basin within Hydrologic Unit H along the North Branch of Oakwood Tributary, would be located upstream of, and within the drainage areas to, other basins. The total pollutant loadings to Lilly Creek itself would not be significantly reduced by these upstream basins because the lower basins would by themselves remove essentially all settleable pollutants. However, upstream basins would help protect stream reaches located between them and the lower basins. The 0.4-acre basin may be expected to reduce pollutant loadings to a 0.6-mile stream segment by about 25 percent, and the 1.2-acre basin may be expected to reduce pollutant loadings to a 0.5-mile stream segment by about 50 percent.

Cost: The estimated costs of each type of nonpoint source control measure included within the alternative plans are presented in Table 45. The total capital costs within the subwatershed for individual types of measures may range from $2,800 for increased street sweeping to $2,836,200 for wet detention basins. The estimated costs for grassed roadside swales represent the change in the drainage system cost associated with provid-

Table 45 (continued)

Hydrologic Unit	Street Sweeping[b]			Grassed Swale: Low- and Suburban-Density Residential[c]			Grassed Swale Medium-Density Residential[d]		
	Description	Capital	Annual Operation and Maintenance	Description	Capital	Annual Operation and Maintenance	Description	Capital	Annual Operation and Maintenance
A	Weekly sweeping of 0.58 curb-mile	$ 200	$ 400	--	$ --	$ --	27,300 feet	$ -72,400	$ 12,300
B	Weekly sweeping of 2.22 curb-miles	700	1,600	--	--	--	7,200 feet	-19,000	3,200
C	Weekly sweeping of 0.46 curb-mile	100	300	--	--	--	18,600 feet	-49,200	8,400
D and E[a]	Weekly sweeping of 2.7 curb-miles	900	2,000	--	--	--	69,400 feet	-183,900	31,200
F	--	--	--	--	--	--	41,800 feet	-110,700	18,800
G	--	--	--	--	--	--	47,000 feet	-124,400	21,100
H	--	--	--	--	--	--	83,700 feet	-221,900	37,700
I	--	--	--	3,600 feet	-9,600	1,600	--	--	--
J	--	--	--	--	--	--	22,000 feet	-58,400	9,900
K	Weekly sweeping of 1.32 curb-miles	400	1,000	--	--	--	--	--	--
L	Weekly sweeping of 1.78 curb-miles	500	1,300	--	--	--	26,700 feet	-70,700	12,000
Total	9.06 curb-miles	$2,800	$6,600	3,600 feet of grassed swales	$-9,600	$1,600	343,700 feet of grassed swales	$-910,600	$154,600

[a] *Hydrologic Units D and E are combined because planned conditions assume a portion of the runoff from Hydrologic Unit E is redirected to Hydrologic Unit D.*

[b] *Includes weekly sweeping of all industrial and commercial streets that do not drain to a wet detention basin.*

[c] *Includes installation of grassed swales in all areas of new suburban- and low-density residential development. The capital and operation and maintenance costs shown are the incremental costs compared to the cost of storm sewers. Compared to storm sewers, the grassed swales would entail a lower capital cost, but a higher maintenance cost.*

[d] *Includes installation of grassed swales in all areas of new medium-density residential development. The capital and operation and maintenance costs shown are the incremental costs compared to the cost of storm sewers. Compared to storm sewers, the grassed swales would entail a lower capital cost, but a higher maintenance cost.*

Source: SEWRPC.

ing swales in lieu of storm sewers. Since only the change in cost would be allocated to the nonpoint source control elements of the alternative plans. While the capital costs of swales are lower than for storm sewers, the annual maintenance costs are much higher, resulting in the total annual costs of swales being higher than for sewers.

The capital costs of the alternative plans are presented in Table 46, and range from a low of $4,660,600 for Alternative Plan No. 4 to a high of $5,580,800 for Alternative Plan No. 2. The equivalent annual costs range from a low of $360,100 for Alternative Plan No. 1 to a high of $477,100 for Alternative Plan No. 4. The capital

Table 46

WATER QUALITY MANAGEMENT ALTERNATIVE PLAN COSTS FOR THE LILLY CREEK SUBWATERSHED: 1985-2010

Alternative No. 1				
Hydrologic Unit	Project and Component Description	Capital Cost	Annual Operation and Maintenance Cost	Equivalent Annual Cost[a]
A	Construction erosion control, 13.2 acres per year	$ 396,000	$ 1,000	$ 15,400
	1. WD1—2.92-acre pond	242,900	6,600	22,000
	Subtotal	$ 638,900	$ 7,600	$ 37,400
B	Construction erosion control, 14.2 acres per year	$ 426,000	$ 1,100	$ 16,400
	1. WD2—1.48-acre pond	136,900	3,800	12,500
	2. WD3—1.36-acre pond	126,200	3,600	11,600
	3. WD4—0.54-acre pond	101,000	1,900	8,300
	4. WD5—1.10-acre pond	160,000	3,100	13,300
	5. WD6—0.59-acre pond	87,800	2,100	7,700
	Subtotal	$1,037,900	$ 15,600	$ 69,800
C	Construction erosion control, 4.6 acres per year	$ 138,000	$ 300	$ 5,300
	1. WD7—0.76-acre pond	73,700	2,400	7,100
	2. WD8—1.62-acre pond	164,400	4,000	14,400
	Subtotal	$ 376,100	$ 6,700	$ 26,800
D and E[b]	Construction erosion control, 16.6 acres per year	$ 498,000	$ 1,200	$ 19,300
	1. WD9—1.46-acre pond	132,500	3,800	12,200
	2. WD10—0.38-acre pond	45,400	1,600	4,500
	3. WD11—0.51-acre pond	56,500	1,900	5,500
	4. WD12—0.86-acre pond	131,300	2,600	10,900
	5. WD14—0.36-acre pond	43,500	1,600	4,400
	Subtotal	$ 907,200	$ 12,700	$ 56,800
F	Construction erosion control, 6.7 acres per year	$ 200,800	$ 500	$ 7,800
	1. WD13—1.08-acre pond	107,000	3,000	9,800
	Subtotal	$ 307,800	$ 3,500	$ 17,600
G	Construction erosion control, 7.5 acres per year	$ 225,000	$ 600	$ 8,800
	1. WD15—1.44-acre pond	190,500	3,700	15,800
	2. WD22—0.45-acre pond	49,100	1,800	4,900
	Subtotal	$ 464,600	$ 6,100	$ 29,500
H	Construction erosion control, 13.8 acres per year	$ 414,000	$ 1,000	$ 16,100
	1. WD16—3.73-acre pond	305,700	8,000	27,400
	2. WD23—1.17-acre pond	110,500	3,200	10,200
	Subtotal	$ 830,200	$ 12,200	$ 53,700
I	Construction erosion control, 1.2 acres per year	$ 36,000	$ 100	$ 1,400
	1. WD17—0.34-acre pond	43,600	1,600	4,400
	Subtotal	$ 79,600	$ 1,700	$ 5,800
J	Construction erosion control, 4.9 acres per year	$ 147,000	$ 400	$ 5,800
	1. WD19—0.80-acre pond	78,900	2,500	7,500
	Subtotal	$ 225,900	$ 2,900	$ 13,300
K	Construction erosion control, 2.6 acres per year	$ 78,000	$ 200	$ 3,000
	1. WD18—0.81-acre pond	109,400	2,500	9,400
	Subtotal	$ 187,400	$ 2,700	$ 12,400
L	Construction erosion control, 6.1 acres per year	$ 183,000	$ 500	$ 7,200
	1. WD21—1.09-acre pond	128,400	3,000	11,200
	2. WD24—2.10-acre pond	211,000	5,200	18,600
	Subtotal	$ 522,400	$ 8,700	$ 37,000
	Total	$5,578,000	$ 80,400	$360,100

Table 46 (continued)

Alternative No. 2[c]				
Hydrologic Unit	Project and Component Description	Capital Cost	Annual Operation and Maintenance Cost	Equivalent Annual Cost[a]
A	Alternative No. 1 plus sweep 0.58 curb-mile of street	$ 639,100	$ 8,000	$ 37,800
B	Alternative No. 1 plus sweep 2.22 curb-miles of street . . .	1,038,600	17,200	71,400
C	Alternative No. 1 plus sweep 0.46 curb-mile of street	376,200	7,000	27,100
D and E[b]	Alternative No. 1 plus sweep 2.70 curb-miles of street . . .	908,100	14,700	72,400
F	Same as Alternative No. 1	307,800	3,500	17,600
G	Same as Alternative No. 1	464,600	6,100	29,500
H	Same as Alternative No. 1	830,200	12,200	53,700
I	Same as Alternative No. 1	79,600	1,700	5,800
J	Same as Alternative No. 1	225,900	2,900	13,300
K	Alternative No. 1 plus sweep 1.32 curb-miles of street . . .	187,800	3,700	13,400
L	Alternative No. 1 plus sweep 1.78 curb-miles of street . . .	522,900	10,000	38,300
Total		$5,580,800	$ 87,000	$380,300

Alternative No. 3[d]				
Hydrologic Unit	Project and Component Description	Capital Cost	Annual Operation and Maintenance Cost	Equivalent Annual Cost[a]
A	Same as Alternative No. 2	$ 639,100	$ 8,000	$ 37,800
B	Same as Alternative No. 2	1,038,600	17,200	71,400
C	Same as Alternative No. 2	376,200	7,000	27,100
D and E[b]	Same as Alternative No. 2	908,100	14,700	72,400
F	Same as Alternative No. 2	307,800	3,500	17,600
G	Same as Alternative No. 2	464,600	6,100	29,500
H	Same as Alternative No. 2	830,200	12,200	53,700
I	Alternative No. 2 plus 3,600 feet of grass swales	70,000	3,300	6,800
J	Same as Alternative No. 2	225,900	2,900	13,300
K	Same as Alternative No. 2	187,800	3,700	13,400
L	Same as Alternative No. 2	522,900	10,000	38,300
Total		$5,571,200	$ 88,600	$381,300

Table 46 (continued)

Alternative No. 4[e]				
Hydrologic Unit	Project and Component Description	Capital Cost	Annual Operation and Maintenance Cost	Equivalent Annual Cost[a]
A	Alternative No. 3 plus 27,300 feet of grass swales in medium-density residential	$ 566,700	$ 20,300	$ 45,500
B	Alternative No. 3 plus 7,200 feet of grass swales in medium-density residential	1,019,600	20,400	72,400
C	Alternative No. 3 plus 18,600 feet of grass swales in medium-density residential	327,000	15,400	32,400
D and E[b]	Alternative No. 3 plus 69,400 feet of grass swales in medium-density residential	724,200	45,900	91,900
F	Alternative No. 3 plus 41,800 feet of grass swales in medium-density residential	197,100	22,300	29,400
G	Alternative No. 3 plus 47,000 feet of grass swales in medium-density residential	340,200	27,200	42,700
H	Alternative No. 3 plus 83,700 feet of grass swales in medium-density residential	608,300	49,900	77,300
I	Same as Alternative No. 3	70,000	3,300	6,800
J	Alternative No. 3 plus 22,000 feet of grass swales in medium-density residential	167,500	12,800	19,500
K	Same as Alternative No. 3	187,800	3,700	13,400
L	Alternative No. 3 plus 26,700 feet of grass swales in medium-density residential	452,200	22,000	45,800
Total		$4,660,600	$243,200	$477,100

[a] *Equivalent annual cost computations assume 50-year life and 6 percent annual interest.*

[b] *Hydrologic Units D and E are combined because planned conditions assume a portion of the runoff from Hydrologic Unit E is redirected to Hydrologic Unit D.*

[c] *Includes weekly sweeping of all industrial and commercial streets that do not drain to a wet detention basin.*

[d] *Includes installation of grassed swales in all areas of new suburban and lowdensity residential development. The capital and operation and maintenance costs shown are the incremental costs compared to the cost of storm sewers. Compared to storm sewers, the grassed swales would entail a lower capital cost, but a higher maintenance cost. Therefore, the capital cost is less than that of Alternative No. 2 because the incremental capital cost of grassed swales is treated as a negative cost in order to provide a consistent basis for comparison with Alternative Nos. 1 and 2, which assume storm sewer drainage, but do not include a cost for storm sewers.*

[e] *Includes installation of grassed swales in all areas of new suburban-, low-, and medium-density residential development. The capital and operation and maintenance costs shown are the incremental costs compared to the cost of storm sewers. Compared to storm sewers, the grassed swales would entail a lower capital cost, but a higher maintenance cost. Therefore, the capital cost is less than that of Alternative No. 3 because the incremental capital cost of swales is treated as a negative cost in order to provide a consistent basis for comparison with Alternative Nos. 1, 2, and 3, which assume storm sewer drainage in areas of planned medium-density residential development, but do not include a cost for storm sewers in those areas.*

Source: SEWRPC.

costs of Alternative Plan Nos. 3 and 4 are less than those of Alternative Plan Nos. 1 and 2 because Alternative Plan Nos. 1 and 2 assume storm-sewer drainage in all areas of new development, while Alternative Plan Nos. 3 and 4 assume the use of grassed swale drainage in areas of suburban-density, low- and/or medium-density residential development. However, when the equivalent annual costs, which include operation and maintenance costs, are compared, Alternative Plan Nos. 3 and 4 are somewhat more costly than Alternative Plan Nos. 1 and 2.

Selection of the Preferred Alternative for Control of Nonpoint Source Pollution within the Lilly Creek Subwatershed

Based on consideration of the level of reduction in pollutant loadings, equivalent annual cost, and compatibility with the preferred alternative stormwater drainage plan, nonpoint source control Alternative Plan No. 3, Wet Detention Basins, Construction Erosion Control, Increased Sweeping of Nondetained Industrial and Commercial Streets, and Grassed-Swale Drainage of New Suburban-Density and New Low-Density Urban Residential Development, is the preferred alternative.

As shown in Table 46, the capital cost of Alternative Plan No. 3 is essentially equal to that of Alternative Plan Nos. 1 and 2, and is about 20 percent higher than that of Alternative Plan No. 4. The equivalent annual cost of Alternative Plan No. 3 is less than 6 percent higher than that of Alternative Plan Nos. 1 and 2, and 25 percent lower than that of Alternative Plan No. 4. The only significant difference between Alternative Plan Nos. 2 and 3 is that Alternative Plan No. 3 includes grassed roadside swales in areas of new suburban-density and low-density urban residential development. Although the nonpoint source pollution contribution from suburban-density and low-density urban residential development is relatively small, grassed swale drainage was considered more practical for these urban residential areas lower of density.

Alternative Plan Nos. 3 and 4 would provide essentially the same level of control of total suspended solids. Alternative Plan No. 4 may be expected to provide a slightly greater level of control of total phosphorus and metals than would Alternative Plan No. 3. In the overall context of the stormwater management plan, Alternative Plan No. 3 is considered superior to Alternative Plan No. 4 on the basis of the achievement of an equal or similar level of reduction in existing nonpoint source pollution loadings to Lilly Creek, of a lower equivalent annual cost, of consistency with the preliminary recommended stormwater drainage plan, and of agreement with policies and preferences of the Village of Menomonee Falls with respect to the various possible approaches to stormwater management.

In addition to the components of the recommended alternative plan listed previously, it is also recommended that a public education program be developed to encourage good urban "housekeeping" practices, to promote the selection of building and construction materials which reduce the runoff contribution of metals and other toxic pollutants, and to promote the acceptance and understanding of the proposed pollution abatement measures and the importance of water quality protection. Urban housekeeping practices and source controls include restricted use of fertilizers and pesticides, improved pet waste and litter control, the reduced use of galvanized steel roof materials and gutters, proper disposal of motor vehicle fluids, increased leaf collection and catch basin cleaning, and reduced use of street-deicing salt. Particular attention should be given to reducing pollutant loadings from high pollutant loading areas, such as industrial and commercial sites, parking lots, and material storage areas. To the extent practicable, rooftop and parking lot stormwater runoff should be diverted to pervious soil and vegetated areas, rather than being directly discharged to a storm sewer. Special spill control or containment facilities, such as earthen berms, may be used to reduce the discharge of such spilled substances as oil and grease into waterways. Material storage areas may be enclosed or periodically cleaned and diversion of stormwater away from these sites may further reduce pollutant loadings.

Other measures, such as reduced use of leaded gasoline and increased air pollution control, which may be implemented on a regional, state, or national level, may also be expected to reduce loadings of certain pollutants including metals. For example, the reduced use of leaded gasoline since 1974 has contributed to reduced dissolved lead levels in nearly two-thirds of the major rivers within the United States.[6]

Integration of the Preferred Stormwater Drainage and Water Quality Management Plans into a Preliminary Recommended Stormwater Management Plan

The preferred alternative stormwater drainage plan, Maximum Detention Storage with a Com-

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[6] *R. B. Alexander and R. A. Smith, "Trends in Lead Concentrations in Major U. S. Rivers and Their Relation to Historical Changes in Gasoline Lead Consumption,"* Water Resources Bulletin, *Vol. 24, No. 3, June 1988, pp. 557-569.*

bination of Open Channel and Storm Sewer Conveyance, and the preferred alternative water quality management plan, Wet Detention Basins, Construction Erosion Control, Sweeping of Nondetained Industrial and Commercial Streets, and Grassed Swale Drainage of New Suburban-Density and Low-Density Residential Development, are compatible and were integrated into a preliminary recommended stormwater management plan for the Lilly Creek subwatershed. The dual-purpose detention basins called for under the drainage alternative are sized and configured to accommodate the permanent ponds proposed under the nonpoint source control alternative. The potential elimination of some quantity-control basins following further investigation and plan refinement during development of the flood control plan would not require elimination of proposed wet detention basins at those sites. Both of the preferred plans call for the maintenance of most existing roadside swales, and rely on new roadside swale systems for conveyance of stormwater or for control of nonpoint source pollution in areas of new suburban and low-density urban residential development. The construction erosion control and street sweeping proposals are essentially independent of the drainage proposals and are readily implementable under the preferred drainage alternative.

The comparative evaluation of three alternative stormwater drainage system plans for the Lilly Creek subwatershed indicated that the capital cost of such plans may be expected to range from $7.4 million to $13.8 million, while the annual operation and maintenance costs may be expected to range from $31,400 to $95,600.

The comparative evaluation also indicated that the drainage alternative calling for maximum detention storage with a combination of open channel and storm sewer conveyance would best satisfy the objectives and supporting standards adopted for this planning effort and would offer the most flexibility in formulating a flood control plan for the main stem of Lilly Creek.

The comparative evaluation of four alternative nonpoint source pollution control system plans for the Lilly Creek subwatershed indicated that the capital cost of such plans may be expected to range from $4.7 million to $5.6 million, while the annual operation and maintenance costs may be expected to range from $80,400 to $243,200.

The comparative evaluation also indicated that the water quality management alternative calling for wet detention basins, construction erosion control, sweeping of nondetained industrial and commercial streets, and grassed swale drainage of suburban- and low-density residential development would best satisfy the objectives and supporting standards adopted for this planning effort. The preferred alternative stormwater drainage and nonpoint source pollution control alternatives are compatible and can be readily integrated into a preliminary recommended stormwater management plan for the Lilly Creek subwatershed.

DESCRIPTION AND EVALUATION OF ALTERNATIVE FLOOD CONTROL PLANS FOR THE MAIN STEM OF LILLY CREEK

Introduction

Alternative flood control plans for the main stem of Lilly Creek were developed on the basis of the preliminary recommended stormwater management plan presented in this chapter.

The preliminary recommendation regarding the provision of detention storage for water quantity control was reevaluated and refined in the context of the flood control element of the plan. The need for the water quantity portion of each basin was evaluated based on three major criteria: 1) the ability of the basin to achieve a significant savings in the cost of downstream conveyance components of the stormwater drainage system, 2) the effect of the basin on flood flows in Lilly Creek, and 3) the cost of the water quantity control portion of the basin relative to the reduction in flood damages along Lilly Creek which the basin would provide. On the basis of those criteria, the quantity control component was eliminated for basin WD7 along the Bowling Green Tributary in Hydrologic Unit C and for basins WD12, WD14, and DD5 in Hydrologic Unit E. A comparison of 100-year recurrence interval flood flows along Lilly Creek under various land use, stormwater drainage, and channel conditions is given in Table 47.

Summary of Potential Flood Damages

The hydrologic and hydraulic analyses performed for this study identified a total of 23 residential and 11 industrial or commercial buildings which would lie in the 100-year recurrence interval floodplain of Lilly Creek under planned land use and existing channel condi-

Table 47

COMPARISON OF 100-YEAR RECURRENCE INTERVAL FLOODS IN LILLY CREEK

River Mile	Existing (1985) Land Use, Drainage, and Channel Conditions	Planned Ultimate Land Use and Existing Lilly Creek and Tributary Channel Conditions[a,b]	Planned Ultimate Land Use, Stormwater Drainage Alternative No. 3 (maximum detention) Components with Detention Basins WD7, 12, 14, and DD5 Eliminated, and Existing Lilly Creek Channel[b]	Planned Ultimate Land Use, Stormwater Drainage Alternative No. 3 (maximum detention) Components with Detention Basins WD7 and 14 Eliminated and Lilly Creek Channel Modified as Called for under Flood Control Alternative No. 3	Planned Ultimate Land Use, Stormwater Drainage Alternative No. 3 (maximum detention) Components with Detention Basins WD7 and 14 Eliminated and Lilly Creek Channel Modified as Called for under Flood Control Alternative No. 4
0.0 (mouth)	2,590	2,810	2,250	2,540	2,120
0.06	2,260	2,500	1,760	2,160	1,740
0.40 (W. Appleton Avenue)	2,260	2,500	1,760	2,160	1,740
0.78	1,840	2,210	1,320	1,800	1,370
0.84 (W. Good Hope Road)	1,840	2,210	1,320	1,800	1,370
0.85	1,830	2,200	1,320	1,800	1,370
0.99	1,770	2,140	1,250	1,730	1,280
1.06 (Brentwood Drive)	1,770	2,140	1,250	1,730	1,280
1.07	1,730	2,100	1,200	1,680	1,230
1.16	1,720	2,080	1,190	1,680	1,230
1.22	1,720	2,070	1,180	1,660	1,230
1.29	1,700	2,040	1,140	1,630	1,200
1.37	1,070	1,260	960	1,460	1,020
1.53	820	970	840	1,310	910
1.71	810	870	750	1,220	860
1.81	640	760	600	990	530
1.88 (W. Mill Road)	640	760	600	990	530
1.89	640	750	600	880	510
2.19	600	660	570	650	460
2.37	550	620	410	450	420
2.43 (Kaul Avenue)	550	620	410	450	420
2.44	520	580	380	400	410
2.48 (Bobolink Avenue)	520	580	380	400	410
2.59 (C&NW Railway)	520	580	380	400	410
2.60	440	490	360	400	390
2.85	420	470	340	370	360
2.97 (W. Silver Spring Drive)	150	160	140	150	140

[a] *Assumes no new detention storage is provided and storm sewer conveyance components are provided in areas of new development.*

[b] *It is assumed that the existing hydraulic structures in Lilly Creek at W. Mill Road and Lilly Road are replaced with a single structure consisting of two reinforced concrete box culverts, extending across the intersection of W. Mill Road and Lilly Road.*

Source: SEWRPC.

tions, assuming the provision of a stormwater drainage system with no constructed detention storage beyond that provided by the significant natural and man-made storage which currently exists in the subwatershed. The total damages due to direct overland flooding to those buildings under 100-year recurrence interval flood conditions may be expected to approximate $352,000 and the average annual flood damages may be expected to approximate $88,500.

Through the provision of additional constructed detention storage, excluding the water quantity control components of detention basins WD7, WD12, WD14, and DD5, and of appropriate drainage components within the preliminary recommended stormwater management plan, the total number of buildings in the 100-year recurrence interval floodplain under planned ultimate land use and existing channel conditions would be reduced to 17 residential and seven industrial or commercial buildings. The total damages due to direct overland flooding of those buildings under 100-year recurrence interval storm conditions may be expected to approximate $219,000 and the average annual damages, $64,700. Therefore, 100-year recurrence interval flood damages may be expected to be reduced by about 38 percent and average annual flood damages by about 27 percent through the provision of the refined preliminary recommended stormwater management measures.

The residential buildings remaining in the 100-year recurrence interval floodplain under planned land use and existing channel conditions with the refined preliminary recommended stormwater management plan components in place would be concentrated in the 0.57-mile-long reach of Lilly Creek extending from a point near Houston Drive and about 1,100 feet downstream of W. Good Hope Road to a point about 350 feet downstream of the intersection of Bay Ridge Lane and Manor Hills Boulevard. Two additional homes within the 100-year floodplain would be located in an 0.11-mile-long reach south of W. Mill Road and west of Lilly Road.

The industrial and commercial buildings remaining in the 100-year recurrence interval floodplain under planned land use and existing channel conditions with the preliminary recommended stormwater management plan components in place would be concentrated in a 0.17-mile-long reach of Lilly Creek, extending from a point about 250 feet downstream of Kaul Avenue to a point about 500 feet upstream of the Bobolink Avenue on the north side of the Chicago & North Western Railway embankment.

Description of Alternative Flood Control Plans

Three alternative flood control plans were initially evaluated for the abatement of overland flooding damages from storms with recurrence intervals up to and including a 100-year recurrence interval event under planned land use and recommended stormwater drainage conditions. Those alternative flood control plans included: 1) Structure Floodproofing, Elevation, and Removal, 2) Acquisition and Removal of Structures and 3) Channel Modification and Bridge Removal or Replacement.[7]

Under each of the three alternatives, because of the provision of off-channel detention storage under the refined preliminary recommended stormwater drainage plan, the flood flow at the confluence of Lilly Creek with the Menomonee River resulting from a 100-year recurrence interval storm would be less than the flow under existing land use and channel conditions. Therefore, the alternative plans would not be expected to create an increase in the peak 100-year recurrence interval flood flows and stages on the Menomonee River. The components of the alternative plans are discussed below. Selected characteristics and the cost of each alternative plan are provided in Table 48.

Alternative Flood Control Plan No. 1: Structure Floodproofing, Elevation, and Removal: The first alternative plan considered calls for the floodproofing of 10 single-family residential buildings, the elevation of five single-family residential buildings, the removal of two single-family residential buildings, and the floodproofing of seven industrial or commercial buildings. The buildings to be floodproofed, elevated, or removed are shown on Map 16.

Full implementation of this alternative plan would serve to eliminate flood damages due to direct overland flooding along Lilly Creek from floods up to and including the 100-year recurrence interval flood event under planned land use and channel conditions, assuming complete implementation of the refined preliminary recommended stormwater drainage element of the system plan. That stormwater drainage element eliminates the water quantity control portions of detention basins WD7, WD12, WD14, and DD5.

In the case of residential buildings, floodproofing was assumed to be feasible if the design flood stage was below the first floor elevation. Structure elevation was considered feasible for residential structures with basements if the estimated cost of elevating the structure and floodproofing the basement was less than the estimated removal cost. Structures to be elevated were assumed to have the first floor raised to an elevation at least two feet above the 100-year recurrence interval flood stage to provide adequate freeboard. For aesthetic reasons, structure elevation was limited to a maximum of four feet. Structures which would have to be elevated more than four feet or structures with flooding of exposed basements were considered for removal. Industrial or commercial buildings with first-floor flooding during a 100-year recurrence interval flood were considered for floodproofing. Potential components of a structure floodproofing system are set forth in Chapter III of this report.

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[7] *The fourth alternative, Bridge Replacement; Road Elevation; and Structure Floodproofing, Elevation and Removal, was developed following the meeting of the Village of Menomonee Falls Lilly Creek Stormwater Management and Flood Control Advisory Committee on September 20, 1990. The alternative was developed in response to comments and suggestions provided by the staff of the Wisconsin Department of Natural Resources. The alternative is presented later in this chapter.*

Table 48

PRINCIPAL FEATURES AND COSTS OF ALTERNATIVE FLOOD CONTROL PLANS FOR LILLY CREEK IN THE VILLAGE OF MENOMONEE FALLS

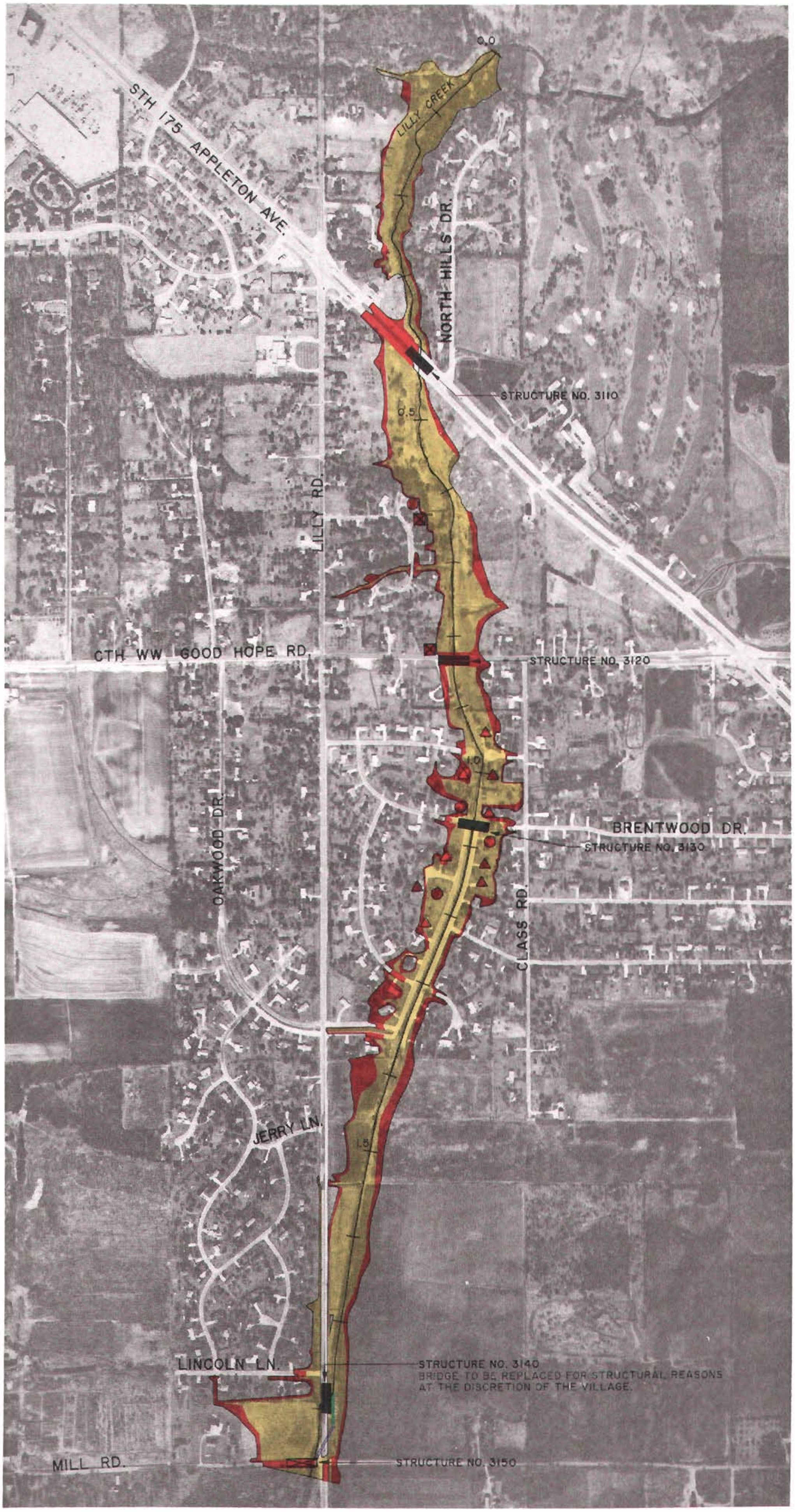
Alternative	Description[a]	Costs				Average Annual Benefits	Benefit-Cost Ratio
		Capital	Amortized Capital[b]	Annual Operation and Maintenance	Total		
No. 1—Structure Floodproofing, Elevation, and Removal	Floodproof 10 residential buildings	$ 55,000					
	Elevate five residential buildings	231,000					
	Remove two residential buildings	217,000					
	Floodproof seven industrial or commercial buildings	53,000					
	Total	$ 556,000	$ 35,300	$ 0	$ 35,300	$64,700	1.8
No. 2—Acquisition and Removal of Structures	Acquisition and removal 17 residential buildings	$1,800,000					
	Acquisition and removal of seven industrial or commercial buildings	884,000					
	Total	$2,684,000	$170,400	$ 0	$170,400	$64,700	0.4
No. 3—Channel Modification and Bridge Removal or Replacement	2.53 miles of channel widening and deepening	$1,521,000					
	Removal and replacement of six road bridges	326,000					
	Removal of three pedestrian bridges and replacement of two of those bridges	89,000					
	Construction of two additional off-channel detention basins	348,000					
	Reconstruction and relocation of Menomonee Manor Boulevard	250,000					
	Additional easements along modified channel	20,000					
	Total	$2,554,000	$162,200	$12,000	$174,200	$64,700	0.4
No. 4—Bridge Replacement; Road Elevation; and Structure Floodproofing, Elevation, and Removal[c]	Removal and replacement of seven road bridges	$1,730,000					
	Road elevation at Bobolink and Kaul Avenues	190,000					
	Floodproof seven residential buildings	39,000					
	Floodproof six industrial or commercial buildings	44,000					
	Elevate four residential buildings	182,000					
	Remove one residential building	137,000					
	Total	$2,322,000	$147,000	$ 0	$147,000	$64,700	0.4

[a] *A single replacement bridge for the existing W. Mill Road and Lilly Road bridges is called for under both alternatives. The bridge replacement would be implemented as part of the arterial street improvements recommended in the Village Land Use and Transportation Plans; its cost is, therefore, not assigned to this flood control plan.*

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

[c] *This alternative plan was developed after the September 20, 1990, meeting of the Village of Menomonee Falls Lilly Creek Stormwater Management and Flood Control Advisory Committee. The alternative is set forth in detail in a subsequent section of this chapter.*

Source: SEWRPC.



Map 16

FLOOD CONTROL ALTERNATIVE PLAN NO. 1: STRUCTURE FLOODPROOFING, ELEVATION, AND REMOVAL

LEGEND

🟥🟨 100-YEAR RECURRENCE INTERVAL FLOODPLAIN- ULTIMATE PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

🟨 100-YEAR RECURRENCE INTERVAL FLOODPLAIN- ULTIMATE PLANNED LAND USE, STORMWATER DRAINAGE ALTERNATIVE 3 CONDITIONS WITH QUANTITY CONTROL PORTIONS OF DETENTION BASINS WD7, WD12, WD14, AND DD5 ELIMINATED, AND EXISTING CHANNEL CONDITIONS

0.5 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

▲ SINGLE FAMILY STRUCTURE TO BE FLOODPROOFED

● SINGLE FAMILY STRUCTURE TO BE ELEVATED

☒ SINGLE FAMILY STRUCTURE TO BE REMOVED

PROPOSED BRIDGE REMOVAL

PROPOSED NEW CULVERT

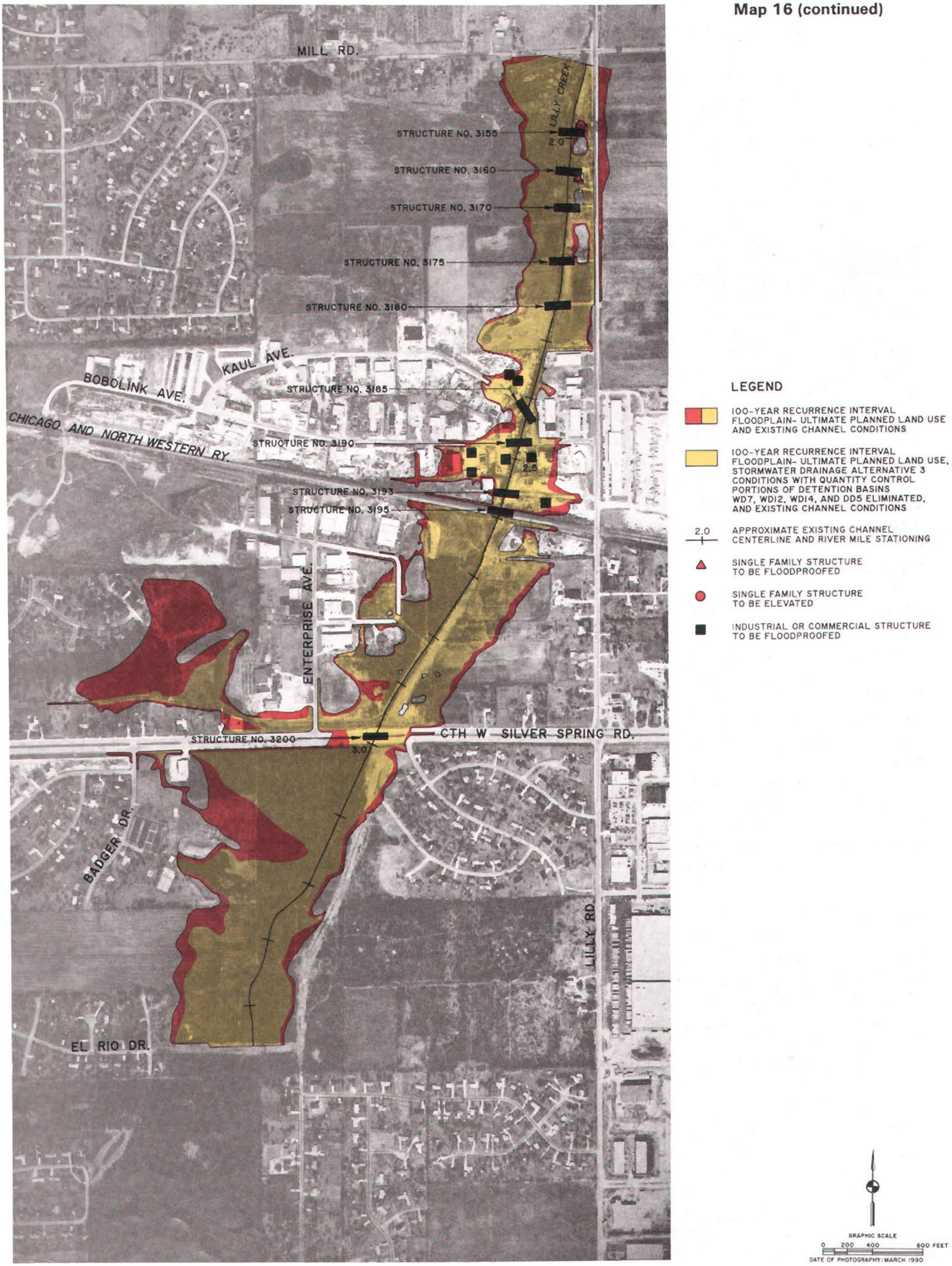
PROPOSED CHANNEL MODIFICATION AND REALIGNMENT

GRAPHIC SCALE

0 200 400 800 FEET

DATE OF PHOTOGRAPHY: MARCH 1990

Map 16 (continued)



Source: SEWRPC.

This alternative would also include replacement of the bridges at Lilly Road and W. Mill Road with a single 275-foot-long structure aligned across the intersection of W. Mill and Lilly Roads. The replacement structure would consist of two eight-foot-wide by five-foot-high reinforced concrete box culverts. The length and alignment of the proposed replacement structure would be the same as proposed by the Village's 1984 channel modification project. Because of the proposed alignment, some limited channel widening and deepening would be required along the east side of Lilly Road in the 340-foot reach downstream of the double box culvert. The existing Lilly Road bridge is badly deteriorated and is in need of replacement on structural considerations alone. In the future, the Village anticipates widening Lilly Road from the present 24-foot pavement on 50-foot-wide right-of-way to two 28-foot pavements with a 22-foot median on a 100-foot right-of-way and also widening W. Mill Road at its intersection with Lilly Road from its present 24-foot pavement on 50-foot right-of-way to two 28-foot pavements on a 100-foot right-of-way. The proposed replacement structure would accommodate the future widening of W. Mill Road and Lilly Road and also eliminate the right-angle turns in the stream channel on the upstream and downstream sides of the existing Lilly Road bridge. The new structure would allow replacement of the existing Lilly Road Bridge with a small culvert designed to pass localized runoff only. Because the bridge replacement would be implemented as part of the arterial street improvements recommended in the Village of Menomonee Falls Land Use and Transportation Plan, the costs of replacement and associated channel modifications are not included as part of this alternative flood control plan.

The total capital cost of the structure floodproofing, elevation, and removal alternative is estimated to be $556,000. This cost includes $55,000 for floodproofing of 10 residential buildings, $53,000 for floodproofing of seven industrial or commercial buildings, $231,000 for elevation of five residential buildings; and $217,000 for removal of two residential buildings. Utilizing an annual interest rate of 6 percent and a project life and amortization period of 50 years, the average annual cost of the alternative plan is estimated at $35,300. The average annual flood damage abatement benefit, assuming full implementation of the refined preliminary recommended stormwater drainage plan, is estimated to be $64,700, yielding a benefit-cost ratio of 1.8.

Alternative Flood Control Plan No. 2: Acquisition and Removal of Structures: The second alternative plan considered calls for the purchase and removal of all 24 buildings which would remain in the 100-year recurrence interval floodplain under planned land use and existing channel conditions, assuming full implementation of the recommended stormwater drainage plan. As set forth in Appendix B of this report, the acquisition and removal cost for single-family residential buildings was calculated as the sum of the structure and site acquisition costs, based on the fair market values obtained from tax records, and a fixed cost of $18,000 which includes the costs of utility disconnection, demolition of structures, site restoration, and occupant relocation. The same procedure was used to compute the cost for removal of industrial and commercial buildings, except that a fixed cost of $50,000 per owner was used for utility disconnection, demolition of structures, site restoration, and occupant relocation. The buildings to be removed are shown on Map 17.

Full implementation of this alternative plan would serve to eliminate flood damages due to direct overland flooding along Lilly Creek for floods up to and including the 100-year recurrence interval flood event under planned land use and channel conditions, assuming complete implementation of the refined preliminary recommended stormwater drainage element of the system plan. That stormwater drainage element eliminates the water quantity control portions of detention basins WD7, WD12, WD14, and DD5.

This alternative would also include replacement of the bridges at Lilly Road and W. Mill Road with a single 275-foot-long structure aligned across the intersection of W. Mill and Lilly Roads. The replacement structure would consist of two eight-foot-wide by five-foot-high reinforced concrete box culverts. Because the bridge replacement would be required to carry out arterial street improvements recommended in the Village Land Use and Transportation Plan, the costs of replacement and associated channel modifications are not included as part of this alternative flood control plan.

The total capital cost of the acquisition and removal alternative is estimated to be $2,684,000. This cost includes $1,800,000 for the acquisition and removal of 17 single-family residential buildings and $884,000 for the acquisition and removal of seven industrial and commercial buildings. Utilizing an annual interest rate of 6 percent and a project life and amortization period of 50 years, the average annual cost of the alternative plan is estimated at $170,400. The average annual flood damage abatement benefit, assuming full implementation of the refined preliminary recommended stormwater drainage plan, is estimated to be $64,700, yielding a benefit-cost ratio of 0.4.

Alternative Flood Control Plan No. 3: Channel Modification and Removal or Replacement of Bridges: The third alternative plan considered calls for the construction of a 2.53-mile-long widened and deepened channel which would essentially be located along the alignment of the existing Lilly Creek channel. As shown on Map 18, the proposed channel modifications would extend from a point 0.21 mile upstream of the mouth of the creek to a point about 0.24 mile downstream of W. Silver Spring Road. The modified flood control channel, a typical section of which is shown on Figure 21, would be trapezoidal in shape with a turf lining, a four- to 12-foot bottom width, one vertical on three horizontal side slopes, and an average depth of about 10 feet. A one-foot-deep, four-foot-wide riprap-lined low-flow channel would be provided in the bottom of the flood control channel.

The alternative calls for the replacement of the existing bridges at Brentwood Drive, Lilly Road, and W. Mill Road, three private drives between Mill Road and the Chicago & North Western Railway embankment, Kaul Avenue, and Bobolink Avenue. The alternative also calls for removal of three pedestrian bridges located at River Miles 1.99, 2.05, and 2.11 and replacement of those three bridges with two structures. Full implementation of this alternative plan would serve to eliminate flood damages due to direct overland flooding along Lilly Creek for floods up to and including the 100-year recurrence interval flood event under planned land use and channel conditions, assuming complete implementation of the recommended stormwater drainage element of the system plan.

The Village of Menomonee Falls has purchased easements along an alignment proposed in 1984 for a channel modification project which was not constructed. With the exception of an 0.38-mile-long reach of channel immediately upstream of Mill Road, the channel alignment proposed under this alternative would be essentially the same as that provided by the existing easement. The shape, side slopes, and dimensions of the modified flood control channel cross-section are the same as those proposed by the Village under the earlier project proposal. In addition, with the exception of a short, 0.18-mile-long reach between W. Good Hope Road and Brentwood Drive, the flood-control channel streambed profile proposed here is the same, or somewhat shallower, than that proposed by the Village under the earlier project proposal. As a result, in general, the proposed channel modification would fit within the easements already obtained by the Village.

From the downstream end of the proposed project at River Mile 0.21 to Kaul Avenue at River Mile 2.43, the modified channel would have a 12-foot bottom width with appropriate transitions in bottom width to accommodate bridges. Within that reach, the existing streambed would be lowered a maximum of about 7.7 feet below the existing bed. The existing three nine-foot-wide by 11.5-foot-high reinforced concrete box culverts at W. Appleton Avenue were constructed with inverts about 4.5 feet below the existing streambed. Those culverts would be retained and the streambed would be lowered about 3.5 feet within the box culverts. The existing 25.5-foot-wide by 16.75-foot-high elliptical structural plate pipe culvert at W. Good Hope Road was constructed with its invert about seven feet below the existing streambed. That pipe would be retained and the streambed would be lowered 3.3 feet within the pipe. The existing bridge at Brentwood Drive would be replaced with two 10-foot-wide by eight-foot-high reinforced concrete box culverts. The streambed would be lowered about 4.3 feet at the culverts.

This alternative would also include replacement of the bridges at Lilly Road and W. Mill Road with a single 275-foot-long structure aligned across the intersection of W. Mill and Lilly Roads. The replacement structure would consist of two 10-foot-wide by seven-foot-high reinforced

Map 17

FLOOD CONTROL ALTERNATIVE PLAN NO. 2: ACQUISITION AND REMOVAL OF STRUCTURES

LEGEND

🟥🟨 100-YEAR RECURRENCE INTERVAL FLOODPLAIN- ULTIMATE PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

🟨 100-YEAR RECURRENCE INTERVAL FLOODPLAIN- ULTIMATE PLANNED LAND USE, STORMWATER DRAINAGE ALTERNATIVE 3 CONDITIONS WITH QUANTITY CONTROL PORTIONS OF DETENTION BASINS WD7, WD12, WD14, AND DD5 ELIMINATED, AND EXISTING CHANNEL CONDITIONS

0.5 ┼ APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

☒ SINGLE FAMILY STRUCTURE TO BE REMOVED

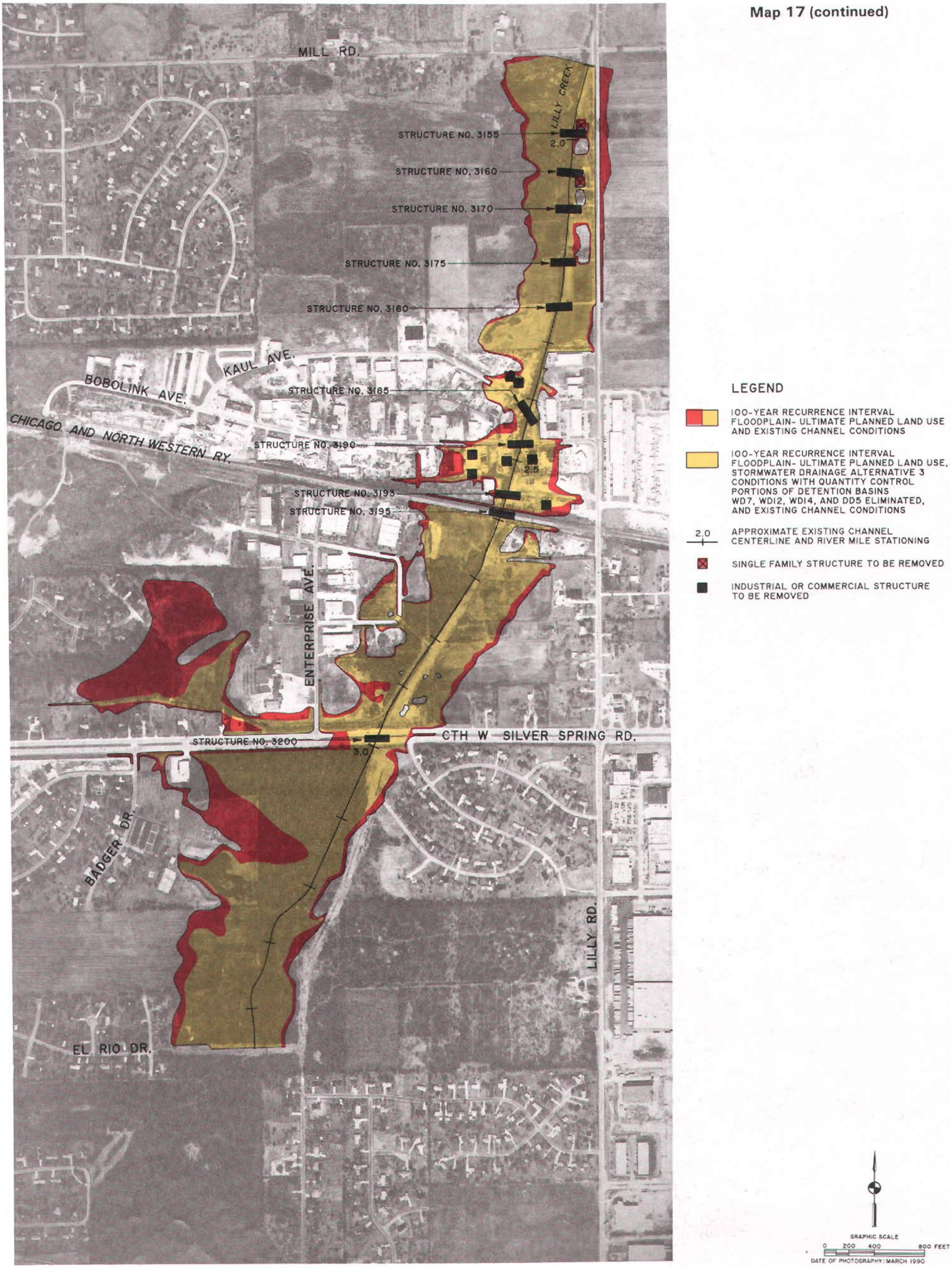
☒ PROPOSED BRIDGE REMOVAL

— PROPOSED NEW CULVERT

— PROPOSED CHANNEL MODIFICATION AND REALIGNMENT

GRAPHIC SCALE
0 200 400 800 FEET
DATE OF PHOTOGRAPHY: MARCH 1990

Map 17 (continued)



Source: SEWRPC.

Map 18

FLOOD CONTROL ALTERNATIVE PLAN NO. 3: CHANNEL MODIFICATION AND REMOVAL OR REPLACEMENT OF BRIDGES

LEGEND

🟥🟨 100-YEAR RECURRENCE INTERVAL FLOODPLAIN- ULTIMATE PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS

🟨 100-YEAR RECURRENCE INTERVAL FLOODPLAIN- ULTIMATE PLANNED LAND USE, PLANNED DRAINAGE AND CHANNEL CONDITIONS (CONTAINED WITHIN MODIFIED CHANNEL EXCEPT WHERE SHOWN OUTSIDE OF CHANNEL)

0.5 APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING

☒ PROPOSED BRIDGE REMOVAL

▥ PROPOSED BRIDGE REPLACEMENT

— PROPOSED NEW CULVERT

PROPOSED CHANNEL WIDENING AND DEEPENING

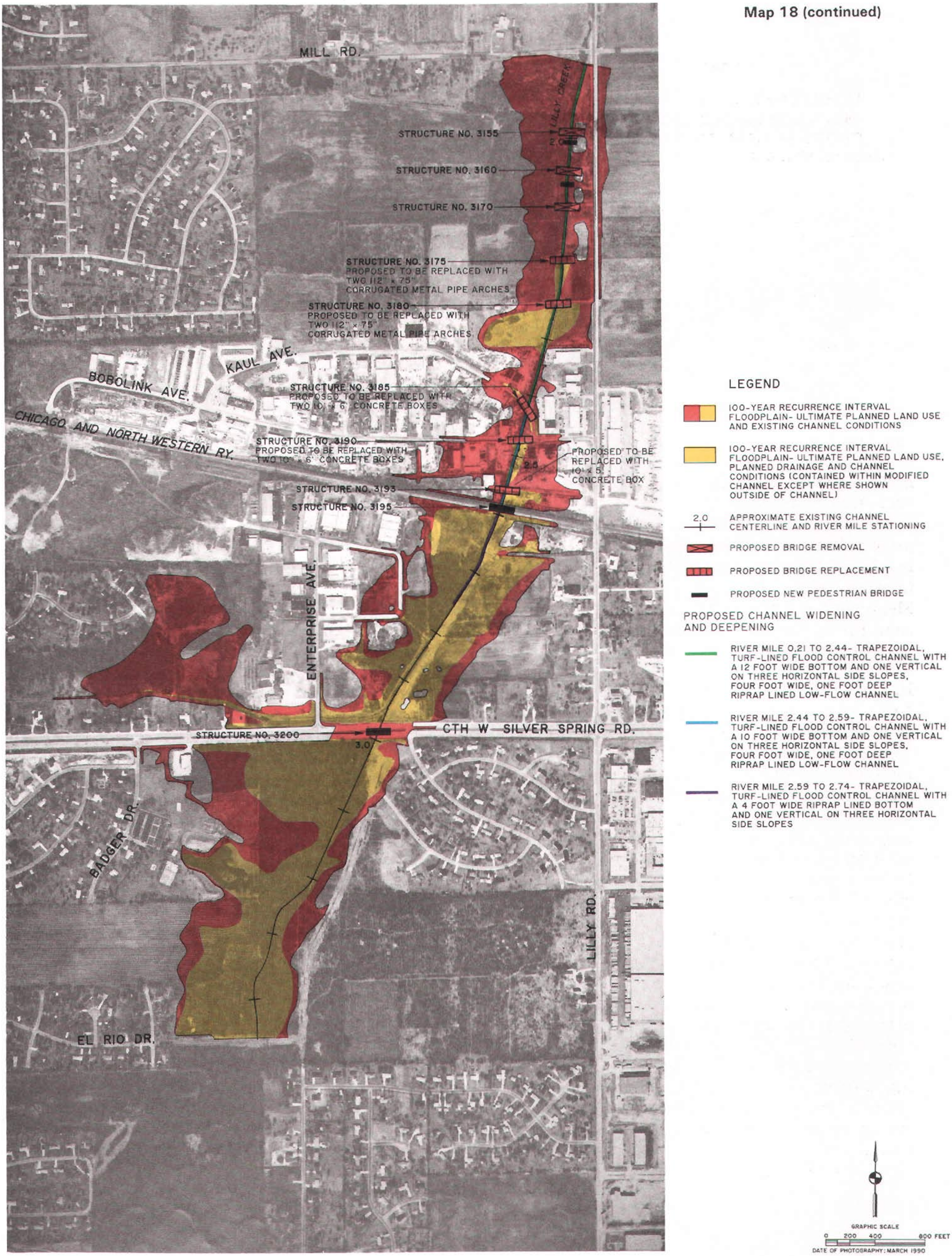
— RIVER MILE 0.21 TO 2.44- TRAPEZOIDAL, TURF-LINED FLOOD CONTROL CHANNEL WITH A 12 FOOT WIDE BOTTOM AND ONE VERTICAL ON THREE HORIZONTAL SIDE SLOPES, FOUR FOOT WIDE, ONE FOOT DEEP RIPRAP LINED LOW-FLOW CHANNEL

PROPOSED CHANNEL REALIGNMENT

- - - TRAPEZOIDAL, TURF-LINED FLOOD CONTROL CHANNEL WITH A 12 FOOT WIDE BOTTOM AND ONE VERTICAL ON THREE HORIZONTAL SIDE SLOPES, FOUR FOOT WIDE, ONE FOOT DEEP RIPRAP LINED LOW-FLOW CHANNEL

GRAPHIC SCALE
0 200 400 800 FEET
DATE OF PHOTOGRAPHY: MARCH 1990

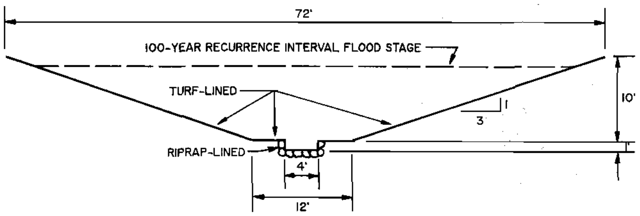
Map 18 (continued)



Source: SEWRPC.

Figure 21

FLOOD CONTROL ALTERNATIVE PLAN NO. 3: TYPICAL MODIFIED CHANNEL CROSS-SECTION ALONG LILLY CREEK FROM RIVER MILE 0.21 TO RIVER MILE 2.44



Source: SEWRPC.

concrete box culverts. The existing streambed would be lowered about seven feet at the replacement structure. The length and alignment of the proposed structure would be the same as those proposed in the 1984 channel modification project. Because the bridge replacement would be implemented as part of the arterial street improvements recommended in the Village of Menomoneee Falls Land Use and Transportation Plan, the cost of replacement was not assigned to this alternative flood control plan.

In the reach from W. Mill Road at River Mile 1.88 to the private drive at River Mile 2.26, the 1984 channel modification proposed by the Village called for the modified channel to be realigned and moved a maximum of about 105 feet to the west of the existing channel. The alignment proposed by the Village would have eliminated the need for replacement of the existing pedestrian bridges which provide access to the west side of properties which lie on either side of the existing channel. Four easements required for that channel relocation were not obtained by the Village because of objections by property owners. Because the alignment proposed by the Village was not accepted by the four property owners, the channel modification proposed under this alternative would follow the approximate alignment of the existing stream with a minor shift to the west to avoid encroaching too closely on existing buildings. If this alternative were to be adopted and if at the time of implementation it was found to be possible to obtain the remaining easements for a channel along the alignment originally proposed by the Village, the use of such an alignment would not significantly alter the upstream or downstream components of the channel modification alternative as herein presented.

This alternative includes removal of the three pedestrian bridges at River Miles 1.99, 2.05, and 2.11 upstream of W. Mill Road and replacement of those bridges with two structures which would cause an insignificant obstruction to flows under 100-year recurrence interval conditions. The private drive bridges at the Brahm property at River Mile 2.20 and the Weyer property at River Mile 2.26 would each be replaced with double 112-inch-wide by 75-inch-high corrugated metal pipe arches. The streambed would be lowered about 3.5 feet at each of those proposed structures.

The existing bridge at Kaul Avenue would be replaced with a double 10-foot-wide by six-foot-high reinforced concrete box and the streambed would be lowered about 3.5 feet at that location.

Beginning upstream of Kaul Avenue at River Mile 2.44 and extending to the Chicago & North Western Railway embankment at River Mile 2.59, a 10-foot modified channel bottom width, with appropriate transitions at bridges, would be provided. The Bobolink Avenue bridge would be replaced with a double 10-foot-wide by six-foot-high reinforced concrete box culvert and the streambed would be lowered 3.5 feet. The private drive culvert at River Mile 2.55 would be replaced with a single 10-foot-wide by five-foot-high reinforced concrete box culvert. Upon construction of the recommended wet detention basin located to the west of that proposed box culvert, the drive could be used for access to the detention basin for maintenance.

The streambed would be lowered about 3.2 feet at the Chicago & North Western Railway bridge, but the channel width would be limited to the bridge width and the sides would be sloped so as to avoid interference with the bridge foundation. In the 0.15-mile-long reach upstream of the railway bridge to River Mile 2.74 the widened and deepened channel would have a four-foot bottom width and one vertical on three horizontal side slopes. The existing streambed profile and channel cross-section would be maintained from River Mile 2.74 through the upstream end of the Lilly Creek and the four existing culverts at Silver Spring Drive would remain in place.

From Brentwood Drive at River Mile 1.06 through the intersection of Manor Hills Boulevard and Oakwood Drive at River Mile 1.35, the two lanes of Manor Hills Boulevard are located along both sides of the existing stream channel. The proposed channel widening and deepening would require relocation of both sides of the boulevard. The Village's 1984 channel modification design provided for such relocation. The channel modification proposed here could be accommodated with the relocation originally proposed by the Village.

The loss of overbank storage in the floodplain resulting from confinement of flows to the proposed channel would increase flood flows. In order to partially offset that increase and to limit the peak 100-year recurrence interval flood flow at the mouth of the Lilly Creek to no more than the existing peak flow, the water quantity control components of detention basin WD12 and dry detention basin DD5, which are located in Hydrologic Unit E southeast of the intersection of Lilly and W. Mill Roads, would be incorporated into the stormwater drainage plan under this flood control alternative. Those detention basins were eliminated during the refinement of the preliminary recommended stormwater management plan and are not included under Alternative Nos. 1 and 2. The 100-year recurrence interval flows in Lilly Creek which were used for Alternative No. 3 are set forth in Table 47.

The total capital cost of the channel modification and bridge removal or replacement alternative is estimated to be $2,554,000. This cost includes $1,521,000 for construction of the widened and deepened channel, $415,000 for bridge removal and replacement, $348,000 for two additional detention basins for water quantity control, $250,000 for reconstruction and relocation of Manor Hills Boulevard, and $20,000 for additional easements not already obtained by the Village. Utilizing an annual interest rate of 6 percent and a project life and amortization period of 50 years, the average annual cost of the alternative plan, including $12,000 annual operation and maintenance costs, is $174,200. The average annual flood damage abatement benefit, assuming full implementation of the refined preliminary recommended stormwater drainage plan, is estimated to be $64,700, yielding a benefit-cost ratio of 0.4.

Comparison and Evaluation of Flood Control Alternative Plan Nos. 1 through 3

The alternative plans were compared with respect to cost, implementability, potential impacts on the stormwater drainage system, potential environmental impacts, and potential impacts on public health and safety. The costs of the alternative plans are provided in Table 48.

Costs: The capital cost of Alternative Plan No. 1: Structure Floodproofing, Elevation, and Removal, is by far the lowest of the three alternative plans. The capital costs of Alternative Plan No. 2: Acquisition and Removal of Buildings, and Alternative Plan No. 3: Channel Modification and Bridge Removal or Replacement are approximately five times greater than the cost of Alternative Plan No. 1. The capital cost of Alternative No. 3 is slightly lower than that of Alternative No. 2, but the average annual cost of Alternative No. 3 is slightly higher because of the addition of operation and maintenance costs for Alternative No. 3.

Alternative No. 3 is clearly superior to Alternative No. 2. Because the costs of these two alternatives are about the same and because, even with complete implementation of Alternative No. 2, there would still be residual problems of potential secondary flooding of basements of buildings located outside of the 100-year recurrence interval floodplain, site and building access problems where street flooding would occur, and potential limitations on the adequate operation of certain storm sewers that would have submerged outlets due to relatively high flood levels in Lilly Creek. Most of the potential disadvantages of Alternative No. 2 would be eliminated or greatly reduced through implementation of Alternative No. 3, and at a comparable cost. Alternative No. 2 is, therefore, eliminated from further consideration and the following discussions of the advantages and disadvantages of the alternatives is focused on Alternative Nos. 1 and 3.

Nonquantifiable Advantages and Disadvantages of the Alternative Plans: Alternative No. 1 requires floodproofing of 17 buildings, elevation of five buildings, and removal of two buildings. Because such floodproofing would be voluntary, complete implementation of that alternative may be difficult and, therefore, there may be the possibility of significant residual flooding problems remaining if that alternative were selected.

An objection is often raised to floodproofing and elevation because it has in some cases been defined as a private cost to be borne by the property owner. If the Village would assume these costs, the implementability of this alternative would be more comparable to that of Alternative No. 3. The possibility of implementation of Alternative No. 3 is improved because the Village has already acquired most of the easements needed for construction. However, obtaining the remaining easements may be difficult. Furthermore, the need to obtain regulatory approvals may also make implementation difficult. Alternative No. 3 would eliminate residual flooding problems because it provides a solution which reduces the 100-year recurrence interval floodplain limits, thereby removing all existing buildings from the floodplain.

Even with complete implementation of Alternative No. 1, localized flooding of streets and yards adjacent to Lilly Creek would still occur. However, neither W. Silver Spring Drive nor the intersection of W. Mill Road and Lilly Road would be overtopped during a 50-year flood; therefore, the planning standard for overtopping of arterial streets and highways as set forth in Chapter IV of this report would be met. All three roadways would be overtopped, however, during a 100-year recurrence interval flood under Alternative No. 1. Under 100-year flood conditions, W. Silver Spring Drive would be expected to be overtopped to a depth of about 0.2 foot and the W. Mill-Lilly Road intersection would be expected to be overtopped to a depth of about 0.3 foot. Those depths of overtopping would not prevent the movement of vehicular traffic, including emergency vehicles. Boldly marked staff gages which would indicate the depth of flooding above the pavement could be provided to assist motorists in determining whether flooded streets or intersections were passable and the pavement edges could be delineated by reflective marks on posts.

During the peak of a 100-year recurrence interval flood under Alternative No. 1, Manor Hills Boulevard would be expected to be flooded to a maximum depth of approximately three feet, making that roadway impassable to any vehicular traffic. A similar situation would exist under 50-year flood conditions, although the depth of flooding would be several tenths of a foot less. The intersection of Oakwood Drive and Manor Hills Boulevard would be flooded to a depth of about 0.3 foot under 100-year conditions. With the exception of properties located along about an 0.15-mile-long stretch of Manor Hills Boulevard, including Manor Hill Court, access to and egress from Manor Hills Boulevard, Bay Ridge Lane, Bay Ridge Court, Brentwood Drive, Ranch Road, and Oakwood Boulevard could be obtained through Lilly Road or Claas Road, both of which would remain passable.

Also under Alternative No. 1, the bridge crossings at Kaul Avenue and Bobolink Avenue would be overtopped by a maximum of 1.6 to 1.8 feet during floods with recurrence intervals of 10 years or less and by about 2.2 feet during a 100-year recurrence interval flood. Those roadways are classified as land access streets which should not be overtopped during a 10-year flood, according to the standard set forth in Chapter IV of this report. The roads would be potentially impassable under 10- to 100-year recurrence interval flood conditions. Since Bobolink Avenue is the only road leading into the Bowling Green Industrial Park, there would be no reliable dry-land access to the industrial park during floods. That situation could be resolved under the floodproofing alternative by extending Bobolink Avenue 1,000 feet to the west to intersect with Pilgrim Road. The approximate capital cost of that road extension, including additional stormwater drainage facilities, would be $200,000. Such a road would pass near the southern boundary of an area of planned medium-density residential development. It would be necessary to design the road alignment to minimize the impacts of industrial truck traffic on the adjacent neighborhood.

Under Alternative No. 3, with the exception of Kaul Avenue and Bobolink Avenue, localized flooding of streets and yards adjacent to Lilly Creek would essentially be eliminated during floods up to and including a 100-year recurrence interval flood. The Kaul and Bobolink Avenue crossings would be overtopped by only 0.1 foot during a 100-year recurrence interval flood and they would meet the applicable standard of not being overtopped during a 10-year flood. Thus, Alternative No. 3 provides much better street and land access during storm events than does Alternative No. 1.

Another potential impact of the street flooding under Alternative No. 1 is secondary basement flooding of buildings outside the 100-year recurrence interval floodplain. Such secondary flooding could occur because of backup of sanitary sewers or through infiltration through basement walls and floors. In addition to the potential flood hazard, a health hazard would also be presented by the potential backup of raw sewage into basements. Such secondary flooding would be essentially eliminated under Alternative No. 3.

Under Alternative No. 1 there would still be the requirement of obtaining flood insurance when securing a mortgage for a floodproofed or elevated home located within the floodplain. Since all existing buildings would be removed from the 100-year floodplain of Lilly Creek under Alternative No. 3, the Village of Menomonee Falls floodplain maps could be amended following implementation of that alternative and thus those homes could be freed from the flood insurance requirement.

At Kaul Avenue and Bobolink Avenue, the efficient operation of proposed storm sewers which discharge directly to Lilly Creek would be hindered by the considerable submergence of the outlets which could occur during floods under Alternative No. 1. Under Alternative No. 3, those conditions would be alleviated.

Because Alternative No. 1 would essentially maintain the existing stream channel, its implementation would involve a less disturbance to aquatic and riparian habitat than would the implementation of Alternative No. 3. However, it would be possible to incorporate in-stream habitat mitigation measures in the channel which could ultimately improve the aquatic habitat over the existing condition. Such measures would include a meandering low-flow channel with alternating pool and riffle sections, habitat mitigation structures to provide cover for fish and aquatic life and to aid in the maintenance of the pool and riffle sequence in the low-flow channel, and culverts designed to permit fish migration within the stream. The incorporation of similar measures to improve habitat would also be desirable under Alternative No. 1; therefore, either alternative could provide a similar degree of improvement in aquatic habitat.

Selection of the Preliminary Recommended Flood Control Plan for the Main Stem of Lilly Creek

The stormwater drainage and nonpoint source pollution control alternatives and flood control Alternative Nos. 1 through 3 as set forth in this chapter were presented by Commission staff to the Village of Menomonee Falls Lilly Creek Stormwater Management and Flood Control Advisory Committee at its meeting on September 20, 1990. Also present at that meeting were the Village's Director of Public Works, Director of Community Development, and Superintendent of Engineering, as well as members of the Wisconsin Department of Natural Resources staff.

The Advisory Committee supported flood control Alternative No. 3: Channel Modification and Bridge Removal or Replacement. The Committee members' endorsement of the channel modification alternative was based on their perception that this alternative would be the most readily implementable and on their desire to eliminate the possibility of significant street flooding and secondary flooding of basements in addition to the elimination of direct overland flooding. It was the opinion of one member of the Committee that flood control Alternative Nos. 1 and 2 were not implementable and that the Village's choice was between no action to control flooding and flood control Alternative No. 3. That opinion was not challenged by the other Committee members.

The Wisconsin Department of Natural Resources representatives stated that, in order for the flood control element of the plan to obtain Department approval, refinements would be necessary to further enhance the biological habitat, aesthetic, and recreational aspects of the flood control element. On the basis of the Advisory Committee preference for flood control Alternative No. 3 and on the indication by the Department of Natural Resources representatives that such a plan could be acceptable to the Department if additional refinements were made, flood control Alternative Plan No. 3 was selected as the preliminary recommended flood control plan element, subject to refinement and modification to address DNR concerns related to the improvement of biological habitat, in-stream and riparian aesthetic characteristics, and the provision of recreational benefits.

At the meeting, the DNR specifically requested that additional consideration be given to the effects on the flood control alternative of: 1) providing detention storage along the Bowling Green Tributary for quantity control of runoff from the Bowling Green Industrial Park in Hydrologic Unit C and 2) increasing the flood storage capacity of the modified channel and decreasing flood flow velocities by widening the channel in the reach south of the intersection of Oakwood Drive and Manor Hills Boulevard and east of Lilly Road. The Village Director of Community Development concurred in the DNR request that additional storage be investigated for the Bowling Green Industrial Park. Following the meeting, Department staff not present at the meeting also requested that consideration be given to the use of a shallower, but wider, modified channel cross-section in the reach of Lilly Creek between W. Appleton Avenue and W. Good Hope Road. The purpose of such a channel section would be to eliminate the need for channel widening or deepening downstream of W. Appleton Avenue and to provide a reach with lower flow velocities where fish could take refuge during floods. Another flood control alternative suggested by the DNR staff following the Committee meeting was bridge replacement and roadway elevations to reduce street flooding in areas where vehicular access is limited. That alternative was expanded to include bridge replacement for the reduction of structure flooding and is presented as flood control Alternative No. 4 in the next section of this chapter.

On October 31, 1990, a meeting of the staffs of the Village, the Department, and the Commission was held to formulate a strategy for proceeding with the completion of the stormwater management and flood control plan. All affected parties were in agreement that the preliminary recommended stormwater management plan element would meet the plan objectives; therefore, the discussion centered on an approach to complete the flood control element of the plan. Department staff reiterated the position that the channel modification alternative would require revision to address the issues of enhancement of biological habitat, improvement of in-stream and riparian aesthetics, and the provision of recreational opportunities. Because of the need to complete the plan in a timely matter and because Department and Commission staff members with expertise in the required fields were already committed to other projects, it was mutually agreed that the Village would retain an additional consultant, or consultants, to work with the Village, the Department, and the Commission to develop a flood control alternative which refined the preliminary recommended alternative by addressing habitat, aesthetic, and recreational considerations in more detail. It was also agreed that the hydraulic and hydrologic analyses of that alternative would be conducted by the Commission staff. The preparation of the additional alternative would be funded by a grant from the State of Wisconsin.

The hydraulic, hydrologic, and fiscal impacts of that additional flood control alternative are quantified and evaluated in Chapter VI of this report, which sets forth the recommended stormwater drainage and flood control plan for the subwatershed.

Additional Analyses Following Review of the Plan by the Village Advisory Committee

As requested by the Village and the Department of Natural Resources, the possible inclusion of detention storage for quantity control of runoff from the Bowling Green Industrial Park was investigated by Commission staff. In comparison to flood control Alternative No. 3, the provision of 12.5 acre-feet of additional detention storage would decrease the planned condition 100-year recurrence interval flood flow along Lilly Creek up to 10 percent in the first 0.55 mile reach downstream of the detention site. Downstream of that reach 100-year recurrence interval flood flows would be reduced by a maximum of only 3 percent. At the mouth of Lilly Creek, the additional detention storage would reduce the 100-year flood flow from 2,540 cubic feet per second (cfs) to 2,500 cfs. Therefore, the provision of the additional detention storage in the industrial park would not decrease flood flows enough to effect any significant reduction in the scope of the flood control alternatives.

As requested by the Department, a preliminary investigation was made by Commission staff of the effects of widening the modified channel in the reach south of the intersection of Oakwood Drive and Manor Hills Boulevard and east of Lilly Road and also of using a shallower, but wider, modified channel cross-section in the reach between W. Appleton Avenue and W. Good Hope Road. The widened and deepened modified channel sections in the two reaches were designed so that the 100-year recurrence interval floodplain width under planned land use and planned channel conditions would approximate the 100-year floodplain width under planned land use and existing channel conditions. In the upstream reach south of the intersection of Oakwood Drive and Manor Hills Boulevard, the top width of the modified channel would increase from about 60 feet under flood control Alternative No. 3 to between 210 feet and 380 feet. In the reach between W. Appleton Avenue and W. Good Hope Road, the modified channel top width, which would range from 80 feet to 260 feet under flood control Alternative No. 3, would be expanded to between 150 feet and 380 feet. The additional flood storage in the wider channel would decrease 100-year recurrence interval flood flows from 20 to 26 percent below the flows for the narrower modified channel proposed under flood control Alternative No. 3. At the mouth of Lilly Creek, the 100-year flood flow would decrease from 2,540 cfs to 1,880 cfs. Thus, these refinements to the channel modification alternative would have sufficient impact on 100-year recurrence interval flood flows and stages to enable some reduction in the degree of channel deepening proposed in the original channel modification alternative. This alternative was accordingly considered further in the final plan set forth in Chapter VI. Any channel widening beyond the limits of the easements already obtained by the Village would have to be carefully designed to avoid conflicts with the recently-constructed trunk sewer along Lilly Creek.

Alternative Flood Control Plan No. 4: Bridge Replacement, Road Elevation, and Structure Floodproofing, Elevation, and Removal: As mentioned above, this alternative was developed as a result of discussions with the DNR staff following the September 20, 1990, meeting of the Village's advisory committee.

This alternative plan calls for the replacement of seven bridges, while essentially maintaining the existing Lilly Creek stream channel. The alternative also proposes raising Kaul and Bobolink Avenues to provide access to, and egress from, the Bowling Green Industrial Park during floods up to and including a 100-year recurrence interval flood. In order to provide protection for all existing buildings in the 100-year floodplain it would also be necessary to floodproof seven residential buildings and seven industrial buildings, to elevate four residential buildings, and to remove one residential building.[8] The components of this alternative are shown on Map 19.

Full implementation of this alternative plan would serve to eliminate flood damages due to direct overland flooding along Lilly Creek from floods up to and including the 100-year recurrence interval flood under planned land use and channel conditions, assuming complete implementation of the recommended stormwater drainage element of the system plan. The recommended plan eliminates the water quantity control portions of detention basins WD7, WD12, WD14, and DD5. The 100-year recurrence interval flows in Lilly Creek under the recommended plan which were used in the design and evaluation of this alternative are given in Table 47.

This alternative calls for the replacement of eight existing bridges; those at W. Appleton Avenue, W. Good Hope, W. Mill, and Lilly Roads, Brentwood Drive, Lilly Road, W. Mill Road, the private drive at River Mile 2.20, Kaul Avenue, and Bobolink Avenue. The existing reinforced concrete triple box culvert at W. Appleton Avenue would be replaced with a two-span bridge with a total length of 80 feet. The existing elliptical structural plate pipe culvert at W. Good Hope Road would be replaced with a two-span bridge with a total length of 122 feet. The existing bridge at Brentwood Drive would be replaced with a 28-foot-long, single span bridge. The existing bridges at Lilly Road and W. Mill Road would be replaced with a single structure, consisting of three 275-foot-long, 10-foot-wide by five-foot-high reinforced concrete box culverts. The existing private bridge at River Mile 2.20 would be replaced with two 10-foot-wide by six-foot-high reinforced concrete box culverts. The existing bridges at Kaul and Bobolink Avenues would each be replaced with a 45-foot-long, single-span bridge. Because the W. Mill and Lilly Road bridge replacement would be implemented as part of the arterial street improvements recommended in the Village Land Use and Transportation Plan, the cost of replacement was not assigned to this alternative flood control plan. The Village Land Use and Transportation Plan recommends widening W. Good Hope Road to four lanes; however, the existing structure could accommodate such widening without modification. The cost of the proposed replacement bridge was therefore assigned to this alternative flood control plan because the bridge would be provided solely for flood control purposes.

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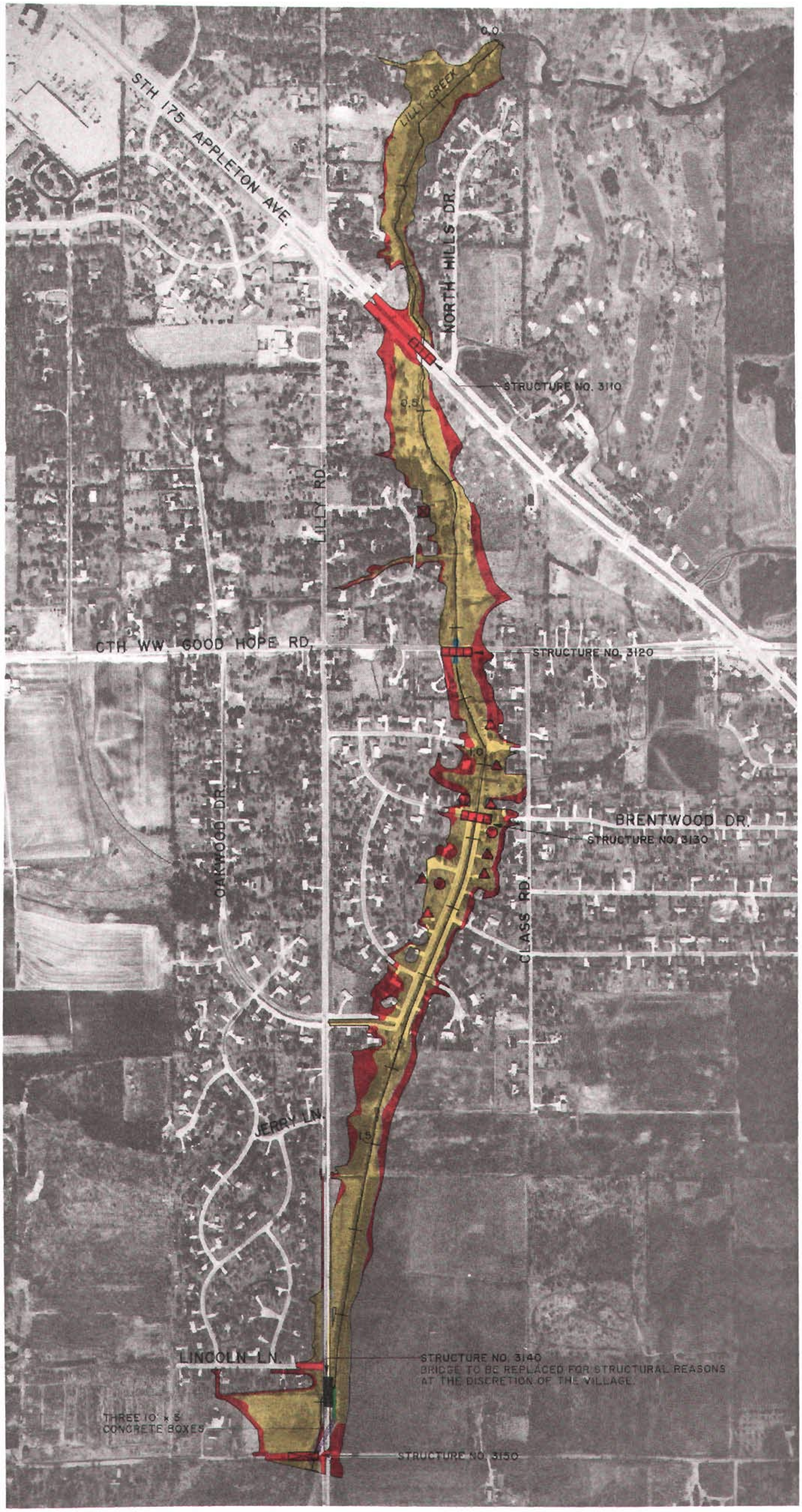
[8] *An additional industrial building which would require floodproofing if left in place is recommended to be purchased and removed to provide right-of-way for a reach of channel which is proposed to be widened to accommodate the recommended bridge at Bobolink Avenue.*

Map 19

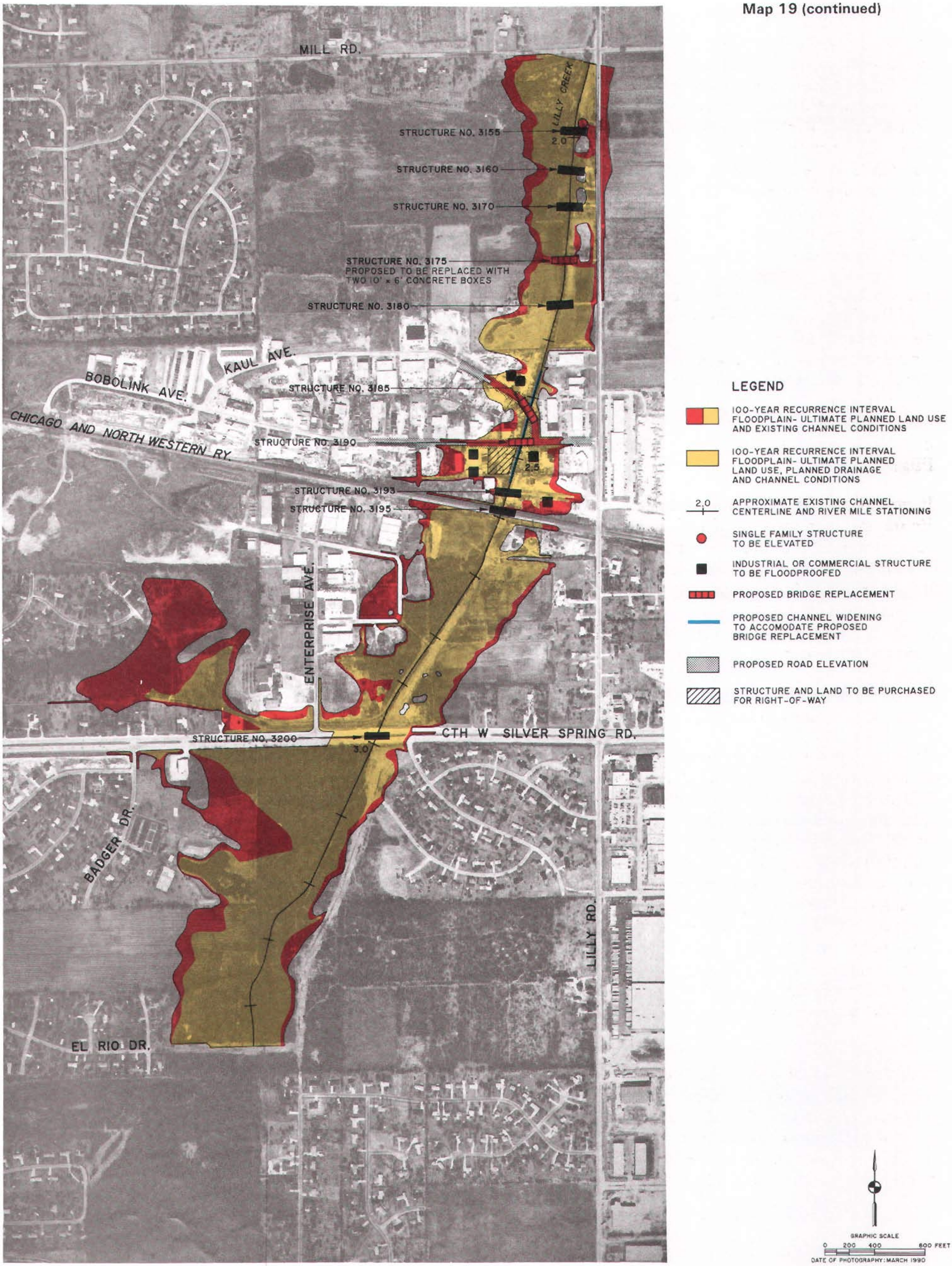
FLOOD CONTROL ALTERNATIVE PLAN NO. 4: BRIDGE REPLACEMENT; ROAD ELEVATION; AND STRUCTURE FLOODPROOFING, ELEVATION, AND REMOVAL



LEGEND

🟥🟨	100-YEAR RECURRENCE INTERVAL FLOODPLAIN- ULTIMATE PLANNED LAND USE AND EXISTING CHANNEL CONDITIONS
🟨	100-YEAR RECURRENCE INTERVAL FLOODPLAIN- ULTIMATE PLANNED LAND USE, PLANNED DRAINAGE AND CHANNEL CONDITIONS
0.5 ┼	APPROXIMATE EXISTING CHANNEL CENTERLINE AND RIVER MILE STATIONING
▲	SINGLE FAMILY STRUCTURE TO BE FLOODPROOFED
●	SINGLE FAMILY STRUCTURE TO BE ELEVATED
☒	SINGLE FAMILY STRUCTURE TO BE REMOVED
☒	PROPOSED BRIDGE REMOVAL
▥	PROPOSED BRIDGE REPLACEMENT
—	PROPOSED NEW CULVERT
—	PROPOSED CHANNEL MODIFICATION AND REALIGNMENT
—	PROPOSED CHANNEL WIDENING TO ACCOMODATE PROPOSED BRIDGE REPLACEMENT

Map 19 (continued)



Source: SEWRPC.

With the exception of limited channel widening required to accommodate the proposed bridges at W. Good Hope Road, River Mile 2.20, Kaul Avenue, and Bobolink Avenue, this alternative would involve no channel deepening or widening. Construction of the proposed bridge at Bobolink Avenue would require the purchase of one building to accommodate the widened channel in the immediate vicinity of the bridge. If not removed to accommodate the channel, that building would require floodproofing.

To permit dry-land access to, and egress from, the Bowling Green Industrial Park under 100-year recurrence interval flood conditions, approximately 0.38 mile of Kaul and Bobolink Avenues would have to be elevated under this alternative. Another option to provide such access for the industrial park would be to extend Bobolink Avenue about 1,000 feet to the west to Pilgrim Road.

It would not be feasible to elevate Manor Hills Boulevard to permit dry-land access under flood conditions. The boulevard runs parallel to Lilly Creek along both sides of the Creek; therefore, if it were raised, the roadway would function as a dike which would increase upstream flood stages. Also, raising the road would create interior drainage problems along the landward side of the road which could lead to stormwater ponding and could actually increase the flood hazard to adjacent residences.

The proposed bridge replacements would remove five single-family residential buildings from the 100-year recurrence interval floodplain under planned land use and channel conditions; however, in order to provide protection for all existing buildings in the 100-year floodplain, it would still be necessary to floodproof seven residential buildings and six industrial or commercial buildings, to elevate four residential buildings, and to remove one residential building. The criteria for the determination of whether to floodproof, elevate or remove are the same as those given for flood control Alternative No. 1.

The total capital cost of the bridge replacement, roadway elevation, and structure floodproofing, elevation, and removal alternative is estimated to be $2,322,000. This cost includes $1,730,000 for bridge replacement and associated channel transitions, easements, and building purchase; $190,000 for raising the grades of the roadways of Kaul and Bobolink Avenues; $39,000 for floodproofing seven residential buildings; $44,000 for floodproofing six industrial or commercial buildings; $182,000 for elevating four residential buildings; and $137,000 for removing one residential building. Utilizing an annual interest rate of 6 percent and a project life and amortization period of 50 years, the average annual cost of the alternative plan is estimated at $147,000. The average annual flood damage abatement benefit, assuming full implementation of the refined preliminary stormwater drainage plan, is estimated to be $64,700, yielding a benefit-cost ratio of 0.4.

The major advantages of flood control Alternative No. 4 over floodproofing Alternative No. 1 are that Alternative No. 4 would remove five buildings from the 100-year recurrence interval floodplain and would permit dry-land access to the Bowling Green Industrial Park during floods. The major disadvantage of Alternative No. 4 compared to Alternative No. 1 is the substantially higher cost of Alternative No. 4.

Although Alternative No. 4 would reduce the number of buildings required to be floodproofed, elevated, or removed, it would still pose problems with implementability similar to those for Alternative No. 1 because of the need to floodproof, elevate, or remove 19 buildings. Also, Alternative No. 4 would not significantly alleviate the potential for street flooding and secondary flooding of basements along Manor Hills Boulevard.

As noted previously in this chapter, under Alternative No. 1, the problem of dry-land access to the Bowling Green Industrial Park during floods could be eliminated by extending Bobolink Avenue 1,000 feet west to Pilgrim Road. With that addition to Alternative No. 1, one major advantage of Alternative No. 4 over Alternative No. 1 is eliminated. Because of the much higher cost of Alternative No. 4 in comparison to Alternative No. 1 and because of the limited advantages of Alternative No. 4, it may be concluded that Alternative No. 4 is inferior to Alternative No. 1. Therefore, the addition of Alternative No. 4 does not change the evaluation of flood control alternatives presented above or the selection of flood control Alternative No. 3: Channel Modification and Bridge Removal or Replacement as the preliminary recommended plan.

Chapter VI

RECOMMENDED STORMWATER MANAGEMENT AND FLOOD CONTROL PLAN

The recommended stormwater management and flood control system plan consists of three elements: a stormwater drainage plan element, a water quality management plan element, and a flood control plan element. The preliminary recommended plan elements presented in Chapter V of this report were refined as described in this chapter to accommodate the recommendations of the report on environmental enhancement measures for Lilly Creek prepared by BRW, Inc., presented in Appendix C of this report, and to reflect changes made to avoid disturbance of wetlands, as described in Appendix D of this report.

REFINEMENT OF THE PRELIMINARY STORMWATER MANAGEMENT PLAN

The preliminary recommended stormwater management plan was designed to achieve the stormwater management objectives set forth in Chapter IV of this report. The preliminary plan was reviewed by representatives of the Village of Menomonee Falls, by an advisory committee created by the Village and composed of officials and concerned citizens of the Village, and by the Wisconsin Department of Natural Resources (DNR). On the basis of the comments and suggestions made by those who reviewed the plan, the preliminary plan was refined and a final recommended plan prepared.

Refinements were made to the preliminary recommended stormwater drainage plan in order to integrate the drainage and the flood control plans by accounting for the lowered Lilly Creek streambed and lowered Lilly Creek flood stages at storm sewer outfalls and to address comments and requests made by staff members of the Village and of the DNR.

The preliminary recommended water quality management plan provided a high level of pollution control, with anticipated pollutant loadings 55 to 72 percent less than the loadings expected under planned ultimate land use conditions. While the desired water use objectives, shown on Map 7 in Chapter IV of this report, would be met over all within the subwatershed, some pollutants, primarily metals, could exceed allowable limits on some stream reaches during certain storm events. In A Nonpoint Source Control Plan for the Menomonee River Priority Watershed Project, 1990, the Wisconsin DNR recommended pollution reduction goals to achieve the water use objectives established by the Department which are identical to the objectives set forth in Chapter IV. These goals included an approximate 50 percent reduction in the existing sediment, phosphorus, and metal loadings to Lilly Creek. While the preliminary recommended plan would achieve the goal for sediment, resulting in a 70 percent reduction in existing loadings, the plan would achieve only about a 6 percent reduction in existing phosphorus loadings, while metal loadings would increase by about 13 percent. Further investigations were, therefore, conducted to determine whether additional pollution control measures could be incorporated into the recommended plan. As a result, three additional wet detention basin sites were identified and the basins were included in the recommended plan. The additional proposed detention basins are WD25, a 0.6-acre basin which would discharge directly to Lilly Creek within Hydrologic Unit B; WD26, a 0.3-acre basin which would discharge to Woodshaven Tributary within Hydrologic Unit J; and WD27, a 0.3-acre basin which would discharge directly to Lilly Creek within Hydrologic Unit K. Characteristics of the recommended wet basins are presented in Table 50. With the addition of these three basins, 50 percent of the existing urban area would be treated by wet detention and the portion of the total subwatershed land area treated by wet detention would be increased from 64 percent to 70 percent. The reductions in existing nonpoint source pollutant loadings under ultimate planned land use conditions would be increased from 70 percent to 73 percent for suspended solids and from 6 percent to 9 percent for phosphorus. The additional three basins would also reduce the increase in planned metal loadings relative to the existing loading, from 13 percent to 8 percent.

The addition to the plan of even more wet detention basins, as well as additional increased street sweeping, grassed swales, and stormwater infiltration facilities, was also considered. However, it was found that none of these measures would provide significant additional pollution abatement because these additional control measures would have only limited effectiveness in the remaining untreated areas. The effectiveness of the additional measures would be limited because the remaining untreated areas are expected to generate relatively low pollutant loadings to Lilly Creek and its tributaries. Thus, the final recommended water quality management plan element, the preliminary plan presented in Chapter V plus three additional wet detention basins, is expected to achieve the maximum level of nonpoint source pollution control practicable. The anticipated water quality benefits of the final recommended water quality management plan element are presented in a subsequent section of this chapter.

RECOMMENDED STORMWATER MANAGEMENT PLAN

A brief summary of the recommended plan components for each of the hydrologic units in the subwatershed is provided below. The recommended components are shown on Map 20. The components and their associated costs are given in Table 49. Hydraulic and hydrologic characteristics of the recommended detention basins are given in Table 50. Those basins for which two-year recurrence interval storm data are provided in Table 50 were designed to regulate frequently occurring storms, thereby controlling streambank erosion and streambed scour.

A general recommendation of the stormwater management plan which is common to all hydrologic units is enforcement of the construction erosion control ordinance adopted by the Village of Menomonee Falls on April 15, 1991. Another such general recommendation is that the street systems required to support future urban development be carefully configured, horizontally and vertically, to provide the necessary major drainage system conveyance capacity.

Hydrologic Unit A, Silver Spring Tributary

Approximately 24 percent of the land in this hydrologic unit was in urban uses as of 1985. It is anticipated that about 86 percent of the hydrologic unit would be in urban uses under ultimate planned land use conditions.

The preliminary recommendation for construction of approximately 0.65 mile of modified channel from the Village Fire Station entrance to a point about 0.25 mile west of Pilgrim Road was scaled back in the final plan. It was found that planned runoff could be adequately conveyed with a total of 0.31 mile of modified channel, including the 0.23-mile-long reach from the Fire Station entrance to a location 270 feet east of Butternut Drive and an 0.08-mile-long reach beginning 680 feet west of Bette Drive. Culvert replacements would be required only at the Fire Station entrance and at Badger Drive.

Storm sewer conveyance would be provided in the western portion of the hydrologic unit, which is now essentially undeveloped, but would be developed predominantly for industrial use under ultimate planned land use conditions.

Dual-purpose detention basin WD1 would receive runoff from a 174-acre area, or about 40 percent of the hydrologic unit area. The total area in each land use category which is tributary to the basin under both existing and planned ultimate land use conditions is given in Table 51. In addition to basin WD1, nonpoint source control would be provided through the maintenance of the existing roadside swale and open channel system in the residential portions of the hydrologic unit and through weekly sweeping of all nondetained commercial streets in spring, summer, and fall.

Also, existing natural detention storage area ND1 would be preserved to provide a reduction in peak flood discharges in Lilly Creek.

Hydrologic Unit B, Phillips Tributary

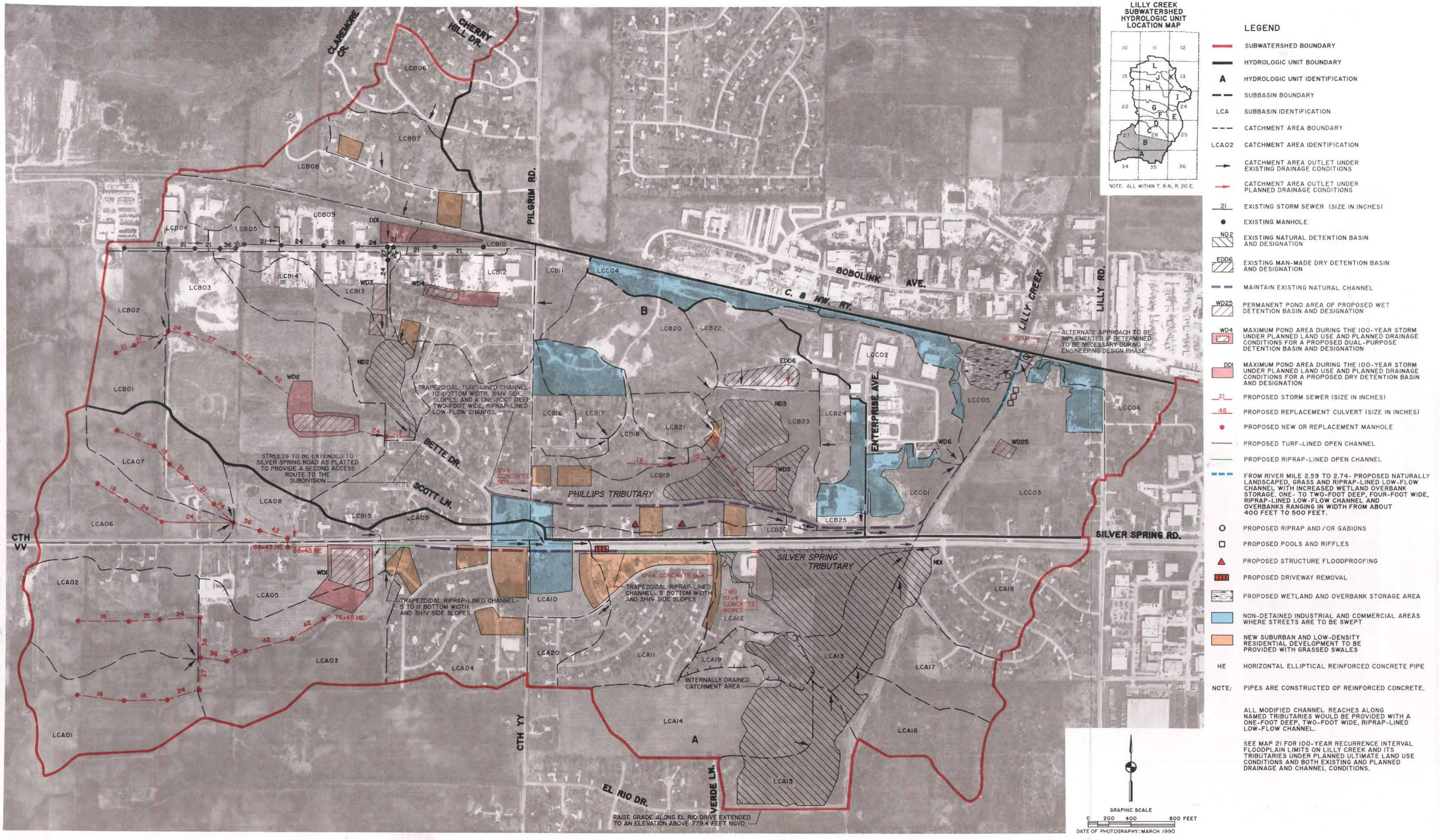
Approximately 41 percent of the land in this hydrologic unit was in urban uses as of 1985. It is anticipated that about 90 percent of the hydrologic unit would be in urban uses under ultimate planned land use conditions.

The staff of the Village requested that, because of ingress and egress problems for some homes along Bette Drive during floods, the preliminary recommended plan be revised to include storm sewers through the existing subdivision along Bette Drive. The preliminary plan recommended that the open channel system along Bette Drive be preserved because the channel has adequate hydraulic capacity to convey the runoff resulting from a 100-year recurrence interval storm under planned ultimate land use and drainage condi-

Map 20

RECOMMENDED SYSTEM PLAN FOR STORMWATER MANAGEMENT AND FLOOD CONTROL IN THE LILLY CREEK SUBWATERSHED

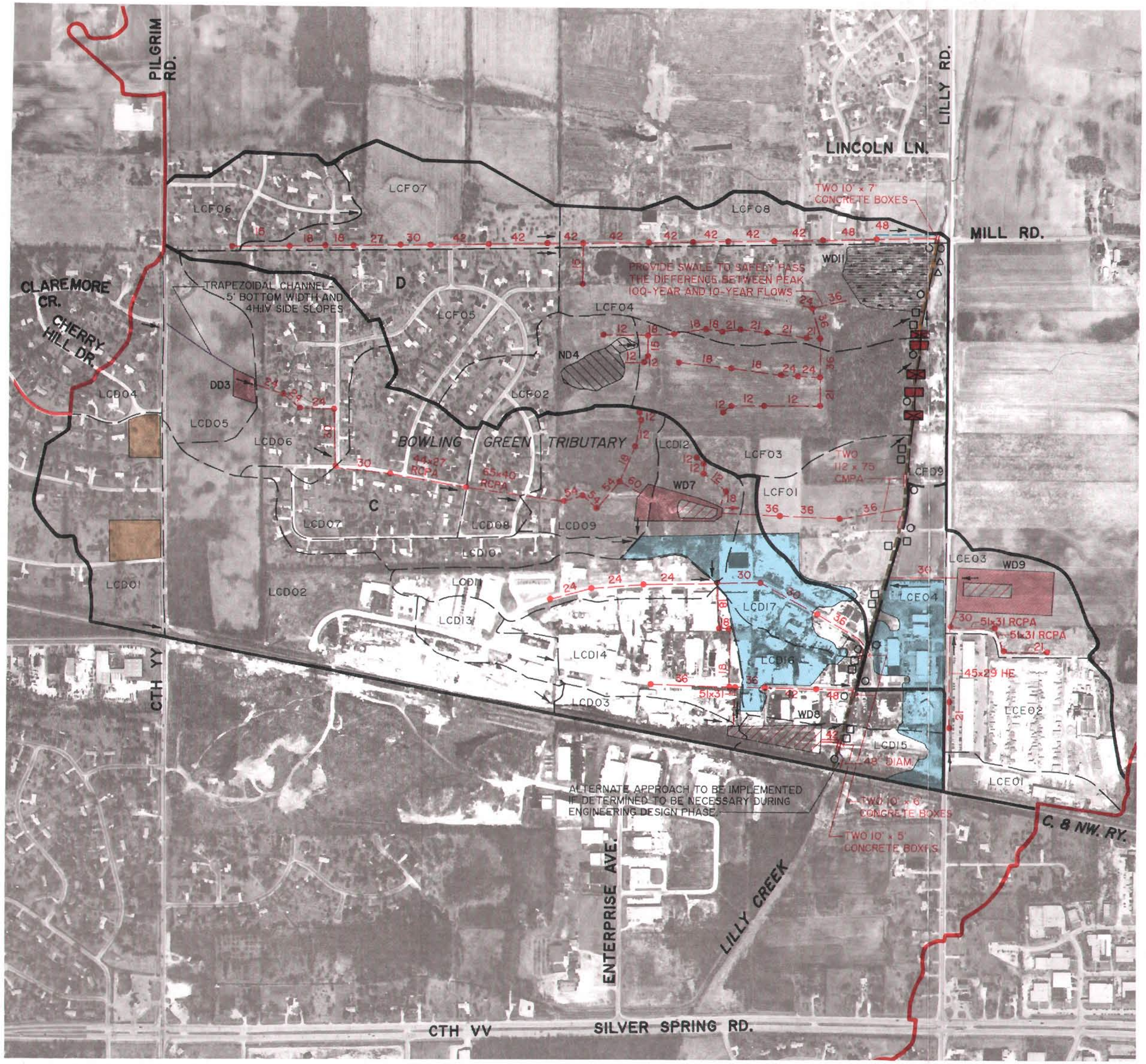
HYDROLOGIC UNITS A AND B



Map 20 (continued)

RECOMMENDED SYSTEM PLAN FOR STORMWATER MANAGEMENT AND FLOOD CONTROL IN THE LILLY CREEK SUBWATERSHED

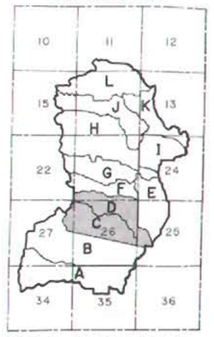
HYDROLOGIC UNITS C AND D



LEGEND

—	SUBWATERSHED BOUNDARY
—	HYDROLOGIC UNIT BOUNDARY
C	HYDROLOGIC UNIT IDENTIFICATION
— —	SUBBASIN BOUNDARY
LCD	SUBBASIN IDENTIFICATION
– – –	CATCHMENT AREA BOUNDARY
LCD05	CATCHMENT AREA IDENTIFICATION
→	CATCHMENT AREA OUTLET UNDER EXISTING DRAINAGE CONDITIONS
→	CATCHMENT AREA OUTLET UNDER PLANNED DRAINAGE CONDITIONS
ND4 ▨	EXISTING NATURAL DETENTION BASIN AND DESIGNATION
WD8 ▨	PERMANENT POND AREA OF PROPOSED WET DETENTION BASIN AND DESIGNATION
WD7 ▨	MAXIMUM POND AREA DURING THE 100-YEAR STORM UNDER PLANNED LAND USE AND PLANNED DRAINAGE CONDITIONS FOR A PROPOSED DUAL-PURPOSE DETENTION BASIN AND DESIGNATION
DD3 ■	MAXIMUM POND AREA DURING THE 100-YEAR STORM UNDER PLANNED LAND USE AND PLANNED DRAINAGE CONDITIONS FOR A PROPOSED DRY DETENTION BASIN AND DESIGNATION
42	PROPOSED STORM SEWER (SIZE IN INCHES)
10	PROPOSED REPLACEMENT CULVERT (SIZE IN INCHES)
●	PROPOSED NEW OR REPLACEMENT MANHOLE
—	PROPOSED TURF-LINED OPEN CHANNEL
– – –	FROM RIVER MILE 1.88 TO 1.99- NATURALLY LANDSCAPED, GRASSED, FLOOD CONTROL AND STREAM RESTORATION CHANNEL WITH INCREASED WETLAND OVERBANK STORAGE. ONE-FOOT DEEP, FOUR-FOOT WIDE, RIPRAP-LINED LOW-FLOW CHANNEL, SURMOUNTED BY FLOOD CONTROL CHANNEL WITH AVERAGE SIDE SLOPES RANGING FROM ONE VERTICAL ON TWO HORIZONTAL TO ONE VERTICAL ON THREE HORIZONTAL AND OVERBANKS RANGING IN WIDTH FROM ABOUT 500 FEET TO 600 FEET.
– – –	FROM RIVER MILE 1.99 TO 2.59- NATURALLY LANDSCAPED, GRASSED, FLOOD CONTROL AND STREAM RESTORATION CHANNEL. ONE-FOOT DEEP, FOUR-FOOT WIDE, RIPRAP-LINED LOW-FLOW CHANNEL. FLOOD CONTROL CHANNEL BOTTOM WIDTH OF 12 FEET WITH AVERAGE SIDE SLOPES OF ONE VERTICAL ON THREE HORIZONTAL. APPROPRIATE BOTTOM WIDTH TRANSITIONS AT CULVERTS.
○	PROPOSED RIPRAP AND/OR GABIONS
□	PROPOSED POOLS AND RIFFLES
△	PROPOSED MEANDER GUIDES
☒	PROPOSED BRIDGE REMOVAL
■	PROPOSED NEW PEDESTRIAN BRIDGE
- — -	PROPOSED RECREATIONAL TRAIL
▤	PROPOSED WETLAND AND OVERBANK STORAGE AREA
🟦	NON-DETAINED INDUSTRIAL AND COMMERCIAL AREAS WHERE STREETS ARE TO BE SWEPT
🟧	NEW SUBURBAN AND LOW-DENSITY RESIDENTIAL DEVELOPMENT TO BE PROVIDED WITH GRASSED SWALES
HE	HORIZONTAL ELLIPTICAL REINFORCED CONCRETE PIPE
RCPA	REINFORCED CONCRETE PIPE ARCH
CMPA	CORRUGATED METAL PIPE ARCH
NOTE:	PIPES ARE CONSTRUCTED OF REINFORCED CONCRETE UNLESS DESIGNATED AS ABOVE.
	SEE MAP 21 FOR 100-YEAR RECURRENCE INTERVAL FLOODPLAIN LIMITS ON LILLY CREEK AND ITS TRIBUTARIES UNDER PLANNED ULTIMATE LAND USE CONDITIONS AND BOTH EXISTING AND PLANNED DRAINAGE AND CHANNEL CONDITIONS.

LILLY CREEK SUBWATERSHED HYDROLOGIC UNIT LOCATION MAP



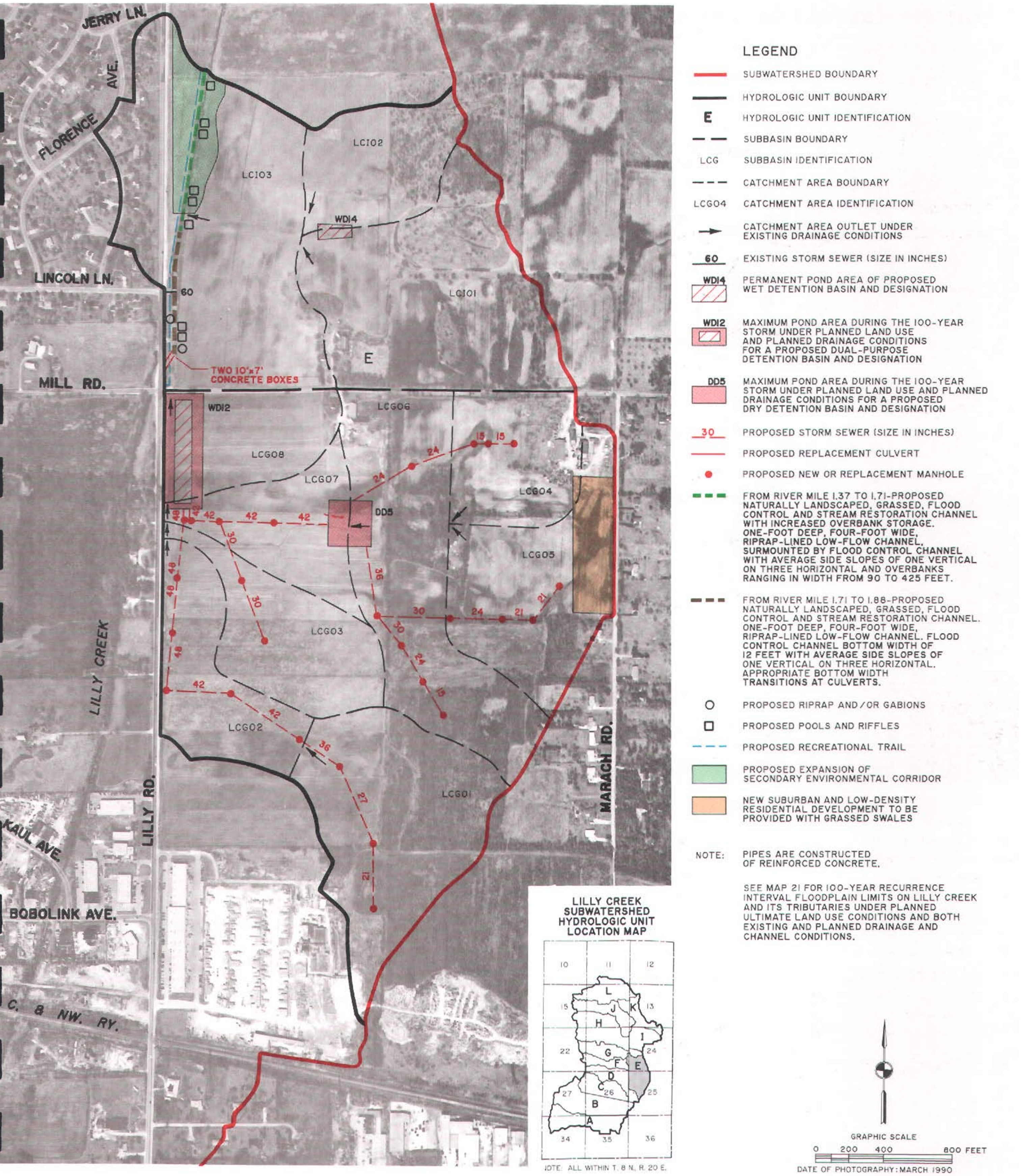
NOTE: ALL WITHIN T. 8 N., R. 20 E.

GRAPHIC SCALE
0 200 400 800 FEET
DATE OF PHOTOGRAPHY: MARCH 1990

Map 20 (continued)

RECOMMENDED SYSTEM PLAN FOR STORMWATER MANAGEMENT AND FLOOD CONTROL IN THE LILLY CREEK SUBWATERSHED

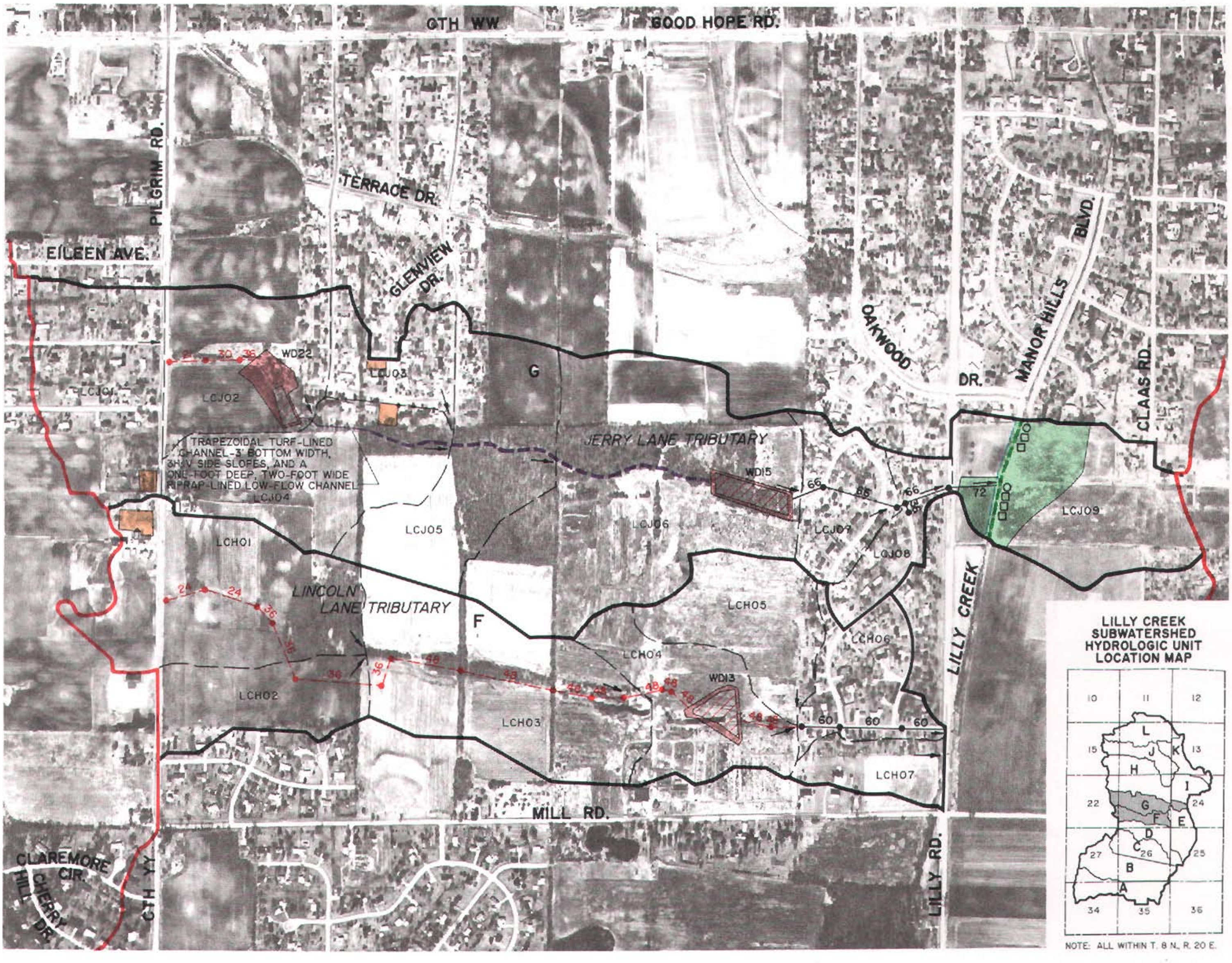
HYDROLOGIC UNIT E



Map 20 (continued)

RECOMMENDED SYSTEM PLAN FOR STORMWATER MANAGEMENT AND FLOOD CONTROL IN THE LILLY CREEK SUBWATERSHED

HYDROLOGIC UNITS F AND G



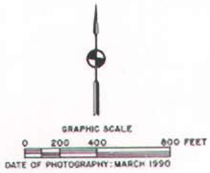
LEGEND

—	SUBWATERSHED BOUNDARY
—	HYDROLOGIC UNIT BOUNDARY
F	HYDROLOGIC UNIT IDENTIFICATION
— —	SUBBASIN BOUNDARY
LCJ	SUBBASIN IDENTIFICATION
- - -	CATCHMENT AREA BOUNDARY
LCJ03	CATCHMENT AREA IDENTIFICATION
→	CATCHMENT AREA OUTLET UNDER EXISTING DRAINAGE CONDITIONS
60	EXISTING STORM SEWER (SIZE IN INCHES)
●	EXISTING MANHOLE
— —	MAINTAIN EXISTING NATURAL CHANNEL
WD22 ▨	PERMANENT POND AREA OF PROPOSED WET DETENTION BASIN AND DESIGNATION
WD22 ▭	MAXIMUM POND AREA DURING THE 100-YEAR STORM UNDER PLANNED LAND USE AND PLANNED DRAINAGE CONDITIONS FOR A PROPOSED DUAL-PURPOSE DETENTION BASIN AND DESIGNATION
48	PROPOSED STORM SEWER (SIZE IN INCHES)
●	PROPOSED NEW OR REPLACEMENT MANHOLE
—	PROPOSED TURF-LINED OPEN CHANNEL
- - -	FROM RIVER MILE 1.37 TO 1.71-PROPOSED NATURALLY LANDSCAPED, GRASSED, FLOOD CONTROL AND STREAM RESTORATION CHANNEL WITH INCREASED OVERBANK STORAGE. ONE-FOOT DEEP, FOUR-FOOT WIDE, RIPRAP-LINED LOW-FLOW CHANNEL, SURMOUNTED BY FLOOD CONTROL CHANNEL WITH AVERAGE SIDE SLOPES OF ONE VERTICAL ON THREE HORIZONTAL AND OVERBANKS RANGING IN WIDTH FROM 90 TO 425 FEET.
○	PROPOSED RIPRAP AND/OR GABIONS
□	PROPOSED POOLS AND RIFFLES
— —	PROPOSED RECREATIONAL TRAIL
🟩	PROPOSED EXPANSION OF SECONDARY ENVIRONMENTAL CORRIDOR
🟧	NEW SUBURBAN AND LOW-DENSITY RESIDENTIAL DEVELOPMENT TO BE PROVIDED WITH GRASSED SWALES

NOTE: PIPES ARE CONSTRUCTED OF REINFORCED CONCRETE.

ALL MODIFIED CHANNEL REACHES ALONG NAMED TRIBUTARIES WOULD BE PROVIDED WITH A ONE-FOOT DEEP, TWO-FOOT WIDE, RIPRAP-LINED LOW-FLOW CHANNEL.

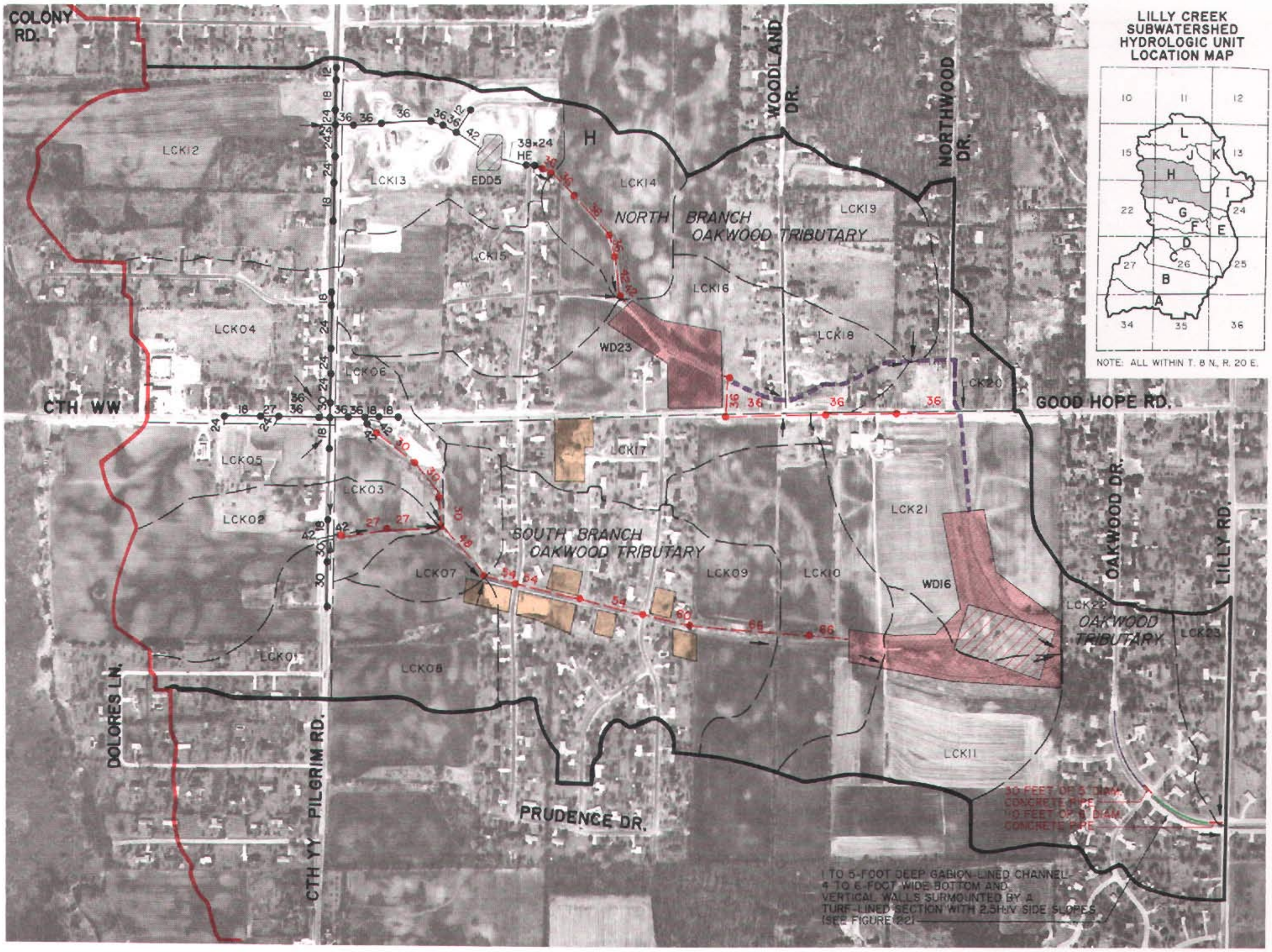
SEE MAP 21 FOR 100-YEAR RECURRENCE INTERVAL FLOODPLAIN LIMITS ON LILLY CREEK AND ITS TRIBUTARIES UNDER PLANNED ULTIMATE LAND USE CONDITIONS AND BOTH EXISTING AND PLANNED DRAINAGE AND CHANNEL CONDITIONS.



Map 20 (continued)

RECOMMENDED SYSTEM PLAN FOR STORMWATER MANAGEMENT AND FLOOD CONTROL IN THE LILLY CREEK SUBWATERSHED

HYDROLOGIC UNIT H



LEGEND

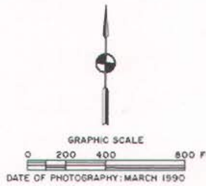
—	SUBWATERSHED BOUNDARY
—	HYDROLOGIC UNIT BOUNDARY
H	HYDROLOGIC UNIT IDENTIFICATION
— —	SUBBASIN BOUNDARY
LCK	SUBBASIN IDENTIFICATION
– – –	CATCHMENT AREA BOUNDARY
LCK12	CATCHMENT AREA IDENTIFICATION
→	CATCHMENT AREA OUTLET UNDER EXISTING DRAINAGE CONDITIONS
36	EXISTING STORM SEWER (SIZE IN INCHES)
●	EXISTING MANHOLE
EDD5 ▨	EXISTING MAN-MADE DRY DETENTION BASIN AND DESIGNATION
— —	MAINTAIN EXISTING NATURAL CHANNEL
☐	PERMANENT POND AREA OF PROPOSED WET DETENTION BASIN
WD16 ☐	MAXIMUM POND AREA DURING THE 100-YEAR STORM UNDER PLANNED LAND USE AND PLANNED DRAINAGE CONDITIONS FOR A PROPOSED DUAL-PURPOSE DETENTION BASIN AND DESIGNATION

WD23 🟥	MAXIMUM POND AREA DURING THE 100-YEAR STORM UNDER PLANNED LAND USE AND PLANNED DRAINAGE CONDITIONS FOR A PROPOSED DRY DETENTION BASIN AND DESIGNATION
36	PROPOSED STORM SEWER (SIZE IN INCHES)
—	PROPOSED REPLACEMENT CULVERT
●	PROPOSED NEW OR REPLACEMENT MANHOLE
—	PROPOSED TURF-LINED OPEN CHANNEL
—	PROPOSED GABION-LINED OPEN CHANNEL
🟧	NEW SUBURBAN AND LOW-DENSITY RESIDENTIAL DEVELOPMENT TO BE PROVIDED WITH GRASSED SWALES ALONE OR GRASSED SWALES OVER STORM SEWERS
HE	HORIZONTAL ELLIPTICAL REINFORCED CONCRETE PIPE

NOTE: PIPES ARE CONSTRUCTED OF REINFORCED CONCRETE.

ALL MODIFIED CHANNEL REACHES ALONG NAMED TRIBUTARIES WOULD BE PROVIDED WITH A ONE-FOOT DEEP, TWO-FOOT WIDE, RIPRAP-LINED LOW-FLOW CHANNEL.

SEE MAP 21 FOR 100-YEAR RECURRENCE INTERVAL FLOODPLAIN LIMITS ON LILLY CREEK AND ITS TRIBUTARIES UNDER PLANNED ULTIMATE LAND USE CONDITIONS AND BOTH EXISTING AND PLANNED DRAINAGE AND CHANNEL CONDITIONS.

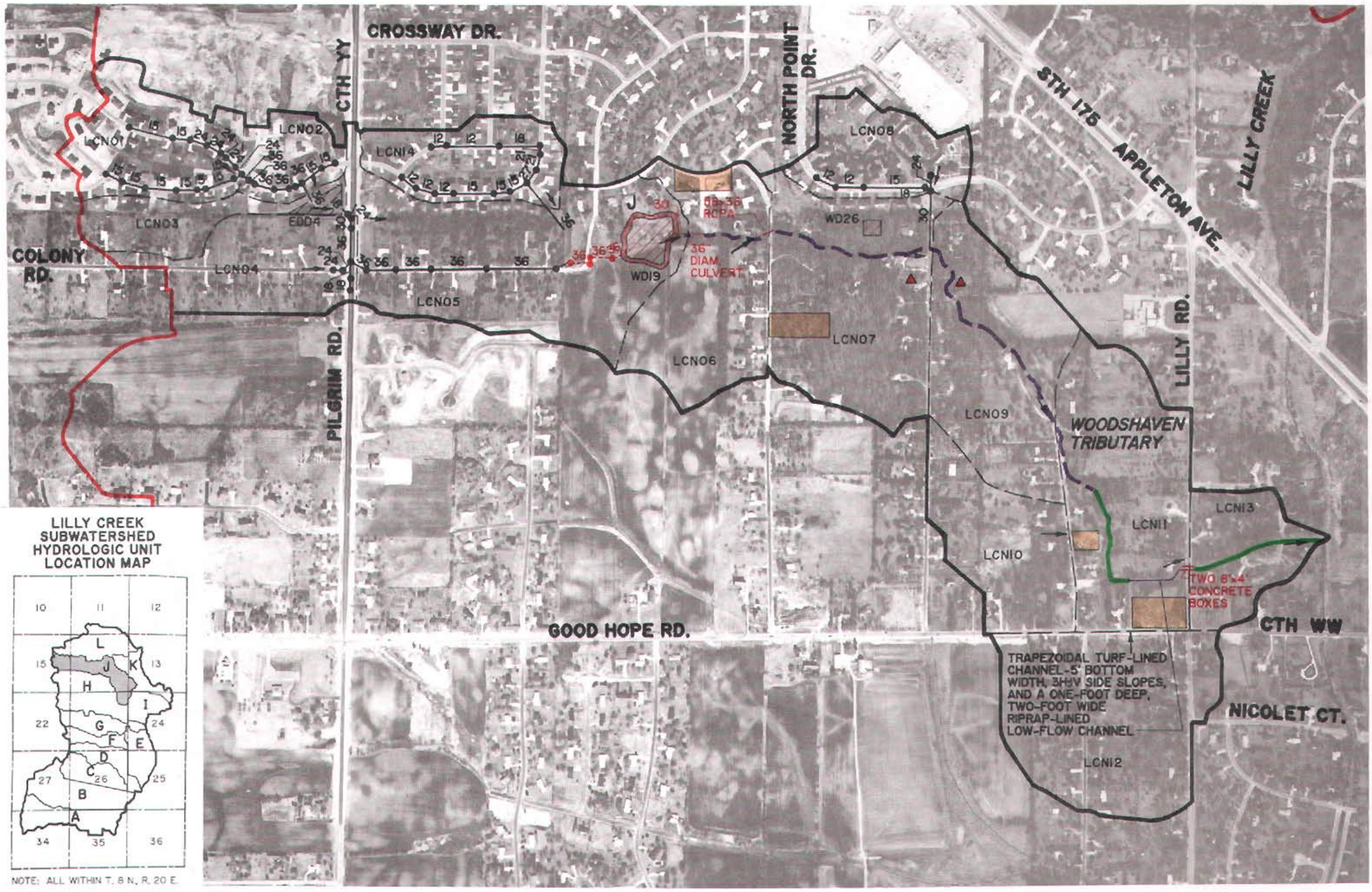


DATE OF PHOTOGRAPHY: MARCH 1990

Map 20 (continued)

RECOMMENDED SYSTEM PLAN FOR STORMWATER MANAGEMENT AND FLOOD CONTROL IN THE LILLY CREEK SUBWATERSHED

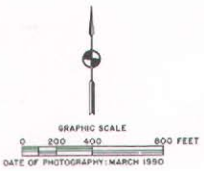
HYDROLOGIC UNIT J



LEGEND

━	SUBWATERSHED BOUNDARY
━	HYDROLOGIC UNIT BOUNDARY
J	HYDROLOGIC UNIT IDENTIFICATION
— —	SUBBASIN BOUNDARY
LCN	SUBBASIN IDENTIFICATION
– – –	CATCHMENT AREA BOUNDARY
LCN14	CATCHMENT AREA IDENTIFICATION
→	CATCHMENT AREA OUTLET UNDER EXISTING DRAINAGE CONDITIONS
12	EXISTING STORM SEWER (SIZE IN INCHES)
●	EXISTING MANHOLE
EDD4 ▨	EXISTING MAN-MADE DRY DETENTION BASIN AND DESIGNATION
— —	MAINTAIN EXISTING NATURAL CHANNEL
━	MAINTAIN EXISTING NATURAL CHANNEL AND PROVIDE RIPRAP ALONG STREAMBANKS AND STREAMBED
WD26 ▨	PERMANENT POND AREA OF PROPOSED WET DETENTION BASIN AND DESIGNATION
WD19 ▨	MAXIMUM POND AREA DURING THE 100-YEAR STORM UNDER PLANNED LAND USE AND PLANNED DRAINAGE CONDITIONS FOR A PROPOSED DUAL-PURPOSE DETENTION BASIN AND DESIGNATION
30	PROPOSED STORM SEWER (SIZE IN INCHES)
36	PROPOSED CULVERT (SIZE IN INCHES)
●	PROPOSED NEW OR REPLACEMENT MANHOLE

―	PROPOSED TURF-LINED OPEN CHANNEL
▲	PROPOSED STRUCTURE FLOODPROOFING
■	NEW SUBURBAN AND LOW-DENSITY RESIDENTIAL DEVELOPMENT TO BE PROVIDED WITH GRASSED SWALES
RCPA	REINFORCED CONCRETE PIPE ARCH
NOTE:	PIPES ARE CONSTRUCTED OF REINFORCED CONCRETE. SEE MAP 21 FOR 100-YEAR RECURRENCE INTERVAL FLOODPLAIN LIMITS ON LILLY CREEK AND ITS TRIBUTARIES UNDER PLANNED ULTIMATE LAND USE CONDITIONS AND BOTH EXISTING AND PLANNED DRAINAGE AND CHANNEL CONDITIONS.



Map 20 (continued)

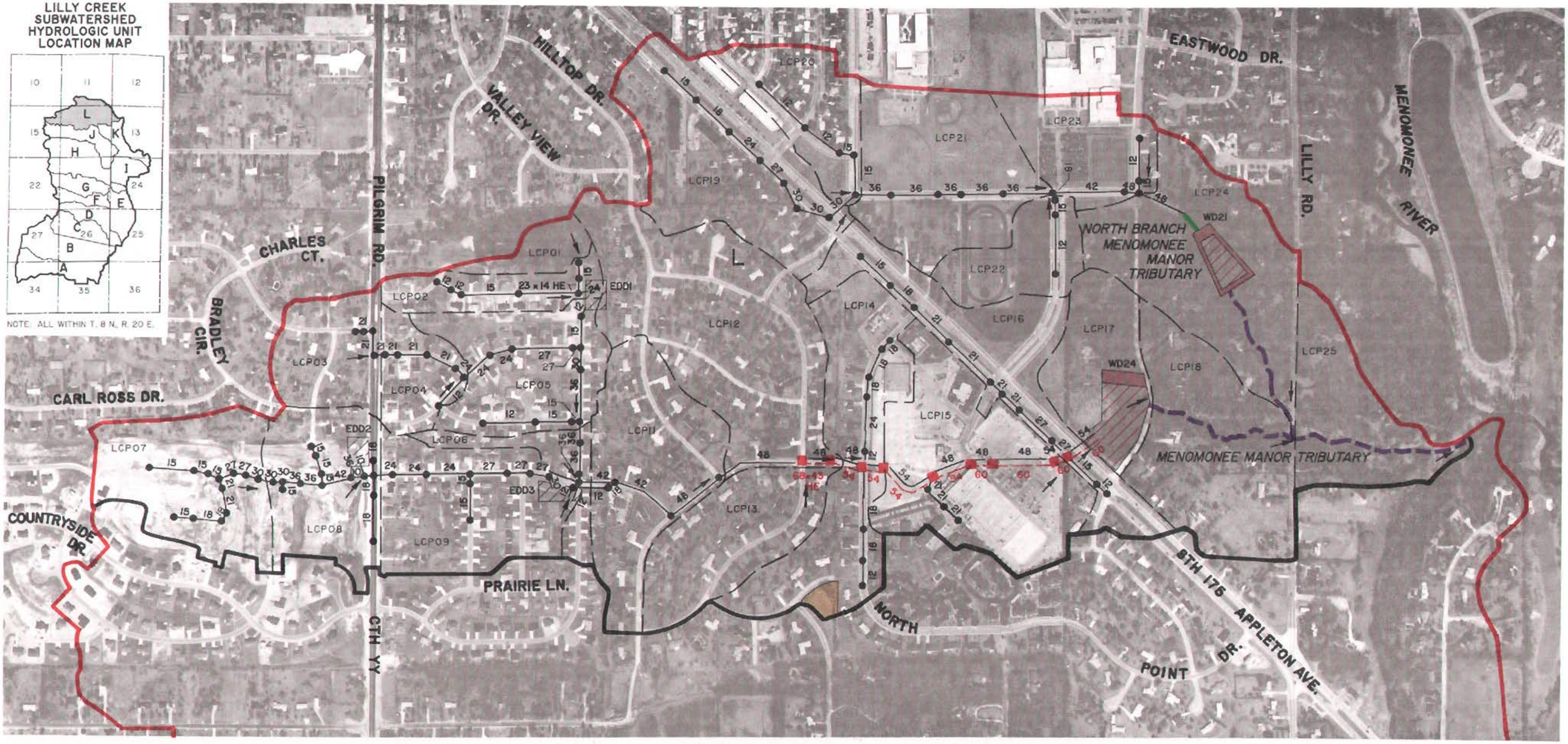
RECOMMENDED SYSTEM PLAN FOR STORMWATER MANAGEMENT AND FLOOD CONTROL IN THE LILLY CREEK SUBWATERSHED

HYDROLOGIC UNITS I AND K

Map 20 (continued)

RECOMMENDED SYSTEM PLAN FOR STORMWATER MANAGEMENT AND FLOOD CONTROL IN THE LILLY CREEK SUBWATERSHED

HYDROLOGIC UNIT L



LEGEND

- SUBWATERSHED BOUNDARY
- HYDROLOGIC UNIT BOUNDARY
- L HYDROLOGIC UNIT IDENTIFICATION
- SUBBASIN BOUNDARY
- LCP SUBBASIN IDENTIFICATION
- CATCHMENT AREA BOUNDARY
- LCP22 CATCHMENT AREA IDENTIFICATION
- CATCHMENT AREA OUTLET UNDER EXISTING DRAINAGE CONDITIONS
- 21 EXISTING STORM SEWER (SIZE IN INCHES)
- ● EXISTING MANHOLE
- EDD3 ▨ EXISTING MAN-MADE DRY DETENTION BASIN AND DESIGNATION
- MAINTAIN EXISTING NATURAL CHANNEL
- MAINTAIN EXISTING NATURAL CHANNEL AND PROVIDE RIPRAP ALONG STREAMBANKS AND STREAMBED
- ▨ PERMANENT POND AREA OF PROPOSED WET DETENTION BASIN AND DESIGNATION
- WD21 ▨ MAXIMUM POND AREA DURING THE 100-YEAR STORM UNDER PLANNED LAND USE AND PLANNED DRAINAGE CONDITIONS FOR A PROPOSED DUAL-PURPOSE DETENTION BASIN AND DESIGNATION
- 60 PROPOSED STORM SEWER (SIZE IN INCHES)
- ■ PROPOSED JUNCTION BOX
- NEW SUBURBAN AND LOW-DENSITY RESIDENTIAL DEVELOPMENT TO BE PROVIDED WITH GRASSED SWALES
- HE HORIZONTAL ELLIPTICAL REINFORCED CONCRETE PIPE

NOTE: PIPES ARE CONSTRUCTED OF REINFORCED CONCRETE.

SEE MAP 21 FOR 100-YEAR RECURRENCE INTERVAL FLOODPLAIN LIMITS ON LILLY CREEK AND ITS TRIBUTARIES UNDER PLANNED ULTIMATE LAND USE CONDITIONS AND BOTH EXISTING AND PLANNED DRAINAGE AND CHANNEL CONDITIONS.

GRAPHIC SCALE
0 200 400 800 FEET
DATE OF PHOTOGRAPHY: MARCH 1990

Source: SEWRPC.

Table 49

COMPOSITION AND COSTS OF THE RECOMMENDED STORMWATER MANAGEMENT PLAN FOR LILLY CREEK

Hydrologic Unit	Project and Component Description[a]	Estimated Cost	
		Capital[b]	Annual Operation and Maintenance[c]
A	Silver Spring Tributary		
	1. 380 feet of 12-inch storm sewer	$ 16,000	$ 200
	2. 200 feet of 15-inch storm sewer	9,000	100
	3. 1,590 feet of 18-inch storm sewer	82,000	600
	4. 825 feet of 21-inch storm sewer	48,000	300
	5. 1,980 feet of 24-inch storm sewer	123,000	800
	6. 300 feet of 27-inch storm sewer	21,000	100
	7. 1,070 feet of 36-inch storm sewer	100,000	200
	8. 1,100 feet of 42-inch storm sewer	123,000	200
	9. 430 feet of 68-inch x 43-inch concrete horizontal elliptical (H.E.) storm sewer	82,000	100
	10. 150 feet of 76-inch x 48-inch concrete H.E. storm sewer	35,000	0
	11. Modify 0.31 mile of channel	52,000	1,000
	12. Replace existing culverts at Fire Station with two 50-foot-long, 10-foot-wide x 4-foot-high concrete box culverts	65,000	0
	13. Replace existing culvert at Badger Drive with a 35-foot-long, 10-foot-wide x 4-foot-high concrete box culvert	23,000	0
	14. Remove private drive	1,000	0
	15. Detention Basin WD1 Incremental 100-year flood control storage volume of 15.5 acre-feet[d]	172,000	3,000
	Water Quantity Subtotal	$ 952,000	$ 6,600
	16. Construction erosion control, 13.2 acres per year	$ 396,000	$ 1,000
	17. Sweep 0.58 curb-mile of street	200	400
	18. Detention Basin WD1 Two-year flood control storage volume of 26.2 acre-feet[e]	373,000	10,500
	Water Quality Subtotal	$ 769,200	$11,900
	Total	$1,721,200	$18,500
B	Phillips Tributary		
	1. 1,020 feet of 12-inch storm sewer	$ 34,000	$ 300
	2. 500 feet of 15-inch storm sewer	23,000	200
	3. 510 feet of 21-inch storm sewer	35,000	200
	4. 540 feet of 24-inch storm sewer	40,000	200
	5. 660 feet of 27-inch storm sewer	52,000	300
	6. 600 feet of 42-inch storm sewer	75,000	100
	7. Construct 475-foot-long trapezoidal, turf-lined channel with a 10-foot bottom width, IV:3H side slopes, and a 1-foot-deep, 2-foot-wide riprap-lined low-flow channel	29,000	200
	8. Floodproof two houses	10,000	0
	9. Replace existing culverts at Pilgrim Road with three 40-foot-long, 8-foot-wide x 4-foot-high concrete box culverts	66,000	0
	10. Detention Basin WD2 Incremental 100-year flood control storage volume of 12.0 acre-feet[d]	150,000	3,100
	11. Detention Basin WD4 Incremental 100-year flood control storage volume of 2.6 acre-feet[d]	83,000	900
	12. Detention Basin DD1 Incremental 100-year flood control storage volume of 11.3 acre-feet[d]	176,000	4,200
	Water Quantity Subtotal	$ 773,000	$ 9,700

Table 49 (continued)

Hydrologic Unit	Project and Component Description[a]	Estimated Cost	
		Capital[b]	Annual Operation and Maintenance[c]
B (continued)	13. Construction erosion control, 14.2 acres per year	$ 426,000	$ 1,100
	14. Sweep 2.22 curb-mile of street	700	1,600
	15. Detention Basin WD2 Two-year flood control storage volume of 14.6 acre-feet[e]	247,000	6,400
	16. Detention Basin WD4 Two-year flood control storage volume of 4.4 acre-feet[e]	138,000	2,700
	17. Detention Basin DD1 Two-year flood control storage volume of 4.6 acre-feet[f]	84,000	2,700
	18. Detention Basin WD3 Permanent wet basin volume of 6.8 acre-feet	126,000	3,600
	19. Detention Basin WD5 Permanent wet basin volume of 5.5 acre-feet	160,000	3,100
	20. Detention Basin WD25 Permanent wet basin volume of 3.0 acre-feet	71,000	2,300
	21. Detention Basin WD6 Permanent wet basin volume of 3.0 acre-feet	71,000	2,100
	Water Quality Subtotal	$1,323,700	$25,600
	Total	$2,096,700	$35,300
C	Bowling Green Tributary		
	1. 590 feet of 12-inch storm sewer	$ 25,000	$ 200
	2. 390 feet of 18-inch storm sewer	20,000	200
	3. 1,770 feet of 24-inch storm sewer	129,000	700
	4. 1,510 feet of 30-inch storm sewer	133,000	600
	5. 1,180 feet of 36-inch storm sewer	124,000	200
	6. 350 feet of 42-inch storm sewer	43,000	100
	7. 230 feet of 48-inch storm sewer	34,000	0
	8. 570 feet of 54-inch storm sewer	89,000	0
	9. 200 feet of 60-inch storm sewer	37,000	0
	10. 520 feet of 44-inch by 27-inch reinforced concrete pipe arch (RCPA) storm sewer	62,000	100
	11. 650 feet of 65-inch by 40-inch RCPA storm sewer	134,000	100
	12. 1,200 feet of 36-inch diversion pipe	150,000	200
	13. Construct 550-foot-long trapezoidal, turf-lined channel with a 5-foot bottom width, and IV:4H side slopes	6,000	200
	14. Detention Basin WD7 Incremental 100-year flood control storage volume of 8.3 acre-feet[d]	135,000	1,000
	15. Detention Basin DD3 Incremental 100-year flood control storage volume of 1.2 acre-feet[d]	14,000	300
	Water Quantity Subtotal	$1,135,000	$ 3,900
	16. Construction erosion control, 4.6 acres per year	$ 138,000	$ 300
	17. Sweep 0.46 curb-mile of street	100	300
	18. Detention Basin WD7 Two-year flood control storage volume of 6.7 acre-feet[e]	135,000	3,500
	19. Detention Basin WD8 Permanent pond volume of 8.1 acre-feet[g]	589,000	4,400
	20. Detention Basin DD3 Two-year flood control storage volume of 0.5 acre-feet[f]	26,000	1,300
	Water Quality Subtotal	$ 888,100	$ 9,800
	Total	$2,023,100	$13,700

Table 49 (continued)

Hydrologic Unit	Project and Component Description[a]	Estimated Cost	
		Capital[b]	Annual Operation and Maintenance[c]
D	Area Predominantly West of Lilly Creek and North and South of W. Mill Road		
	1. 1,180 feet of 12-inch storm sewer	$ 50,000	$ 500
	2. 850 feet of 15-inch storm sewer	43,000	300
	3. 1,660 feet of 18-inch storm sewer	89,000	600
	4. 1,345 feet of 21-inch storm sewer	79,000	500
	5. 300 feet of 24-inch storm sewer	20,000	100
	6. 325 feet of 27-inch storm sewer	26,000	100
	7. 750 feet of 30-inch storm sewer	65,000	400
	8. 700 feet of 36-inch storm sewer	69,000	100
	9. 2,645 feet of 42-inch storm sewer	332,000	500
	10. 800 feet of 48-inch storm sewer	119,000	200
	11. 520 feet of 45-inch x 29-inch concrete H.E. storm sewer	56,000	100
	12. 540 feet of 51-inch x 31-inch RCPA storm sewer	73,000	100
	13. Detention Basin WD9 Incremental 100-year flood control storage volume of 5.1 acre-feet[d]	98,000	2,000
	Water Quantity Subtotal	$1,119,000	$ 5,500
	14. Construction erosion control, 6.1 acres per year	$ 183,000	$ 500
	15. Sweep 1.3 curb-mile of street	500	1,200
	16. Detention Basin WD9 Two-year flood control storage volume of 7.2 acre-feet[e]	132,000	3,700
	17. Detention Basin WD11 Permanent wet basin volume of 4.5 acre-feet	81,000	2,600
	Water Quality Subtotal	$ 396,500	$ 8,000
	Total	$1,515,500	$13,500
E	Area East of Lilly Creek and North and South of W. Mill Road		
	1. 420 feet of 15-inch storm sewer	$ 20,000	$ 100
	2. 825 feet of 21-inch storm sewer	48,000	300
	3. 1,340 feet of 24-inch storm sewer	83,000	600
	4. 490 feet of 27-inch storm sewer	34,000	200
	5. 1,515 feet of 30-inch storm sewer	117,000	600
	6. 2,075 feet of 36-inch storm sewer	193,000	400
	7. 1,870 feet of 42-inch storm sewer	210,000	400
	8. 1,200 feet of 48-inch storm sewer	162,000	200
	9. Detention Basin WD12 Incremental 100-year flood control storage volume of 7.0 acre-feet[d]	105,000	2,500
	10. Detention Basin DD5 Incremental 100-year flood control storage volume of 3.9 acre-feet[d]	52,000	1,400
	Water Quantity Subtotal	$1,024,000	$ 6,700
	11. Construction erosion control, 10.5 acres per year	$ 315,000	$ 800
	12. Detention Basin WD12 Two-year flood control storage volume of 10.5 acre-feet[e]	230,000	5,000
	13. Detention Basin DD5 Two-year flood control storage volume of 2.4 acre-feet[f]	51,000	1,800
	14. Detention Basin WD14 Permanent wet basin volume of 1.8 acre-feet	44,000	1,600
	Water Quality Subtotal	$ 640,000	$ 9,200
	Total	$1,664,000	$15,900

Table 49 (continued)

Hydrologic Unit	Project and Component Description[a]	Estimated Cost	
		Capital[b]	Annual Operation and Maintenance[c]
F	Lincoln Lane Tributary		
	1. 655 feet of 24-inch storm sewer	$ 41,000	$ 300
	2. 1,320 feet of 36-inch storm sewer	123,000	300
	3. 2,630 feet of 48-inch storm sewer	355,000	500
	4. Detention Basin WD13 Incremental 100-year flood control storage volume of 9.5 acre-feet[d,h]	156,000	3,800
	Water Quantity Subtotal	$ 675,000	$ 4,900
	6. Construction erosion control, 6.7 acres per year	$ 201,000	$ 500
	7. Detention Basin WD13 Permanent wet basin volume of 5.4 acre-feet[h]	107,000	3,000
	Water Quality Subtotal	$ 308,000	$ 3,500
	Total	$ 983,000	$ 8,400
G	Jerry Lane Tributary		
	1. 290 feet of 21-inch storm sewer	$ 17,000	$ 100
	2. 200 feet of 30-inch storm sewer	15,000	100
	3. 100 feet of 36-inch storm sewer	9,000	0
	4. 410-foot-long trapezoidal, turf-lined channel with 3-foot bottom width, 3H:1V side slopes, and 1-foot-deep, 2-foot-wide, riprap-lined, low-flow channel	20,000	200
	5. Detention Basin WD15 Incremental 100-year flood control storage volume of 9.7 acre-feet[d]	219,000	3,300
	6. Detention Basin WD22 Incremental 100-year flood control storage volume of 4.2 acre-feet[d]	73,000	1,600
	Water Quantity Subtotal	$ 353,000	$ 5,300
	7. Construction erosion control, 7.5 acres per year	$ 225,000	$ 600
	8. Detention Basin WD15 Two-year flood control storage volume of 10.6 acre-feet[e]	276,000	5,000
	9. Detention Basin WD22 Two-year flood control storage volume of 2.8 acre-feet[e]	56,000	2,000
	Water Quality Subtotal	$ 557,000	$ 7,600
	Total	$ 910,000	$12,900
H	Oakwood Tributary		
	South Branch		
	1. 600 feet of 27-inch storm sewer	$ 41,000	$ 200
	2. 805 feet of 30-inch storm sewer	62,000	300
	3. 450 feet of 48-inch storm sewer	61,000	100
	4. 970 feet of 54-inch storm sewer	151,000	200
	5. 250 feet of 60-inch storm sewer	46,000	0
	6. 960 feet of 66-inch storm sewer	207,000	200
	7. 1,500 feet of channel modification, including 1,000 feet of gabion-lined low-flow channel	63,000	600
	8. Replace existing culvert at Manor Hills Boulevard with a 6-foot-diameter, 51-foot-long reinforced concrete culvert	10,000	0
	9. Replace existing culvert at Lilly Road with a 6-foot-diameter, 110-foot-long reinforced concrete culvert	20,000	0
	10. Replace existing culvert at Faye Court/Memory Road with a 5-foot-diameter, 30-foot-long reinforced concrete culvert	4,000	0

Table 49 (continued)

Hydrologic Unit	Project and Component Description[a]	Estimated Cost	
		Capital[b]	Annual Operation and Maintenance[c]
H (continued)	North Branch		
	11. 2,320 feet of 36-inch storm sewer	$ 173,000	$ 400
	12. 315 feet of 42-inch storm sewer	26,000	100
	13. Detention Basin WD23 100-year flood control storage volume of 17.8 acre-feet	296,000	4,700
	North and South Branches		
	14. Detention Basin WD16 Incremental 100-year flood control storage volume of 28.0 acre-feet[d]	424,000	6,900
	Water Quantity Subtotal	$1,584,000	$13,700
	North and South Branches		
	15. Construction erosion control, 13.8 acres per year	$ 414,000	$ 1,000
	16. Detention Basin WD16 Two-year flood control storage volume of 38.8 acre-feet[e]	406,000	11,000
	Water Quality Subtotal	$ 820,000	$12,000
	Total	$2,402,000	$25,700
I	Area East of Lilly Creek and South of W. Good Hope Road		
	1. 2,100 feet of 2-foot-deep roadside swale with driveway culverts	$ 44,000	$ 1,300
	Water Quantity Subtotal	$ 44,000	$ 1,300
	2. Construction erosion control, 1.2 acres per year	$ 36,000	$ 100
	3. Detention Basin WD17 Permanent wet basin volume of 1.7 acre-feet	44,000	1,600
	Water Quality Subtotal	$ 80,000	$ 1,700
	Total	$ 124,000	$ 3,000
J	Woodshaven Tributary		
	1. 72 feet total of double 8-foot-wide x 4-foot-high concrete box culvert	$ 40,000	$ 0
	2. Replace existing culvert at Woodland Drive with 40 feet of 58-inch-wide x 36-inch-high RCPA culvert	7,000	0
	3. 470 feet of 36-inch storm sewer	44,000	100
	4. 115 feet of 36-inch reinforced concrete pipe culvert	9,000	0
	5. 350-foot-long trapezoidal, turf-lined channel with 5-foot bottom width, 3H:1V side slopes, and a 1-foot-deep, 2-foot-wide, riprap-lined low-flow channel	39,000	200
	6. Riprap along 0.3 mile of existing stream	13,000	300
	7. Floodproof two houses	10,000	- -
	8. Detention Basin WD19 Incremental 100-year flood control storage volume of 7.6 acre-feet[d]	123,000	2,800
	Water Quantity Subtotal	$ 285,000	$ 3,400
	9. Construction erosion control, 4.9 acres per year	$ 147,000	$ 400
	10. Detention Basin WD19 Two-year flood control storage volume of 8.5 acre-feet[e]	148,000	4,200
	11. Detention Basin WD26 Permanent wet basin volume of 1.3 acre-feet	35,000	1,400
	Water Quality Subtotal	$ 330,000	$ 6,000
	Total	$ 615,000	$ 9,400

Table 49 (continued)

Hydrologic Unit	Project and Component Description[a]	Estimated Cost	
		Capital[b]	Annual Operation and Maintenance[c]
K	Area along W. Appleton Avenue and East and West of Lilly Creek		
	1. Replace existing 36-inch storm sewer in Appleton Avenue with 566 feet of 42-inch storm sewer	$ 71,000	$ 100
	2. Replace existing 30-inch storm sewer in Appleton Avenue with 94 feet of 36-inch storm sewer	10,000	0
	3. Replace existing 27-inch storm sewer in Appleton Avenue with 142 feet of 30-inch storm sewer	12,000	100
	4. 790 feet of 36-inch storm sewer	74,000	200
	5. 240 feet of 42-inch storm sewer	27,000	0
	6. 115 feet of 48-inch storm sewer	16,000	0
	7. Floodproof one house	5,000	0
	Water Quantity Subtotal	$ 215,000	$ 400
	8. Construction erosion control, 2.6 acres per year	$ 78,000	$ 200
	9. Sweep 0.23 curb-mile of street	100	200
	10. Detention Basin WD18 Permanent wet basin volume of 4.0 acre-feet	109,000	2,500
	11. Detention Basin WD27 Permanent wet basin volume of 1.5 acre-feet	35,000	1,400
	Water Quality Subtotal	$ 222,100	$ 4,300
	Total	$ 437,100	$ 4,700
L	Menomonee Manor Tributary		
	1. 900 feet of 54-inch storm sewer	$ 156,000	$ 200
	2. 710 feet of 60-inch storm sewer	141,000	100
	3. 210 feet of 68-inch x 43-inch concrete H.E. storm sewer	43,000	0
	4. Riprap existing stream	6,000	100
	Water Quantity Subtotal	$ 346,000	$ 400
	5. Construction erosion control, 6.1 acres per year	$ 183,000	$ 500
	6. Detention Basin WD21 Two-year flood control storage volume of 8.8 acre-feet[e]	189,000	4,300
	7. Detention Basin WD24 Two-year flood control storage volume of 18.8 acre-feet[e]	331,000	7,600
	Water Quality Subtotal	$ 703,000	$12,400
	Total	$1,049,000	$12,800

[a] *All new and replacement storm sewers are concrete pipe.*

[b] *Capital costs include 35 percent for engineering, administration, and contingencies. Costs based on 1989 Engineering News-Record Construction Cost Index = 4,725. Costs would be incurred over the 20-year planning period from 1990 through 2010.*

[c] *Costs were noted to be zero when the project proposed replacement of a component with a component which has similar operation and maintenance costs, or when the annual operation and maintenance cost was estimated to be less than $50.*

[d] *The incremental volume is the volume in addition to the two-year flood control storage volume, or in addition to the permanent pond volume for those basins where two-year flood control is not recommended.*

[e] *For wet detention basins, this volume includes the permanent pond volume.*

[f] *Dry basin not designed for removal of nonpoint source pollutants.*

[g] *Cost includes low-flow diversion storm sewer, consisting of 740 lineal feet of 18-inch-diameter reinforced concrete storm sewer and 350 lineal feet of 51-inch by 31-inch reinforced concrete pipe arch storm sewer, plus 300 lineal feet total of double 42-inch-diameter reinforced concrete pipe for the basin outlet.*

[h] *This basin was constructed with a permanent pond volume of 11.7 acre-feet and an incremental 100-year flood control storage volume of 15.6 acre-feet.*

Source: SEWRPC.

Table 50

HYDROLOGIC AND HYDRAULIC CHARACTERISTICS OF RECOMMENDED DETENTION BASINS IN THE LILLY CREEK SUBWATERSHED

Basin Designation	Permanent Pond Area (acres)	Permanent Pond Volume (acre-feet)	Incremental Pond Volume for Control of a Two-Year, 24-Hour Storm[a] (acre-feet)	Total Pond Volume for Control of a Two-Year, 24-Hour Storm (acre-feet)	Required Available Storage Volume for Control of a 100-Year, 24-hour Storm[a,b] (acre-feet)	Total Pond Volume for Control of a 100-Year, 24-Hour Storm (acre-feet)	Peak Outflow from Detention Basin during a Two-Year, 24-Hour Storm (cfs)	Peak Outflow from Detention Basin during a 100-Year, 24-Hour Storm (cfs)
WD1	2.9	14.6	11.6	26.2	27.1	41.7	35	120
WD2	1.5	7.4	7.2	14.6	19.2	26.6	5	20
WD3	1.4	6.8	- -	- -	- -	- -	- -	- -
WD4	0.5	2.7	1.7	4.4	4.3	7.0	5	35
WD5	1.1	5.5	- -	- -	- -	- -	- -	- -
WD6	0.6	3.0	- -	- -	- -	- -	- -	- -
WD7	0.8	3.8	2.9	6.7	11.2	15.0	10	40
WD8	1.6	8.1	- -	- -	- -	- -	- -	- -
WD9	0.8	4.2	3.0	7.2	8.1	12.3	5	30
WD11[c]	0.9	4.5	- -	- -	- -	- -	- -	- -
WD12	1.4	7.0	3.5	10.5	10.5	17.5	25	110
WD13	1.1	5.4[d]	- -	- -	9.5[d]	14.9	- -	130
WD14	0.4	1.8	- -	- -	- -	- -	- -	- -
WD15	1.4	7.2	3.4	10.6	13.1	20.3	40	120
WD16	3.7	18.6	20.2	38.8	48.2	66.8	25	190
WD17	0.3	1.7	- -	- -	- -	- -	- -	- -
WD18	0.8	4.0	- -	- -	- -	- -	- -	- -
WD19	0.8	4.0	4.5	8.5	12.1	16.1	10	40
WD21	1.1	5.2	3.6	8.8	- -	- -	20	- -
WD22	0.5	2.2	0.6	2.8	4.8	7.0	10	30
WD23	- -[e]	- -[e]	6.9	6.9	17.8	17.8	15	65
WD24	2.1	10.5	8.3	18.8	- -	- -	50	- -
WD25	0.6	3.0	- -	- -	- -	- -	- -	- -
WD26	0.3	1.5	- -	- -	- -	- -	- -	- -
WD27	0.3	1.5	- -	- -	- -	- -	- -	- -
DD1	- -	- -	4.6	4.6	15.9	15.9	5	15
DD3	- -	- -	0.5	0.5	1.7	1.7	5	20
DD5	- -	- -	2.4	2.4	6.3	6.3	10	55
Total	26.5	134.2	82.0	165.6	198.6	271.9	- -	- -

[a] *Incremental volume above the permanent pond volume.*

[b] *Where applicable, the incremental 100-year flood control storage volumes given in Table 49 are the volumes above the two-year flood control storage volumes.*

[c] *Basins WD10 and WD11 are combined under final recommended plan.*

[d] *This basin was constructed with a permanent pond volume of 11.7 acre-feet and an incremental 100-year flood control storage volume of 15.6 acre-feet.*

[e] *Permanent pond eliminated under final recommended plan.*

Source: SEWRPC.

tions without causing flooding of any buildings. However, the intersections of Cheryln Drive and Bette Drive and of Elmway Drive and Bette Drive could be flooded to depths of from 0.6 to 0.8 foot under 10-year recurrence interval conditions, causing potential access problems for 10 houses. Provision of storm sewers along Bette Drive would eliminate street flooding during storms with recurrence intervals up to 10 years. If an adequate outlet were to be provided for the additional storm sewers, it would be necessary to extend the originally proposed 0.09-mile-long channel modification 0.53 mile farther east. The entire 0.62-mile length of the modification would include a two-foot-wide, one-foot-deep, riprap-lined low-flow channel. The lower 0.22 mile would consist solely of a low-flow channel, while the remaining 0.40 mile would require a trapezoidal channel with turf-lined sides and a riprap-lined bottom. The channel side slopes would be one vertical on three horizontal and the bottom width would be five feet. A single eight-foot-wide by four-foot-high reinforced concrete box culvert would be required at Pilgrim Road. Six pedestrian bridges would be removed and replaced by three new bridges.

Table 51

LAND USES TRIBUTARY TO RECOMMENDED WET DETENTION BASINS IN THE LILLY CREEK SUBWATERSHED

Hydrologic Unit "A"	WD1	
Land Use Category	Existing (1985) Area Tributary to Basin (acres)	Planned Ultimate Area Tributary to Basin (acres)
Urban		
Industrial	0.0	121.4
Commercial	0.0	0.0
Governmental and Institutional	0.0	0.0
Residential		
Low-Density	0.0	26.6
Medium-Density	12.4	17.2
Medium-High-Density	0.0	0.0
High-Density	0.0	0.0
Suburban-Density	0.0	0.0
Construction Site	0.0	8.1
Transportation, Commercial, and Utilities	0.0	0.0
Subtotal	12.4	173.3
Rural		
Prime-Agricultural	103.4	0.0
Parks and Recreation	0	0.0
Water	0	0.4
Woodlands	2.5	0.4
Wetlands	0	0.0
Other	55.8	0.0
Subtotal	161.7	0.8
Percent Urban	7	99

Hydrologic Unit "B"	WD2		WD3		WD4	
Land Use Category	Existing (1985) Area Tributary to Basin (acres)	Planned Ultimate Area Tributary to Basin (acres)	Existing (1985) Area Tributary to Basin (acres)	Planned Ultimate Area Tributary to Basin (acres)	Existing (1985) Area Tributary to Basin (acres)	Planned Ultimate Area Tributary to Basin (acres)
Urban						
Industrial	0.0	64.2	9.3	40.7	3.9	20.4
Commercial	0.0	0.0	5.4	0.0	9.5	0.0
Governmental and Institutional	0.0	0.0	0.0	0.0	0.0	0.0
Residential						
Low-Density	0.3	0.2	37.5	38.6	12.8	13.4
Medium-Density	0.0	12.9	0.0	7.4	0.0	0.0
Medium-High-Density	0.0	0.0	0.0	0.0	0.0	0.0
High-Density	0.0	0.0	0.0	0.0	0.0	0.0
Suburban-Density	0.0	0.0	0.0	5.7	0.0	0.0
Construction Site	0.0	4.1	0.0	2.4	0.0	0.9
Transportation, Commercial, and Utilities	0.0	0.0	9.1	9.2	0.9	0.8
Subtotal	0.3	81.4	61.3	104.0	27.1	35.5

Table 51 (continued)

Hydrologic Unit "B"	WD2		WD3		WD4	
Land Use Category	Existing (1985) Area Tributary to Basin (acres)	Planned Ultimate Area Tributary to Basin (acres)	Existing (1985) Area Tributary to Basin (acres)	Planned Ultimate Area Tributary to Basin (acres)	Existing (1985) Area Tributary to Basin (acres)	Planned Ultimate Area Tributary to Basin (acres)
Rural						
Prime-Agricultural	44.3	0.0	1.4	0.0	0.0	0.0
Parks and Recreation	0.0	0.0	0.0	0.0	0.0	0.0
Water	0.0	0.0	0.0	0.0	0.0	0.0
Woodlands	0.0	0.0	0.0	0.0	0.0	0.0
Wetlands	2.6	2.5	6.9	6.7	0.0	0.0
Other	36.8	0.0	41.0	0.0	8.4	0.0
Subtotal	83.7	2.5	49.3	6.7	8.4	0.0
Percent Urban	0.4	97	55	94	76	100

Hydrologic Unit "B"	WD5		WD6		WD25	
Land Use Category	Existing (1985) Area Tributary to Basin (acres)	Planned Ultimate Area Tributary to Basin (acres)	Existing (1985) Area Tributary to Basin (acres)	Planned Ultimate Area Tributary to Basin (acres)	Existing (1985) Area Tributary to Basin (acres)	Planned Ultimate Area Tributary to Basin (acres)
Urban						
Industrial	5.0	43.2	10.1	25.1	1.7	2.2
Commercial	0.0	0.0	0.0	0.0	0.0	0.0
Governmental and Institutional	0.0	0.0	4.4	3.4	2.8	0.0
Residential						
Low-Density	9.0	11.7	0.0	0.0	8.5	0.0
Medium-Density	0.0	0.0	0.0	0.0	0.0	0.0
Medium-High-Density	0.0	0.0	0.0	0.0	0.0	33.3
High-Density	0.0	0.0	0.0	0.0	0.0	0.0
Suburban-Density	0.0	0.0	0.0	0.0	0.0	0.0
Construction Site	0.0	2.2	0.0	0.8	0.0	1.7
Transportation, Commercial, and Utilities	0.0	0.0	0.0	0.0	0.0	0.0
Subtotal	14.0	57.1	14.5	29.3	13.0	37.2
Rural						
Prime-Agricultural	0.0	0.0	0.0	0.0	0.0	0.0
Parks and Recreation	2.6	0.0	0.0	0.0	0.0	0.0
Water	0.0	0.0	0.0	0.0	0.0	0.0
Woodlands	0.0	0.0	0.0	0.0	0.0	0.0
Wetlands	17.6	17.6	1.7	1.6	2.9	2.8
Other	40.5	0.0	14.8	0.0	26.3	0.9
Subtotal	60.7	17.6	16.5	1.6	29.2	3.7
Percent Urban	19	76	47	95	31	91

Table 51 (continued)

Hydrologic Unit "C"	WD7		WD8	
Land Use Category	Existing (1985) Area Tributary to Basin (acres)	Planned Ultimate Area Tributary to Basin (acres)	Existing (1985) Area Tributary to Basin (acres)	Planned Ultimate Area Tributary to Basin (acres)
Urban				
Industrial	0.0	0.0	42.0	61.3
Commercial	0.0	0.0	5.5	0.0
Governmental and Institutional	0.0	0.0	0.0	0.0
Residential				
Low-Density	60.7	58.4	21.0	23.3
Medium-Density	0.0	34.6	0.0	11.9
Medium-High-Density	0.0	0.0	0.0	0.0
High-Density	0.0	0.0	0.0	0.0
Suburban-Density	0.0	0.0	0.0	0.0
Construction Site	0.0	1.8	0.0	1.8
Transportation, Commercial, and Utilities	0.0	0.0	10.4	9.9
Subtotal	60.7	94.8	78.9	108.2
Rural				
Prime-Agricultural	0.0	0.0	0.0	0.0
Parks and Recreation	0.0	0.0	0.0	0.0
Water	0.0	0.0	0.0	0.0
Woodlands	0.0	0.0	4.4	4.4
Wetlands	0.0	0.0	0.0	0.0
Other	34.2	0.0	29.2	0.0
Subtotal	34.2	0.0	33.6	4.4
Percent Urban	64	100	70	96

Hydrologic Unit "D"	WD9		WD11	
Land Use Category	Existing (1985) Area Tributary to Basin (acres)	Planned Ultimate Area Tributary to Basin (acres)	Existing (1985) Area Tributary to Basin (acres)	Planned Ultimate Area Tributary to Basin (acres)
Urban				
Industrial	27.5	39.7	0.0	0.0
Commercial	0.0	0.0	0.0	0.0
Governmental and Institutional	0.0	0.0	0.0	0.0
Residential				
Low-Density	0.3	0.0	34.0	41.1
Medium-Density	0.0	0.0	7.8	56.9
Medium-High-Density	0.0	0.0	0.0	0.0
High-Density	0.0	0.0	0.7	0.0
Suburban-Density	0.0	0.0	0.0	0.0
Construction Site	0.0	2.4	0.0	3.0
Transportation, Commercial, and Utilities	0.5	0.5	0.0	0.0
Subtotal	28.3	42.6	42.5	101.0
Rural				
Prime-Agricultural	0.0	0.0	0.0	0.0
Parks and Recreation	0.0	0.0	0.0	0.0
Water	0.0	0.0	1.6	1.2
Woodlands	0.0	0.0	0.0	0.0
Wetlands	0.0	0.0	14.3	12.4
Other	14.0	0.0	56.3	0.0
Subtotal	14.0	0.0	72.2	13.6
Percent Urban	67	100	37	88

Table 51 (continued)

Hydrologic Unit "E"	WD12		WD14	
Land Use Category	Existing (1985) Area Tributary to Basin (acres)	Planned Ultimate Area Tributary to Basin (acres)	Existing (1985) Area Tributary to Basin (acres)	Planned Ultimate Area Tributary to Basin (acres)
Urban				
Industrial	0.0	24.5	0.0	0.0
Commercial	0.0	0.0	0.0	0.0
Governmental and Institutional	0.0	0.0	0.0	0.0
Residential				
Low-Density	2.9	9.6	0.5	0.0
Medium-Density	0.0	75.1	0.0	42.8
Medium-High-Density	0.0	0.0	0.0	0.0
High-Density	0.0	0.0	0.0	0.0
Suburban-Density	0.0	0.0	0.0	0.0
Construction Site	0.0	5.6	0.0	2.3
Transportation, Commercial, and Utilities	0.0	0.0	0.0	0.0
Subtotal	2.9	114.8	0.5	45.1
Rural				
Prime-Agricultural	0.0	0.0	0.0	0.0
Parks and Recreation	0.0	0.0	0.0	0.0
Water	0.0	0.0	0.0	0.0
Woodlands	14.5	14.9	2.2	0.0
Wetlands	8.4	2.3	1.3	0.0
Other	106.4	0.0	41.1	0.0
Subtotal	129.3	17.2	44.6	0.0
Percent Urban	2	87	1	100

Hydrologic Unit "F"	WD13	
Land Use Category	Existing (1985) Area Tributary to Basin (acres)	Planned Ultimate Area Tributary to Basin (acres)
Urban		
Industrial	0.0	121.4
Commercial	0.0	0.0
Governmental and Institutional	0.0	0.0
Residential		
Low-Density	5.2	5.1
Medium-Density	0.0	120.9
Medium-High-Density	0.0	0.0
High-Density	0.0	0.0
Suburban-Density	0.7	2.3
Construction Site	0.0	6.5
Transportation, Commercial, and Utilities	0.0	0.0
Subtotal	5.9	134.8
Rural		
Prime-Agricultural	0.0	0.0
Parks and Recreation	0.0	0.0
Water	0.0	0.0
Woodlands	0.0	0.0
Wetlands	5.7	0.0
Other	123.1	0.0
Subtotal	128.8	0.0
Percent Urban	4	100

Table 51 (continued)

Hydrologic Unit "G"	WD15		WD22	
Land Use Category	Existing (1985) Area Tributary to Basin (acres)[a]	Planned Ultimate Area Tributary to Basin (acres)[a]	Existing (1985 Area Tributary to Basin (acres)	Planned Ultimate Area Tributary to Basin (acres)
Urban				
Industrial	0.0	0.0	0.0	0.0
Commercial	0.0	0.0	0.0	0.0
Governmental and Institutional	0.0	0.0	0.0	0.0
Residential				
Low-Density	40.9	41.1	23.7	22.4
Medium-Density	0.0	109.7	0.0	26.5
Medium-High-Density	0.0	0.0	0.0	0.0
High-Density	0.0	0.0	0.0	0.0
Suburban-Density	0.5	2.4	0.2	2.1
Construction Site	0.0	5.9	0.0	1.4
Transportation, Commercial, and Utilities	0.0	0.0	0.0	0.0
Subtotal	41.4	159.1	23.9	52.4
Rural				
Prime-Agricultural	0.0	0.0	0.0	0.0
Parks and Recreation	0.0	0.0	0.0	0.0
Water	0.0	0.0	0.0	0.0
Woodlands	12.2	12.0	0.0	0.0
Wetlands	16.3	16.3	0.0	0.0
Other	117.5	0.0	27.9	0.0
Subtotal	146.0	28.3	27.9	0.0
Percent Urban	22	85	46	100

Hydrologic Unit "H"	WD16	
Land Use Category	Existing (1985) Area Tributary to Basin (acres)	Planned Ultimate Area Tributary to Basin (acres)
Urban		
Industrial	0.0	0.0
Commercial	0.0	0.0
Governmental and Institutional	12.0	18.2
Residential		
Low-Density	166.3	167.2
Medium-Density	0.0	247.1
Medium-High-Density	0.0	0.0
High-Density	0.0	0.0
Suburban-Density	0.0	0.0
Construction Site	0.0	13.4
Transportation, Commercial, and Utilities	1.1	0.0
Subtotal	179.4	445.9
Rural		
Prime-Agricultural	0.0	0.0
Parks and Recreation	0.0	0.0
Water	0.0	0.0
Woodlands	0.0	0.0
Wetlands	4.9	0.0
Other	261.6	0.0
Subtotal	266.5	0.0
Percent Urban	40	100

Table 51 (continued)

Hydrologic Unit "I"	WD17	
Land Use Category	Existing (1985) Area Tributary to Basin (acres)	Planned Ultimate Area Tributary to Basin (acres)
Urban		
Industrial	0.0	0.0
Commercial	0.0	0.0
Governmental and Institutional	0.0	0.0
Residential		
Low-Density	17.6	33.3
Medium-Density	0.0	0.0
Medium-High-Density	0.0	0.0
High-Density	0.0	0.0
Suburban-Density	0.0	0.0
Construction Site	0.0	0.8
Transportation, Commercial, and Utilities	0.0	0.0
Subtotal	17.6	34.1
Rural		
Prime-Agricultural	0.0	0.0
Parks and Recreation	0.0	0.0
Water	0.0	0.0
Woodlands	0.0	0.0
Wetlands	0.0	0.0
Other	27.7	11.2
Subtotal	27.7	11.2
Percent Urban	39	75

Hydrologic Unit "J"	WD19		WD26	
Land Use Category	Existing (1985) Area Tributary to Basin (acres)	Planned Ultimate Area Tributary to Basin (acres)	Existing (1985) Area Tributary to Basin (acres)	Planned Ultimate Area Tributary to Basin (acres)
Urban				
Industrial	0.0	0.0	0.0	0.0
Commercial	0.0	0.0	0.0	0.0
Governmental and Institutional	0.6	0.0	0.0	0.0
Residential				
Low-Density	38.9	43.2	1.4	1.4
Medium-Density	0.0	53.2	0.0	0.0
Medium-High-Density	0.0	0.0	11.2	11.1
High-Density	0.0	0.0	0.0	0.0
Suburban-Density	0.0	0.0	0.0	0.0
Construction Site	0.0	3.0	0.0	0.1
Transportation, Commercial, and Utilities	0.0	0.0	0.0	0.0
Subtotal	39.5	99.4	12.6	12.6
Rural				
Prime-Agricultural	0.0	0.0	0.0	0.0
Parks and Recreation	0.0	0.0	0.0	0.0
Water	0.0	0.0	0.0	0.0
Woodlands	0.0	0.0	0.0	0.0
Wetlands	0.0	0.0	0.0	0.0
Other	59.9	0.0	0.0	0.0
Subtotal	59.9	0.0	0.0	0.0
Percent Urban	40	100	100	100

Table 51 (continued)

Hydrologic Unit "K"	WD18		WD27	
Land Use Category	Existing (1985) Area Tributary to Basin (acres)	Planned Ultimate Area Tributary to Basin (acres)	Existing (1985) Area Tributary to Basin (acres)	Planned Ultimate Area Tributary to Basin (acres)
Urban				
Industrial	0.0	0.0	0.0	0.0
Commercial	19.5	24.4	0.0	0.0
Governmental and Institutional	0.0	0.0	0.0	0.0
Residential				
Low-Density	18.5	17.8	1.8	0.0
Medium-Density	0.0	2.1	0.0	0.0
Medium-High-Density	0.0	11.1	0.0	12.3
High-Density	0.0	0.0	0.0	0.0
Suburban-Density	0.0	0.0	0.0	0.0
Construction Site	0.0	1.0	0.0	0.6
Transportation, Commercial, and Utilities	0.0	0.0	0.0	0.0
Subtotal	38.0	56.4	1.8	12.9
Rural				
Prime-Agricultural	0.0	0.0	0.0	0.0
Parks and Recreation	1.9	0.2	0.0	0.0
Water	0.0	0.0	0.0	0.0
Woodlands	0.0	0.0	0.0	0.0
Wetlands	5.5	5.7	0.0	0.0
Other	16.7	0.0	10.6	0.0
Subtotal	24.1	5.9	10.6	0.0
Percent Urban	61	90	14	100

Hydrologic Unit "L"	WD21		WD24	
Land Use Category	Existing (1985) Area Tributary to Basin (acres)	Planned Ultimate Area Tributary to Basin (acres)	Existing (1985) Area Tributary to Basin (acres)	Planned Ultimate Area Tributary to Basin (acres)
Urban				
Industrial	0.0	0.0	0.0	0.0
Commercial	4.9	8.2	25.4	33.4
Governmental and Institutional	18.5	15.4	4.0	4.0
Residential				
Low-Density	16.7	7.9	96.1	88.6
Medium-Density	5.1	5.4	0.0	79.6
Medium-High-Density	0.0	15.1	0.0	4.9
High-Density	2.2	0.0	2.3	0.0
Suburban-Density	0.0	0.0	0.1	0.1
Construction Site	0.0	1.0	0.0	4.5
Transportation, Commercial, and Utilities	0.0	0.0	0.0	0.0
Subtotal	47.4	53.0	127.9	215.1
Rural				
Prime-Agricultural	0.0	0.0	0.0	0.0
Parks and Recreation	0.0	0.0	0.0	0.0
Water	0.0	0.0	0.0	0.0
Woodlands	0.0	0.0	0.0	0.0
Wetlands	0.0	0.0	0.0	0.0
Other	5.5	0.0	87.2	0.0
Subtotal	5.5	0.0	87.2	0.0
Percent Urban	90	100	60	100

[a] *Includes all area which is also tributary to WD22.*

Source: SEWRPC.

The proposed channel modification was discussed during the December 11, 1992, interagency staff meeting which was attended by the Village's staff; the Wisconsin DNR staff; staff from BRW, Inc., the Village's consultant for the development of environmental enhancement and stream restoration measures for the Lilly Creek channel; and the Regional Planning Commission staff. At that meeting, the DNR staff unequivocally stated that the modification of the Phillips Tributary channel required to accommodate a new storm sewer in Bette Drive was unacceptable to the Department and that it would be highly unlikely that permits for such a project would be granted. The primary reasons given by the Department for the unacceptability of the channel modification were potential adverse impacts on a stream reach with valuable aquatic habitat and the potential impacts of the project on the adjacent wetland.

The DNR staff suggested the investigation of two possible alternatives which would provide an adequate outlet for the proposed Bette Drive storm sewer: 1) extending the proposed storm sewer downstream along the south bank of Phillips Tributary until an adequate outlet is provided and 2) extending the proposed storm sewer to the south, across W. Silver Spring Road to the Silver Spring Tributary.

Village staff stated that the Village had given preliminary approval to a plat which calls for the extension of Bette Drive and Scott Lane to the west and south, to W. Silver Spring Road. That extension would provide ingress and egress for new development and also for seven of the 10 houses where ingress and egress would be affected during a 10-year recurrence interval storm under existing drainage conditions. Therefore, a third alternative to the provision of a storm sewer in Bette Drive and the attendant modification of the Phillips Tributary channel would be to retain the recommendation of stormwater drainage Alternative Plan No. 3, presented in Chapter V of this report, and to extend Bette Drive as approved by the Village. Alternative Plan No. 3 in Chapter V did not call for the installation of storm sewers in Bette Drive and retained the existing Phillips Tributary channel downstream of the east side of Pilgrim Road. Therefore, under planned development conditions with Bette Drive extended, ingress to and egress from all new development and seven of the 10 existing houses would be provided and modification of the Phillips Tributary channel would be avoided.

The first alternative suggested by the DNR was analyzed and it was found that in order to implement that alternative it would be necessary to extend the proposed Bette Drive storm sewer 0.77 mile along Phillips Tributary to Lilly Creek. In addition, the Lilly Creek channel would have to be lowered about 15 feet to accommodate the outfall. The cost of the additional storm sewer alone would be about $730,000. The alternative was considered impractical because of its extremely high cost and because of the need to lower the Lilly Creek streambed significantly in order to implement the alternative.

The second alternative suggested by that Department would require extending the proposed Bette Drive storm sewer about 570 feet south to the Silver Spring Tributary and deepening the Silver Spring Tributary a maximum of about seven feet, resulting in a maximum channel depth of eleven to twelve feet below Silver Spring Drive. This alternative would have a total cost of about $300,000. The alternative was also eliminated from further consideration because of its relatively high cost and because the construction of the deepened channel would create a safety hazard along Silver Spring Drive, an arterial street.

The third alternative, extending Bette Drive and Scott Lane to the west and south to W. Silver Spring Road, as preliminarily approved by the Village, not installing a storm sewer in Bette Drive, and not modifying the Phillips Tributary channel downstream of the east side of Pilgrim Road, is recommended to be implemented. That alternative would eliminate new storm sewers, channel modifications, and pedestrian bridge replacements, with a total capital cost of about $250,000 and, during a 10-year recurrence interval storm, it would provide ingress to and egress from new development as well as for seven of the 10 houses which do not have such ingress and egress under existing drainage conditions. Because the extension of Bette Drive and Scott Lane is a municipal improvement preliminarily approved by the Village and necessary for the development of additional land, the cost of such an extension is not assigned to this plan.

The staff of the Village Department of Public Works specifically requested that an adequate outlet be provided for surface water runoff from the existing subdivision located along El Camino Drive, Mesa Drive, and Vista Lane north of the Phillips Tributary in the southwest one-quarter of U. S. Public Land Survey Section 26, Township 8 North, Range 20 East. Thus, the recommended plan calls for the construction of 1,020 lineal feet of 12-inch-diameter storm sewer and 500 lineal feet of 15-inch-diameter storm sewer to convey runoff from the subdivision through a wetland and to Phillips Tributary. Because the DNR has specifically ruled out deepening of the reach of Phillips Tributary to which the storm sewers discharge, it is not possible to achieve sufficient slope on the pipes to convey the runoff from a 10-year recurrence interval storm. Thus, the recommended storm sewers are sized to fit the available slope from the existing subdivision outlets to the Phillips Tributary channel and would function to drain ponded water which currently collects for the lack of an outlet. The recommended storm sewers would be located in a wetland. An alternatives analysis as required under Chapter NR 103 of the Wisconsin Administrative Code is provided in Appendix D of this report. That analysis demonstrates that the recommended storm sewers are the only practicable alternative available to abate the problem concerned.

The recommended plan calls for the use of storm sewer conveyance facilities in areas of planned medium-density residential and industrial development. To eliminate flooding of the commercial area on the west side of Pilgrim Road, the existing culverts under Pilgrim Road would be replaced with three eight-foot-wide by four-foot-high reinforced concrete box culverts and the 475-foot-long reach of the Phillips Tributary channel located west of Pilgrim Road would be deepened and widened. The modified, trapezoidal, turf-lined channel would have a 10-foot bottom width and side slopes of one vertical on three horizontal. A one-foot-deep, two-foot-wide, riprap-lined low-flow channel would be provided.

Floodproofing would be required for two houses located along Phillips Tributary east of Pilgrim Road.

Single-purpose detention basins WD3, WD5, WD6, and WD25 would be constructed to provide control of nonpoint source pollution and dual-purpose detention basins WD2 and WD4 would provide both water quantity and water quality control. At the request of Village's staff, basin WD6 was moved from its preliminarily recommended location west of Shawn Circle to a location south of Shawn Circle. Also, as discussed in Appendix D of this report, basin WD2 was reconfigured to avoid a wetland area. Together, the six wet detention basins would control runoff from about 71 percent of the hydrologic unit area. The total area in each land use category which is tributary to the basins under both existing and planned ultimate land use conditions is given in Table 51. In addition, nonpoint source pollution control would be provided through the maintenance of most of the existing roadside swale and open channel system in the residential portions of the hydrologic unit and through weekly sweeping of all nondetained commercial and industrial streets in spring, summer, and fall.

Dry detention basin DD1 would be located between Kohler Lane and the Chicago & North Western Railway on land already purchased by the Village for the purpose of providing detention storage. Existing natural detention storage areas ND2 and ND3 would be preserved to provide water quantity control benefits.

Hydrologic Unit C, Bowling Green Tributary

Approximately 68 percent of the land in this hydrologic unit was in urban uses as of 1985. It is anticipated that about 98 percent of the hydrologic unit would be in urban uses under ultimate planned land use conditions.

The recommended plan for this hydrologic unit calls for the continued use of roadside swales in areas of existing residential development, supplemented by new storm sewers along the alignment of the main collector channel through the Hiawatha Estates subdivision; the construction of storm sewers in the Bowling Green Industrial Park; construction of dry detention basin DD3, wet detention basin WD8, and dual-purpose detention basin WD7; and diversion of the Bowling Green Tributary to Lilly Creek, avoiding the Industrial Park. The preliminary recommended plan presented in Chapter V of this report has been refined to address wetland issues as discussed in Appendix D of this report, to reflect detailed subdivision development plans provided to the Village following the formulation of the preliminary recommendation, and to address the requests of the Village's staff regarding the provision of storm sewers to improve system maintenance in areas with relatively deep roadside swales.

The preliminary recommended drainage plan called for the existing roadside swale system in the Hiawatha Estates subdivision in the northwestern part of the hydrologic unit to be maintained in conjunction with the construction of upstream detention basin DD3. However, the Village's public works and engineering staff reported maintenance problems associated with the relatively deep roadside swales located along How Avenue and Pontiac Drive in the subdivision. Thus, the final recommended plan calls for the installation of storm sewers beginning at the outlet of basin DD3 and extending along Pontiac Drive and How Avenue. These storm sewers would connect with storm sewers which are proposed for the Mill Ridge subdivision and which would discharge to detention basin WD7. The installation of the recommended storm sewers would enable the use of shallower roadside swales with flatter side slopes. Such swales would be easier to maintain.

The configuration of detention basin WD7 and of the drainage facilities tributary to the basin were refined on the basis of the proposed detailed drainage plan prepared by the developer of the Mill Ridge subdivision. That proposal differs from the preliminary recommendation made in Chapter V in that the developer's plan calls for WD7 to provide water quantity control and the developer's plan calls for a storm sewer, rather than an open channel, along the Tributary upstream of WD7. The water quantity control function of WD7 was eliminated under the preliminary recommended plan since the peak flow reduction provided by WD7 would not significantly affect the preliminarily recommended downstream open channel conveyance facilities. However, because of the limitation on modification of the Lilly Creek channel resulting from the stream restoration recommendations set forth in Appendix C of this report, quantity control in WD7 could provide flood control benefits within the context of the final recommended plan. The provision of quantity control in basin WD7 would also enable the substitution of piped diversion for open channel diversion of the Bowling Green Tributary. Therefore, basin WD7 is recommended to be a dual-purpose basin with storage volumes and release rates as specified in Table 50.

As discussed in Appendix D of this report, the diversion facility alignment was reexamined in order to avoid disturbance of a wetland located along the originally proposed diversion route. In the course of that reexamination, it was found that a piped diversion would be superior to an open channel diversion, since a piped diversion would have lower total capital and annual operation and maintenance costs and would place fewer restrictions on future development along the route of the diversion. Therefore, the preliminary recommended drainage plan was revised to call for a 1,200-foot-long, 36-inch-diameter reinforced concrete diversion pipe to be located to the north of the wetland in the southeast one-quarter of the northeast one-quarter of Section 26.

Recommended wet basins WD7 and WD8 would treat runoff from about 78 percent of the hydrologic unit area. The total area in each land use category which is tributary to the basins under both existing and planned ultimate land use conditions is given in Table 51. Low flows from the areas tributary to the recommended storm sewers in Kaul and Bobolink Avenues would be diverted from those sewers into basin WD8 using 740 lineal feet of 18-inch-diameter reinforced concrete storm sewer and 350 lineal feet of 51-inch-wide by 31-inch-high reinforced concrete pipe arch storm sewer. The diversion storm sewers would be laid at elevations below the proposed sewers in Kaul and Bobolink and would divert low flows from those storm sewers at common manholes located in the two streets. In addition, nonpoint source pollution control would be provided through the maintenance of much of the existing roadside swale system in the residential portions of the hydrologic unit and through weekly sweeping of all nondetained commercial and industrial streets in spring, summer, and fall.

It should be noted that, because of past toxic materials spills and materials handling practices, contaminated soil and groundwater may be encountered during the construction of recommended facilities in the Bowling Green Industrial Park. Special remediation measures would be required in such instances.

Hydrologic Unit D, Area Predominantly West of Lilly Creek and North and South of W. Mill Road

Approximately 45 percent of the land in this hydrologic unit was in urban uses as of 1985. It is anticipated that about 93 percent of the hydrologic unit would be in urban uses under ultimate planned land use conditions.

Under existing drainage conditions, catchment areas LCG01 and LCG02 are located in Hydrologic Unit E. For the stormwater management plan prepared for the Village in 1984 by Ruekert & Mielke, Inc., Consulting Engineers, it was assumed that drainage from those areas would be rerouted to the south into what is here designated as Hydrologic Unit D. The rerouting of drainage from LCG01 and LCG02 was also assumed for the analysis of alternatives presented in Chapter V of this report; the basis for that rerouting was a small savings in cost by conveying runoff from LCG01 and LCG02 to Lilly Creek over a shorter distance. In formulating the recommended plan, it was decided to convey runoff from LCG01 and LCG02 in the existing drainage direction, which is to the north along the east side of Lilly Road in Hydrologic Unit E. That decision was made to avoid potential difficulties under the rerouting proposal whereby runoff from storms with recurrence intervals of 10 years or less would be conveyed to the south into Hydrologic Unit D, while runoff exceeding that from a 10-year storm would be conveyed to the north in the direction of existing drainage.

Storm sewer conveyance is recommended in W. Mill Road and in areas of new, medium-density residential development. Following preparation of the preliminary recommended stormwater management plan, more detailed drainage and grading plans were prepared by the developer of the areas of planned medium-density residential development in the Mill Ridge subdivision in catchment areas LCF03 and LCF04. The concept of those plans was similar to the preliminary recommendations; however, the final recommended plan presented here has been refined to be consistent with the developer's plans and to recommend modifications to the developer's plans where necessary to meet the stormwater management objectives of this system plan. Under the final recommended plan, natural detention area ND4 would be provided with a storm sewer outlet to convey runoff from storms with recurrence intervals up to, and including, 10 years and an overflow swale would be provided to convey outflow from ND4 in excess of that from a 10-year storm. The developer's plan generally provides adequate major drainage system hydraulic capacity, but this system plan recommends that an overflow swale be provided to the east of the intersection of the proposed Mill Ridge Drive and the existing drainage swale along the back lot lines of the low-density residential development along W. Mill Road and the adjacent planned medium-density residential development along the proposed Ash Drive. The combined capacity of that swale and the recommended storm sewers should be adequate to convey the runoff from a 100-year recurrence interval storm across Mill Ridge Drive and beyond the proposed houses on the east side of Mill Ridge Drive without flooding any existing or proposed houses.

The refined recommended plan calls for the consolidation of single-purpose wet detention basins WD10 and WD11 into one enlarged wet detention basin, designated WD11. The enlarged basin WD11 was moved east of its preliminarily recommended location to accommodate the subdivision layout. The enlarged basin would be expected to achieve the same degree of treatment of nonpoint source pollution as was called for under the preliminary plan. Basin WD11 was constructed during the time that this plan was being prepared.

The recommended plan calls for storm sewer conveyance, supplemented by dual-purpose detention basin WD9, in areas of existing and planned industrial development. Because it would not detain runoff from catchment areas LCG01 and LCG02, basin WD9 would be reduced in size in comparison to the basin called for under drainage Alternative No. 3. The two wet basins would treat runoff from about 64 percent of the hydrologic unit area. The total area in each land use category which is tributary to the basins under both existing and planned ultimate land use conditions is given in Table 51. Existing natural detention basin ND4 would be preserved to provide water quantity and quality control benefits. Additional nonpoint source control would be provided through weekly sweeping of all nondetained industrial streets in spring, summer, and fall.

Hydrologic Unit E, Area East of Lilly Creek and North and South of W. Mill Road

Approximately 4 percent of the land in this hydrologic unit was in urban uses as of 1985. Under ultimate planned land use conditions, it is anticipated that about 93 percent of the hydrologic unit would be in urban uses.

As set forth in the preceding discussion of the recommended measures for Hydrologic Unit D, catchment areas LCG01 and LCG02 were included in Hydrologic Unit E under recommended plan conditions. That inclusion resulted in an increase in both the permanent wet pond and flood control area and volume of dual-purpose detention basin WD12, relative to preliminary recommended drainage Alternative No. 3.

The recommended plan calls for the provision of storm sewer conveyance in areas of planned medium-density residential development, supplemented by detention storage in basin WD12 and single-purpose dry basin DD5. As discussed in Chapter V of this report, the water quantity control portion of basin WD14 was eliminated during the evaluation of the impacts of off-channel detention storage on Lilly Creek flood elevations. The plan still calls for construction of WD14 as a single-purpose wet basin which, along with WD12, would treat runoff from about 75 percent of the hydrologic unit area. The total area in each land use category which is tributary to the basins under both existing and planned ultimate land use conditions is given in Table 51.

The preliminary recommended drainage alternative called for storm sewers to be constructed through the wetland in subbasin LCG07 in the southeast quadrant of the hydrologic unit. As discussed in Appendix D of this report, under the final recommended plan those storm sewers have been rerouted around the wetland, avoiding disturbance of the wetland.

Hydrologic Unit F, Lincoln Lane Tributary

Approximately 16 percent of the land in this hydrologic unit was in urban uses as of 1985. It is anticipated that all of the hydrologic unit would be in urban uses under ultimate planned land use conditions.

The recommended plan provides storm sewer conveyance in areas of planned medium-density residential development. The preliminary recommended drainage alternative called for storm sewers to be constructed through the wetland in subbasin LCH01 and LCH02 in the western part of the hydrologic unit. As discussed in Appendix D of this report, under the final recommended plan those storm sewers have been rerouted around the wetland, avoiding disturbance of the wetland. The areas of existing low-density residential development which are currently served by roadside swales would also be served by swales under planned conditions.

The storage volume provided by dual-purpose detention basin WD13 would reduce 100-year recurrence interval flows to a level that would not exceed the hydraulic capacity of the existing 60-inch-diameter reinforced concrete storm sewer in Lincoln Lane. Basin WD 13 would treat nonpoint source pollutants in the runoff from about 84 percent of the hydrologic unit area. The total area in each land use category which is tributary to the basin under both existing and planned ultimate land use conditions is given in Table 51.

Prior to publication of this report, basin WD13 and appurtenant storm sewers in the southeast one-quarter of U. S. Public Land Survey Section 23 were constructed as shown on Map 20. Basin WD13 was designed to provide the level of control of the 100-year storm which is called for under this system plan.

Hydrologic Unit G, Jerry Lane Tributary

Approximately 27 percent of the land in this hydrologic unit was in urban uses as of 1985. It is anticipated that about 88 percent of the hydrologic unit would be in urban uses under ultimate planned land use conditions.

The recommended plan calls for the preservation of about 0.40 mile of the existing Jerry Lane Tributary in its existing state. Valuable habitat is provided by the stream, which flows through a wetland and woodland area that is to be preserved under planned ultimate land use conditions. Dual-purpose detention basin WD22 would be located near the upstream end of the tributary, outside the wetland. That basin would protect the water quality of the stream through the removal of nonpoint source pollutants washed off from areas of existing low-density and planned medium-density residential development. The basin would also provide extended detention storage to control flood flows and the more frequently occurring flows which determine the size and shape of the natural low-flow channel. At the suggestion of the DNR staff, the riprap lining proposed to be placed in the existing streambed under the preliminary recommended stormwater management plan has been eliminated from the recommended plan. The potential for disturbance to the stream channel and riparian lands during placement of the riprap was felt to outweigh the possible erosion control benefits of the riprap.

The plan calls for the construction of a 410-foot-long section of modified channel to accommodate the outlet from basin WD22. The modified channel would be trapezoidal in shape with a three-foot bottom width, one vertical on three horizontal side slopes, and a small low-flow channel. Natural vegetation would be allowed to reestablish itself in and along the channel. As discussed in Appendix D of this report, the channel length was reduced to 410 feet to avoid disturbance of a wetland.

Areas of planned medium-density residential development would be served by storm sewers. The areas of existing low-density residential development which are currently served by roadside swales would also be served by swales under planned conditions.

Dual-purpose detention basin WD15 would be located at the downstream end of the existing natural channel where it enters an existing 66- to 72-inch-diameter reinforced concrete storm sewer. The storage volume provided by WD15 would reduce 100-year recurrence interval flows to a level that would not exceed the hydraulic capacity of the existing downstream storm sewer. Wet basins WD15 and WD22 would control runoff from about 77 percent of the hydrologic unit area. The total area in each land use category which is tributary to the basins under both existing and planned ultimate land use conditions is given in Table 51.

Basin WD15 would be located in a wetland. An alternatives analysis as required under Chapter NR 103 of the Wisconsin Administrative Code is provided in Appendix D of this report. That analysis demonstrates that the recommended basin is the only practicable alternative.

Hydrologic Unit H, Oakwood Tributary

Approximately 44 percent of the land in this hydrologic unit was in urban uses as of 1985. It is anticipated that all of the hydrologic unit would be in urban uses under ultimate planned land use conditions.

Under the recommended plan, the 0.37-mile-long reach of Oakwood Tributary from its mouth at Lilly Creek upstream to the confluence of the North and South Branches of Oakwood Tributary would remain an open channel. To match the proposed Lilly Creek streambed elevation at the confluence of Lilly Creek and Oakwood Tributary, thereby enabling fish migration between streams, the streambed would be lowered in the downstream 0.29-mile reach of Oakwood Tributary. As shown on Figure 22, the lower 1,000 feet of that reach would have a cross-section consisting of a low-flow channel with stepped, vertical gabion sidewalls surmounted by grassed slopes at one vertical on two horizontal. The channel cross-section in the upstream 500 feet of the channel reach would have a trapezoidal shape with turf lining, a four-foot bottom width, and side slopes ranging from one vertical on 2.5 horizontal to one vertical on three horizontal.

To accommodate the lowered channel as well as the future widening of Lilly Road, as called for under the Waukesha County Jurisdictional Highway Plan, the existing culvert at Lilly Road would be replaced with a six-foot-diameter, 110-foot-long reinforced concrete pipe culvert. The existing culvert at Manor Hills Boulevard would be replaced with a six-foot-diameter, 51-foot-long reinforced concrete culvert; a five-foot-diameter, 30-foot-long replacement culvert would be provided at Faye Court/Memory Road.

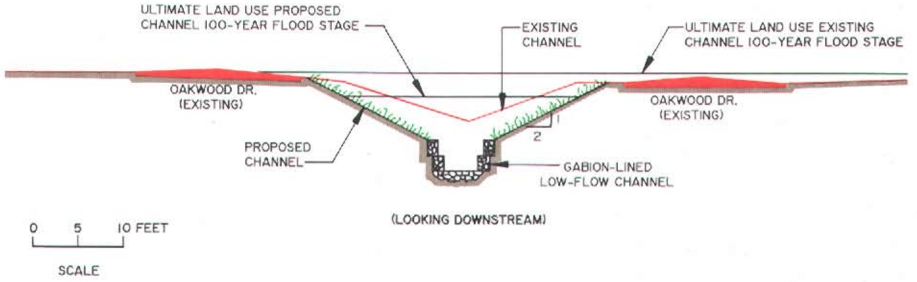
Dual-purpose detention basin WD16 would be constructed on the North and South Branches of Oakwood Tributary just upstream of their confluence. That basin would control runoff from a large area which includes existing low-density residential and governmental and institutional uses and planned medium-density residential and governmental and institutional uses.

Storm sewer conveyance would be provided in the areas of planned medium-density residential development tributary to the South Branch of the Oakwood Tributary and also in Terrace Drive, which follows the alignment of the South Branch through an existing subdivision with low-density development. It is proposed that the existing roadside swales along Terrace Drive remain in place and that the new storm sewer be constructed below the swales. The remainder of the streets in that subdivision and in the other existing subdivisions in the hydrologic unit would be served by roadside swales, as under existing conditions. The preliminary recommended stormwater management plan was expanded to include the new storm sewers required to connect the proposed storm sewer in Terrace Drive with the existing storm sewer outfalls in W. Good Hope Road and Pilgrim Road.

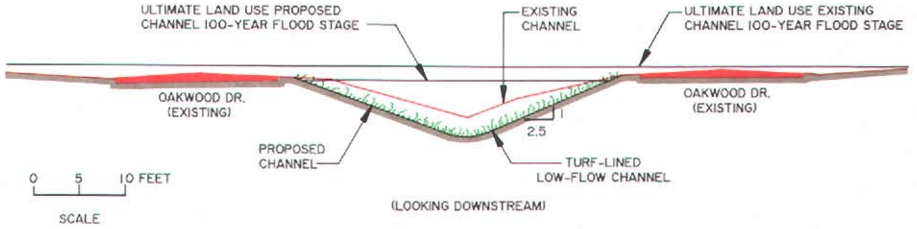
Figure 22

TYPICAL CROSS-SECTIONS OF EXISTING AND PROPOSED CHANNEL ALONG THE OAKWOOD TRIBUTARY FROM RIVER MILE 0.00 TO RIVER MILE 0.29

RIVER MILE 0.00 TO 0.19



RIVER MILE 0.19 TO 0.29



Source: SEWRPC.

A portion of the storm sewer recommended along the South Branch would be located in a wetland. An alternative analysis as required under Chapter NR 103 of the Wisconsin Administrative Code is provided in Appendix D of this report. That analysis demonstrates that the recommended storm sewer is the only practicable alternative.

The North Branch of Oakwood Tributary would remain as an open channel from the upper end of WD16 to the outlet of dual-purpose basin WD23, which is proposed to be located northwest of the intersection of W. Good Hope Road and Woodland Drive. As requested by the Village's staff, outflow from WD23 would be conveyed in storm sewers south to W. Good Hope Road and then east in W. Good Hope Road to the existing channel of the North Branch of Oakwood Tributary. As a result, under planned conditions, the existing North Branch channel from WD23 to W. Good Hope Road would convey only local inflows and outflows in excess of the 10-year recurrence interval flood from basin WD23. Areas of planned medium-density residential development along the North Branch would be served by storm sewers.

Because construction of the W. Good Hope Road storm sewer would cause the smaller floods which transport most of the nonpoint source pollutant load to bypass the 0.25-mile-long reach of the North Tributary upstream of W. Good Hope Road, and because runoff conveyed by that storm sewer would be treated in Basin WD16 prior to reaching Lilly Creek, the permanent pond originally called for in WD23 was no longer considered to be necessary and was eliminated under the recommended plan. Elimination of that pond would have no significant effect on nonpoint source pollutant loadings to Lilly Creek.

Basin WD16 would treat runoff from about 91 percent of the hydrologic unit area. The total area in each land use category which is tributary to the basin under both existing and planned ultimate land use conditions is given in Table 51. Basin WD16 would be located in a wetland. An NR 103 alternatives analysis, which demonstrates that the basin is the only practicable alternative, is presented in Appendix D of this report.

Hydrologic Unit I, Area East of Lilly Road and South of W. Good Hope Road

Approximately 78 percent of the land in this hydrologic unit was in urban uses as of 1985. It is anticipated that about 92 percent of the hydrologic unit would be in urban uses under ultimate planned land use conditions.

Under existing conditions, most of this hydrologic unit consists of low-density residential development which is served either by roadside swales or by roadside swales supplemented with storm sewers, as exist along Brentwood and Harding Drives, Claas Road, and Nicolet Court. Under planned conditions, it is recommended that stormwater drainage be accomplished and nonpoint source pollution be controlled through the use of roadside swales in the area of planned low-density residential development in catchment area LCL10. To enhance the control of nonpoint source pollutants by facilitating infiltration, it is recommended that the roadside swales in catchment area LCL10 have a three-foot-wide bottom.

It is also recommended that single-purpose wet detention basin WD17 be provided at the outlet of LCL10. That basin would treat runoff from about 26 percent of the area of the hydrologic unit. The total area in each land use category which is tributary to the basin under both existing and planned ultimate land use conditions is given in Table 51.

Hydrologic Unit J, Woodshaven Tributary

Approximately 70 percent of the land in this hydrologic unit was in urban uses as of 1985. It is anticipated that all of the hydrologic unit would be in urban uses under ultimate planned land use conditions.

The recommended plan calls for the preservation of 1.1 miles of the Woodshaven Tributary channel in essentially its existing state.

The existing unmodified low-flow channel in the lower 0.4-mile reach of the stream would be protected through erosion control measures. The erosion control could be accomplished through the provision of riprap, or it may be possible to utilize an alternative approach, such as soil bioengineering.

In the 350-foot-long reach upstream of Lilly Road, the channel would be deepened slightly and widened. The modified channel would be trapezoidal in shape with a five-foot bottom width and one vertical on three horizontal side slopes. Natural vegetation would be allowed to reestablish itself along the modified stream.

Floodproofing is recommended for one house on the east side of Northwood Drive and one house on the west side of Northwood Drive. It is also recommended that the culverts at Lilly Road and Woodland Drive be replaced to alleviate potential flooding under 100-year recurrence interval conditions.

In the upper reach of the tributary, planned medium-density residential development would be served by storm sewers which would connect to the existing storm sewer in Colony Road. Dual-purpose detention basin WD19 would control flood flows and nonpoint source pollution. In addition, the basin would provide extended detention storage to regulate frequently occurring storms, thereby controlling erosion of the downstream channel. Single-purpose wet basin WD26 would control nonpoint source pollution. The two wet basins would collect runoff from about 39 percent of the hydrologic unit area. The total area in each land use category which is tributary to the basins under both existing and planned ultimate land use conditions is given in Table 51.

Hydrologic Unit K, Area along W. Appleton Avenue and East and West of Lilly Creek

Approximately 72 percent of the land in this hydrologic unit was in urban uses as of 1985. It is anticipated that about 94 percent of the hydrologic unit would be in urban uses under ultimate planned land use conditions.

The recommended plan calls for the maintenance of roadside swale drainage in areas of existing suburban- and low-density residential development. The existing storm sewers in W. Appleton Avenue between Lilly Road and Lilly Creek would be replaced with larger sewers and storm sewer conveyance would be provided for areas of planned medium-high-density residential development.

One single-family residence located southwest of the intersection of Lilly Road and W. Appleton Avenue would be floodproofed. The low point of the intersection is above the grade at the house; therefore, even with upgrading of the storm sewers in W. Appleton Avenue, floodproofing would be required to protect the house during a 100-year recurrence interval storm.

It is recommended that single-purpose wet detention basins WD18 and WD27 be constructed. The two wet basins would collect runoff from about 36 percent of the total hydrologic unit area. The total area in each land use category which is tributary to the basins under both existing and planned ultimate land use conditions is given in Table 51.

Hydrologic Unit L,
Menomonee Manor Tributary

Approximately 66 percent of the land in this hydrologic unit was in urban uses as of 1985. It is anticipated that about 96 percent of the hydrologic unit would be in urban uses under ultimate planned land use conditions.

Significant medium-density residential development has occurred in the western portion of this hydrologic unit since 1985. That development is served by storm sewers and three onsite dry detention basins which were sized in accordance with the Village's interim detention policy as set forth in Chapter II of this report. Within Hydrologic Unit L, this plan recommends maintenance of the existing mix of storm sewers in areas of medium-density residential development; roadside swales with trunk storm sewers in areas of low-density residential development; roadside swales in areas of suburban-density residential development; and storm sewers in areas of commercial, governmental, and institutional development.

The preliminary recommended stormwater management plan called for a parallel 54-inch storm sewer to be constructed next to the existing 48- and 54-inch storm sewers and the proposed 60-inch sewer which run from Manor Drive, across North Point Drive and W. Appleton Avenue, to Lilly Creek. Subsequent to formulation of that preliminary recommendation, the redevelopment plan for the North Pointe Centre was revised by the developer to accommodate a different building configuration than was originally proposed. That revision substituted a realigned and shortened 54-inch-diameter reinforced concrete storm sewer for the originally-proposed 60-inch storm sewer. That 54-inch storm sewer was constructed. In addition to the new 54-inch-diameter storm sewer, it is recommended that a parallel storm sewer be constructed as shown on Map 20.

The recommended plan calls for the construction of wet detention basins WD21 and 24 to treat runoff from a total of about 75 percent of the hydrologic unit area. Those basins would also provide erosion control benefits along Menomonee Manor Tributary and its North Branch by controlling frequently occurring storms up to those with a two-year recurrence interval. The total area in each land use category which is tributary to the basins under both existing and planned ultimate land use conditions is given in Table 51. Basin WD24 would be located in a wetland. An NR 103 alternatives analysis, which demonstrates that the basin is the only practicable alternative, is presented in Appendix D of this report.

RECOMMENDED FLOOD CONTROL PLAN

As discussed in the sections of Chapter V of this report dealing with the description of the alternative flood control plans and the selection of the preliminary recommended flood control plan for the main stem of Lilly Creek, the channel modification and bridge removal or replacement alternative was selected as the preliminary recommended flood control plan, subject to refinements to improve aquatic and riparian habitat.

The final recommended flood control plan for Lilly Creek calls for the construction of a 2.08-mile-long widened and deepened channel. The components of the recommended flood control plan are shown on Map 20, and its principal features and costs are listed in Table 52. The recommended streambed profile and 100-year recurrence interval flood water surface profile under planned land use, channel, and drainage conditions are shown on Figure 23. Typical existing and modified channel cross-sections are shown on Figure 24. A comparison of two-, 10-, and 100-year recurrence interval flood flows under various combinations of land use and channel conditions is given in Table 53.

Table 52

PRINCIPAL FEATURES AND COSTS OF THE RECOMMENDED FLOOD CONTROL PLAN FOR THE LILLY CREEK WATERSHED

Name	Description[a]	Costs			
		Capital[b,c]	Amortized Capital[d]	Annual Operation and Maintenance	Total
Channel Modification and Bridge Removal or Replacement	Widening and deepening of 2.08 miles of channel	$2,725,000[e]	$173,000	$5,000	$178,000
	Removal and replacement of six road bridges	326,000	20,700	- -	20,700
	Removal of three pedestrian bridges and replacement of two of those bridges	89,000	5,700	- -	5,700
	Reconstruction and relocation of Menomonee Manor Boulevard	250,000	15,900	- -	15,900
	Additional easements along modified channel	20,000	1,300	- -	1,300
	Floodproofing of one house	25,000	1,600	- -	1,600
	Stream restoration measures in reaches where channel is to be modified[f]	180,000	11,400	- -	11,400
Total		$3,615,000	$229,600	$5,000	$234,600

[a] *A single replacement bridge for the existing W. Mill Road and Lilly Road bridges is called for under the recommended plan. The bridge replacement would be implemented as part of the arterial street improvements recommended in the Waukesha County Jurisdictional Highway Plan and its cost is, therefore, not assigned to this flood control plan.*

[b] *Capital costs include 35 percent for engineering, administration, and contingencies. Costs based on 1989 Engineering News-Record Construction Cost Index: 4,725.*

[c] *The cost of the water quantity control portions of added detention basins WD7, WD12, and DD5 are included in Table 49, which sets forth the costs of the recommended stormwater management plan.*

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

[e] *Includes the cost of construction erosion control measures.*

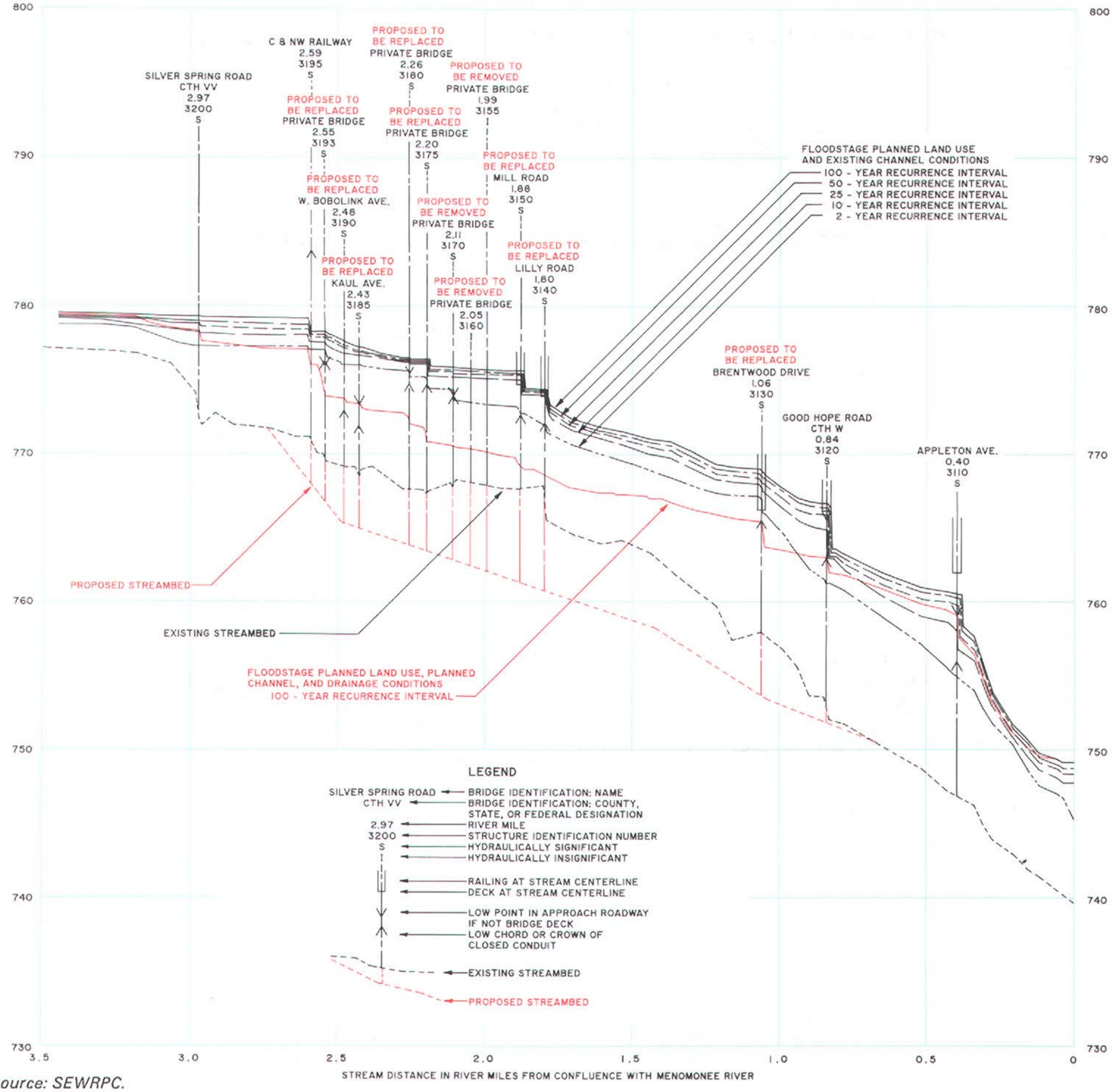
[f] *The stream enhancement plan presented in Appendix C of this report also calls for an additional $2,300 of stream habitat enhancement in Reach 2 of Lilly Creek between River Mile 0.28 and 0.66 and $156,000 for park creation and recreational trail construction.*

Source: SEWRPC.

The recommended modified channel would essentially be located along the alignment of the existing Lilly Creek channel, except near the intersection of W. Mill Road and Lilly Road, where a short reach of channel realignment is recommended to eliminate a right-angle bend in the channel and to replace two bridges with a single structure. The Village of Menomonee Falls purchased easements along an alignment proposed in 1984 for a channel modification project which was not constructed. With the exception of an 0.38-mile-long reach of channel immediately upstream of W. Mill Road, the channel alignment proposed under this alternative would be essentially the same as that provided by the existing easements. The recommended channel alignment and streambed profile were also designed to avoid conflicts with

Figure 23

RECOMMENDED PLAN FLOOD STAGE PROFILE FOR LILLY CREEK



Source: SEWRPC.

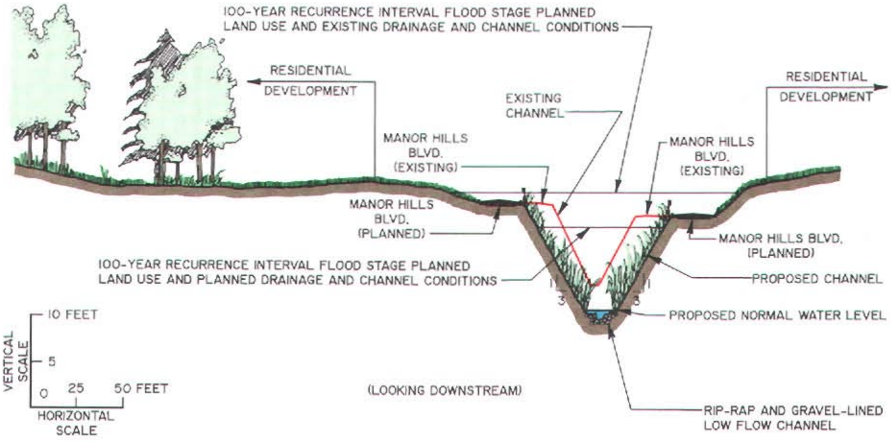
the Lilly Creek sanitary interceptor sewer constructed by the Village in the late 1980s and early 1990s.

The preliminary recommended flood control plan set forth in Chapter V of this report called for reinstatement of the water quantity control aspects of detention basins WD12 and DD5, both in Hydrologic Unit E, in order to reduce the 100-year recurrence interval flood flow in Lilly Creek at its confluence with the Menomonee River. Under this final recommended plan, the water quantity control portion of detention basin WD7 in Hydrologic Unit C has also been included, since that basin is located in an area where preliminary subdivision plans are likely to be approved by the Village prior to the adoption of this system plan and it is likely that

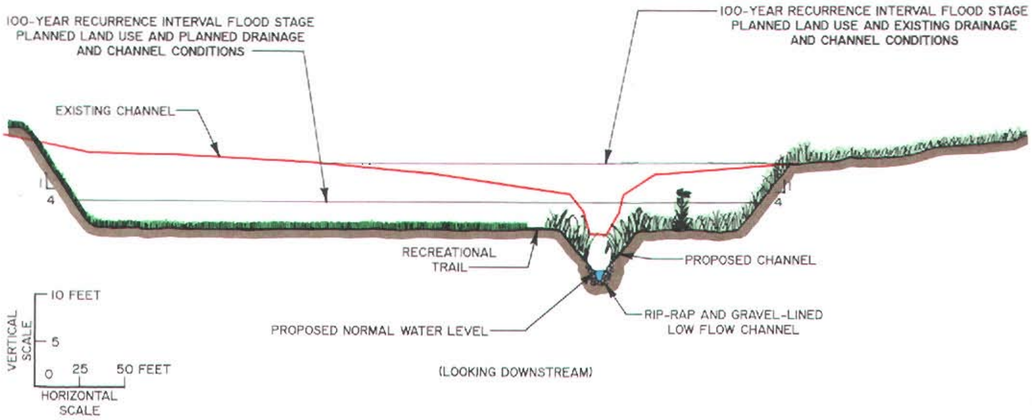
Figure 24

TYPICAL CROSS-SECTIONS OF EXISTING AND PROPOSED CHANNEL ALONG LILLY CREEK

REACH 4: RIVER MILE 0.84—1.37



REACH 5: RIVER MILE 1.37—1.71



REACH 6: RIVER MILE 1.71—1.88

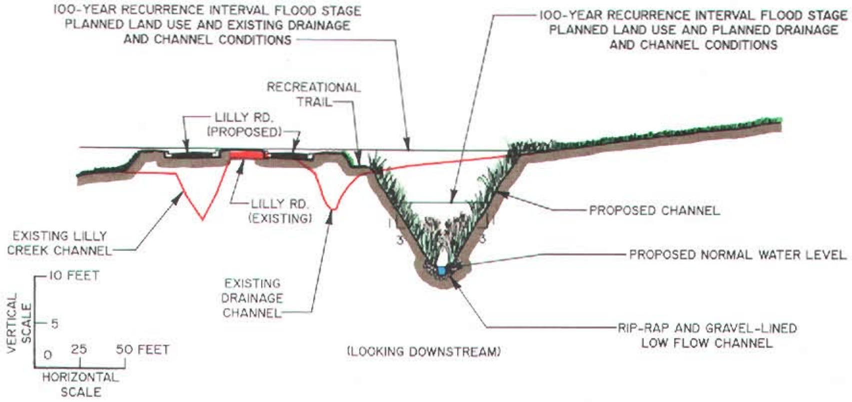
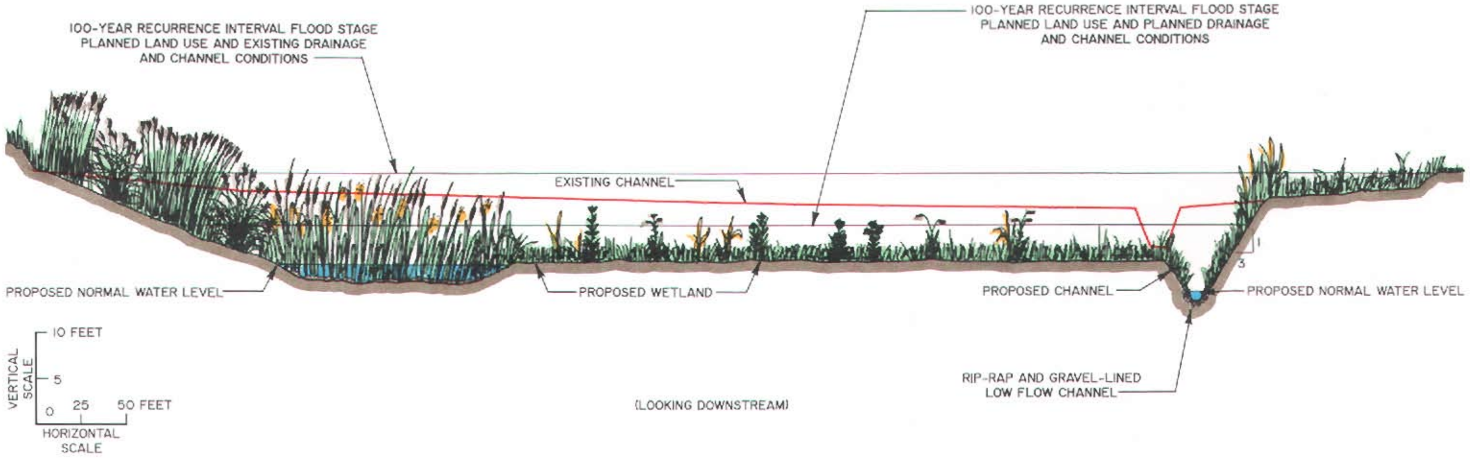
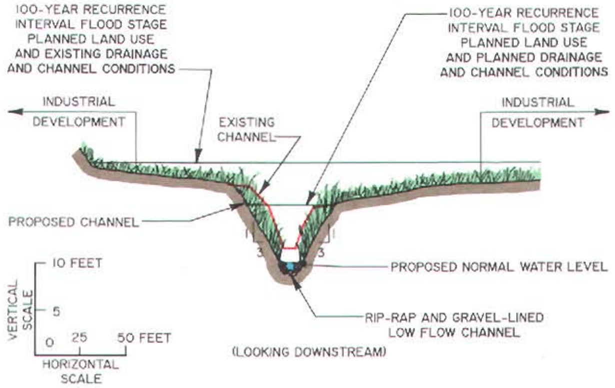


Figure 24 (continued)

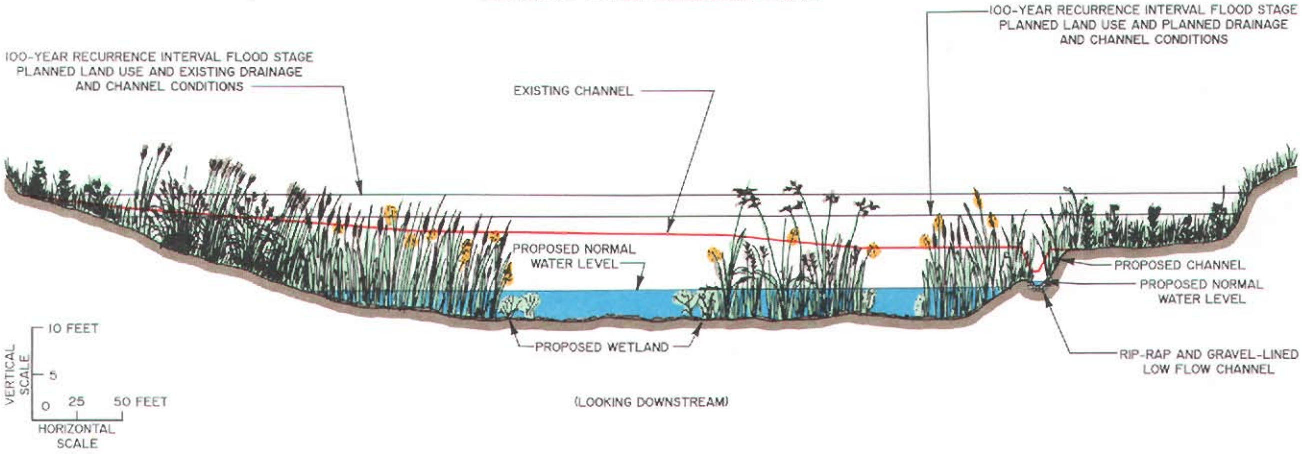
REACH 7: RIVER MILE 1.88—2.32



REACH 8: RIVER MILE 2.32—2.59



REACH 9: RIVER MILE 2.59—2.74



Source: BRW, Inc., and SEWRPC.

Table 53

COMPARISON OF TWO-, 10-, AND 100-YEAR RECURRENCE INTERVAL FLOOD FLOWS IN LILLY CREEK

River Mile	Existing (1985) Land Use, Drainage and Channel Condition Flows (cfs)			Planned Ultimate Land Use and Existing Lilly Creek and Tributary Channel Condition Flows[a,b] (cfs)			Planned Ultimate Land Use and Recommended Stormwater Management and Flood Control Plan Flows (cfs)		
	2-Year	10-Year	100-Year	2-Year	10-Year	100-Year	2-Year	10-Year	100-Year
0.0 (mouth)	680	1,510	2,590	940	1,740	2,810	540	1,110	1,890
0.06	600	1,310	2,260	820	1,540	2,500	480	960	1,630
0.40 (W. Appleton Avenue)	600	1,310	2,260	820	1,540	2,500	480	960	1,630
0.78	500	1,060	1,840	700	1,310	2,210	410	820	1,370
0.84 (W. Good Hope Road)	500	1,060	1,840	700	1,310	2,210	410	820	1,370
0.85	500	1,060	1,830	700	1,300	2,200	410	820	1,370
0.99	480	1,020	1,770	680	1,270	2,140	400	800	1,340
1.06 (Brentwood Drive)	480	1,020	1,770	680	1,270	2,140	400	800	1,340
1.07	470	1,000	1,730	670	1,240	2,100	400	790	1,320
1.16	470	1,000	1,720	670	1,230	2,080	400	780	1,310
1.22	470	1,000	1,720	670	1,220	2,070	400	780	1,310
1.29	470	990	1,700	660	1,210	2,040	390	770	1,300
1.37	340	650	1,070	450	780	1,260	370	680	1,080
1.71	270	450	810	340	560	870	310	580	950
1.88 (W. Mill Road)	210	350	640	240	380	760	250	450	710
1.89	210	350	640	240	380	750	240	430	670
2.19	210	370	600	250	420	660	210	380	590
2.37	210	350	550	240	390	620	190	330	490
2.43 (Kaul Avenue)	210	350	550	240	390	620	190	330	490
2.44	200	340	520	230	370	580	170	300	460
2.48 (Bobolink Avenue)	200	340	520	230	370	580	170	300	460
2.59 (C&NW Railway)	200	340	520	230	370	580	170	300	460
2.60	130	280	440	180	340	490	150	280	450
2.85	120	270	420	180	340	470	130	260	450
2.97 (W. Silver Spring Drive)	60	130	150	90	130	160	70	140	200

[a] *Assumes no new detention storage is provided and storm sewer conveyance components are provided in areas of new development.*

[b] *It is assumed that the existing hydraulic structures in Lilly Creek at W. Mill Road and Lilly Road are replaced with a single structure consisting of two reinforced concrete box culverts extending across the intersection of W. Mill Road and Lilly Road.*

Source: SEWRPC.

the basin would be required under the Village's stormwater management guidelines, which are being applied in the absence of an approved stormwater management plan. As stated in a previous section of this chapter, the inclusion of basin WD7 also eliminates the need to purchase property for the construction of the recommended Bowling Green Tributary diversion, since the basin reduces flows to a level which enables the economical construction of a piped diversion.

A consultant was retained by the Village to prepare a report which documents refinements to the recommended flood control plan both through the development of environmental restoration and enhancement measures and through the addition of recreational facilities which are compatible with the riparian setting. That report, prepared by BRW, Inc., is presented in Appendix C of this report.

The BRW report recommended the following: 1) limiting modification of the Lilly Creek channel to the reach upstream of River Mile 0.66, 2) creation of wetlands in the west overbank southeast of the intersection of W. Mill Road and Lilly Road between River Miles 1.90 and 1.98 and in the west overbank between River Miles 2.60 and 2.69 south of the Chicago & North Western Railway, 3) construction of a recreational trail along the stream corridor from W. Appleton Avenue to W. Mill Road, 4) widening of the secondary environmental corridor in the west overbank from River Mile 1.37 to 1.71 between Lilly Road and Lilly Creek and south of Manor Hills Boulevard, 5) stream restoration measures in reaches where the channel is recommended to be modified for flood control purposes, including construction of a low-flow channel, bank preservation, establishment of pools and riffles, substrate enhancement, bank revegetation emphasizing shading of the low-flow channel, provision of riprap erosion protection at hydraulic structures, and provision of meander guides to control bank erosion, and 6) stream enhancement measures in reaches where no channel modification is recommended, including riprap and gabion erosion protection at hydraulic structures and on the outside of existing meanders and substrate enhancement through the addition of gravel to the streambed.

The two wetlands which are recommended to be created and the widened segment of the secondary environmental corridor to be located east of Lilly Road are designed to provide flood storage capacity which would reduce peak flood flows and stages along Lilly Creek. As zones of low flow velocities during floods, those flood storage areas would also provide refuge for fish at times when main channel flow velocities would be high. The BRW report also provides recommendations regarding the type of vegetation to be established in the modified channel reaches and in the wetlands which are recommended to be created. As set forth in Appendix C, the vegetation to be established in the channel would consist primarily of grasses which would require no mowing, would grow to heights of from two to five feet, and which would tend to lie down during large floods and reduce the resistance to flow.

The BRW recommendations were developed in the context of the preliminary recommended channel modification plan for flood control as set forth Chapter V of this report. The main changes to the preliminary recommended plan which were required to incorporate the stream enhancement and restoration measures were the elimination of channel widening and deepening between River Mile 0.21 and River Mile 0.66 and the provision of additional flood storage in the overbanks.

Reaches 1 and 2, from the Mouth of Lilly Creek at the Menomonee River to River Mile 0.66 South of W. Appleton Avenue

No channel modifications are recommended in these reaches. Recommended stream enhancement measures include the provision of riprap and gabion bank erosion controls at the locations indicated on Map 20 and removal of debris from the channel.

Because no channel modifications are recommended for this reach, it is recommended that the exposed basement of one house located at W138 N7336 Melville Drive be floodproofed. Implementation of the recommended stormwater management and flood control plan would reduce the frequency of flooding of that house. Plan implementation would also reduce the peak 100-year recurrence interval flood stage at that house under planned land use conditions by about 1.2 feet through the provision of detention storage, which would reduce the peak flood flow by about 28 percent. However, the basement could still be flooded to a depth of up to 2.4 feet during a 100-year flood, making the recommendation for floodproofing necessary. The house might be floodproofed through construction of a floodwall or by sealing the basement walls and providing removable bulkhead closures for doorways. In order for the floodproofing to be in compliance with Chapter NR 116 of the Wisconsin Administrative Code, which deals with floodplain management, it would be necessary to floodproof the basement to a height two feet above the 100-year flood elevation of 760.8 feet NGVD29 under planned land use, drainage, and channel conditions. Thus, if a floodwall were constructed, it would have a maximum height of about 4.3 feet. Floodproofing measures should be designed by a registered professional engineer experienced in structural design practices. Floodproofing the house would not enable its official removal from the floodplain nor would it remove the requirement that flood insurance be obtained if the house were sold.

Reach 3, from River Mile 0.66 to River Mile 0.83, Downstream of W. Good Hope Road

The recommended modifications in this reach would consist only of a one-foot-deep, four-foot-wide, riprap-lined low-flow channel, with pools and riffles created at the areas shown on Map 20 and with meander guides provided to avoid erosion of the bank at the existing bend in the channel near River Mile 0.66. The recommended recreational trail would be located along the east bank of Lilly Creek.

Reach 4, from W. Good Hope Road at River Mile 0.84 through the Intersection of Oakwood Drive and Manor Hills Boulevard

In this reach, Lilly Creek runs through an area of existing residential development which encroaches closely on the channel, limiting the width of the recommended modified flood control channel. As shown on Figure 24, the modified channel would be essentially trapezoidal in shape, with average side slopes of one vertical on three horizontal and a one-foot-deep, four-foot-wide low-flow channel. The flood control channel bottom width would be twelve feet with transitions to wider sections at culverts. As shown on Figure 23, the existing streambed would be lowered a maximum of about five feet.

The existing 25.5-foot-wide by 16.75-foot-high elliptical structural plate pipe culvert at W. Good Hope Road was constructed with its invert located about seven feet below the existing streambed. That culvert would be retained and the streambed would be lowered 1.6 foot within the pipe. The existing 15.2-foot-wide by 7.6-foot-high corrugated metal arch at Brentwood Drive would be replaced with two 10-foot-wide by eight-foot-high reinforced concrete box culverts. The streambed would be lowered about 4.3 feet at the culverts. Within this reach, two lanes of Manor Hills Boulevard are located along each side of the existing stream channel. The proposed channel widening and deepening would require relocation of both sides of the boulevard. The 1984 channel modification design provided for such relocation. The modified flood control channel cross-sectional shape, side slopes, dimensions, and streambed profile are the same as, or similar to, those proposed by the Village in 1984. Therefore, the recommended channel modification could be accommodated within the parameters of the street relocation originally proposed by the Village. The 1984 street relocation in the Village called for the grade of Manor Hills Boulevard to be lowered by up to about 1.2 feet. Under the flood control plan presented here, it is recommended that the grade of the relocated street be about the same as the existing street so as to avoid street flooding along the length of Manor Hills Boulevard during a 100-year recurrence interval flood. During the engineering design phase, the Village may give consideration to using structural slope stabilization measures which would permit the use of steeper channel side slopes in this reach. The use of steeper slopes might enable modification of the street relocation recommendation. Bank stabilization could be accomplished in an environmentally sensitive manner which would meet the habitat objectives of this plan.

Stream restoration measures recommended for this reach include the creation of pool and riffle areas in the locations shown on Map 20, the provision of meander guides to avoid streambank erosion at the bend in the channel at Nicolet Court, and the provision of riprap erosion protection at hydraulic structures and drainage channel junctions with Lilly Creek. It is recommended that the recreational trail be located in the street right-of-way along Manor Hills Boulevard, Brentwood Drive, and Claas Road.

Reaches 5 and 6, from River Mile 1.37 to the Intersection of W. Mill Road and Lilly Road at River Mile 1.88

There is considerable open land available in this reach, thus, as shown on Figure 24, a widened channel section is recommended to provide additional flood storage capacity and fish refuge areas. The widened and deepened channel and overbank cross-section was designed to accommodate the proposed widening of Lilly Road and, with the exception of the reach between River Miles 1.37 and 1.53, the limits of the widened cross-section were restricted to the 100-year recurrence interval floodplain limits under planned land use and existing channel conditions. Between River Miles 1.37 and 1.53, the widened east overbank was extended beyond the existing floodplain limits to provide sufficient storage volume while avoiding significant disturbance of three residential lots in the west overbank where houses were built since 1990. The grades of the widened overbank were set so that the overbank would not be inundated during floods with recurrence intervals up to, and including, two years. Thus, on the average, use of the expanded secondary environmental corridor would be interrupted for only a relatively short time about once every two years.

Flood control channel bottom widths between River Miles 1.41 and 1.61 would range from about 40 to 100 feet and total floodplain widths during a 100-year flood would range from about 100 feet to 280 feet. Modified channel side slopes would be one vertical on four horizontal and a one-foot-deep, four-foot-wide low-flow channel would be constructed. Upstream of River Mile 1.61, flood control channel bottom widths would transition back to 12 feet and side slopes would be steepened to one vertical on three horizontal. The streambed would be lowered from five to seven feet within the entire reach.

The existing 72-inch-diameter Jerry Lane storm sewer is located perpendicular to the channel in the west overbank at approximately River Mile 1.45. Construction of the west overbank storage area would require excavation adjacent to that storm sewer, but the sewer itself would remain in place.

The bridges at Lilly Road and W. Mill Road would be replaced with a single 275-foot-long structure aligned across the intersection of the roads. The replacement structure would consist of two 10-foot-wide by seven-foot-high reinforced concrete box culverts. The existing streambed would be lowered about seven feet at the replacement structure. The length and alignment of the proposed structure would be approximately the same as those proposed under the 1984 channel modification project. Because the bridge replacement would be implemented as part of the arterial street improvements recommended in the Village's Land Use and Transportation Plan, the cost of replacement was not assigned to this flood control plan. The existing hydraulic structures under Lilly Road consist of a bridge which is in a deteriorated condition and an 86-inch-wide by 54-inch-high corrugated metal pipe arch. The Village plans to replace the bridge based on structural considerations. A replacement structure would need hydraulic capacity to convey only local inflow from the area west of Lilly Road.

The recommended recreation trail would be located along the west side of the modified flood control channel. Stream restoration measures recommended in this reach include the provision of pools and riffles within the low-flow channel and the provision of riprap erosion protection at stormwater drainage outfalls.

Reach 7, from the Intersection of W. Mill Road and Lilly Road at River Mile 1.88 to the North Boundary of the Bowling Green Industrial Park at River Mile 2.32

As shown on Figure 23, the recommended plan calls for the existing streambed to be lowered by about four to seven feet in this reach. The main flood control channel would have a 12-foot bottom width and one vertical on three horizontal side slopes. A one-foot-deep, four-foot-wide, riprap-lined low-flow channel would also be provided.

As shown on Figure 24, it is recommended that a wetland be constructed in the west overbank between River Miles 1.90 and 1.98. The existing west bank of Lilly Creek would be preserved in that reach. The connection between the wetland and the main Lilly Creek channel would be at elevation 766 feet National Geodetic Vertical Datum, 1929 adjustment (NGVD29) in order to allow frequent flooding of the wetland during floods with recurrence intervals of less than two years. The recommended stormwater management plan element calls for wet detention basin WD 11 to be located in the same area as the recommended wetland. Basin WD11 was constructed as part of the Mill Ridge subdivision. The recommended wetland is conceptually consistent with the recommendation for the provision of wet detention for the control of water quality. At such time as the flood control project is designed and constructed, the wetland can be designed to accommodate basin WD11.

There are existing wetlands located immediately west of the existing Lilly Creek channel between River Miles 1.98 and 2.14 and also between River Miles 2.26 and 2.32, respectively. The recommended modified channel would extend about 50 feet into the approximately 400-foot-wide wetland between River Miles 1.98 and 2.14, disturbing about one acre of the 8.5-acre wetland, and about 40 feet into the 650-foot-wide wetland between River Miles 2.26 and 2.32, disturbing about 0.3 acre of the 4.6-acre wetland. As discussed in the practicable alternatives analysis presented in Appendix D of this report, construction of the modified channel along essentially the same alignment as the existing channel is the only practicable alternative and it is, therefore, recommended.

In the reach from W. Mill Road to the private drive at River Mile 2.26, the 1984 channel modification proposed by the Village called for the modified channel to be realigned and moved a maximum of about 105 feet to the west of the existing channel. The alignment proposed by the Village would have eliminated the need for replacement of the existing pedestrian bridges which provide access to the west side of properties which lie on either side of the existing channel. Four easements required for that channel relocation were not obtained by the Village because of objections by property owners. The recommended alignment along the existing channel is selected because the alignment proposed by the Village was not accepted by the four property owners and because the alignment originally proposed by the Village would disturb more of the wetland along the west bank than would the recommended alignment.

The recommended plan calls for the removal of the three pedestrian bridges at River Miles 1.99, 2.05, and 2.11 and replacement of those bridges with two structures which would cause an insignificant obstruction to flows under 100-year recurrence interval flood conditions. The two bridges carrying private drives at the Brahm property at River Mile 2.20 and at the Weyer property at River Mile 2.26 would both be replaced with double 112-inch-wide by 75-inch-high corrugated metal pipe arch culverts. The streambed would be lowered about four feet at each of those proposed structures.

Recommended stream restoration measures for this reach in addition to the wetland creation and preservation of the west bank include creation of pools and riffles and the provision of riprap erosion protection at culverts.

Reach 8, from the North Boundary of the Bowling Green Industrial Park at River Mile 2.32 to the Chicago & North Western Railway at River Mile 2.59

In this reach, Lilly Creek flows through an area of existing industrial development which encroaches closely on the channel, limiting the width of the recommended modified flood control channel. As shown on Figure 24, the modified channel would be essentially trapezoidal, with average side slopes of one vertical on three horizontal and a one-foot-deep, four-foot-wide, riprap-lined low-flow channel. The flood control channel bottom width would be from 10 to 12 feet, with transitions to wider sections at culverts. As shown on Figure 23, the existing streambed would be lowered a maximum of about 4.5 feet. The modified flood control channel cross-section shape, side slopes, dimensions, and streambed profile are the same as, or similar to, those proposed by the Village in 1984. Therefore, the recommended channel modification could be accommodated within the easements which have been obtained by the Village.

The existing bridge at Kaul Avenue would be replaced with two 10-foot-wide by six-foot-high reinforced concrete box culverts. The streambed would be lowered about 4.3 feet at the culverts.

Beginning upstream of Kaul Avenue at River Mile 2.44 and extending to the Chicago & North Western Railway embankment at River Mile 2.59, a 10-foot modified channel bottom width, with appropriate transitions at bridges, would be provided. The Bobolink Avenue bridge would be replaced with two 10-foot-wide by six-foot-high reinforced concrete box culverts and the streambed would be lowered three feet. The private drive culvert at River Mile 2.55 would be replaced with a single 10-foot-wide by five-foot-high reinforced concrete box culvert. Upon construction of recommended wet detention basin WD8, to the west of that proposed box culvert, the drive could be used for maintenance access to the detention basin.

The streambed would be lowered about 3.2 feet at the Chicago & North Western Railway bridge, but the channel width would be limited to the bridge width and the sides would be sloped so as to avoid interference with the bridge foundation.

If, during the engineering design phase, modifications of the channel within the railroad bridge were found to present structural problems relative to the bridge foundation, the existing channel and streambed could be maintained through the bridge and a culvert could be installed through the railway embankment to connect the lowered streambed upstream and downstream of the railway. As shown on Map 20, that alternative would consist of a 75-foot-long, 48-inch-diameter reinforced concrete culvert encased in concrete with a 66-inch-diameter steel liner. Approach and exit channels would be excavated upstream and downstream of the alternative culvert.

By connecting the lowered streambeds on both sides of the railroad, the proposed culvert would provide a means of fish migration and the culvert, in combination with the existing bridge opening, would provide essentially the same hydraulic capacity as the recommended modified bridge opening alone.

Stream restoration measures recommended for this reach include the establishment of pools and riffles at the locations shown on Map 20 and the provision of riprap erosion control at culverts and locations where tributary drainage channels discharge to Lilly Creek.

Reach 9, from the Chicago & North Western Railway at River Mile 2.59 to River Mile 2.74

The channel modification in this reach is limited to that which is necessary for the streambed to return to its existing grade at River Mile 2.74. The widened and deepened channel would have a four-foot bottom width and one vertical on three horizontal side slopes. Between River Miles

2.66 and 2.74, the channel modification would be limited to the east bank to avoid disturbance of a wetland along the west bank.

It is recommended that a wetland be created in the west overbank between River Miles 2.60 and 2.69. As shown on Figure 24, the wetland inlet from the Lilly Creek channel would be set at approximate elevation 770 feet NGVD29 to enable frequent inundation of the wetland.

Reach 10, Upstream of River Mile 2.74

Under the recommended plan, the existing streambed profile and channel cross-section would be maintained from River Mile 2.74 through the upstream end of Lilly Creek. No stream enhancement measures are recommended in this reach. The four existing culverts at W. Silver Spring Drive would remain in place.

The upstream end of this reach at River Mile 3.47 is the divide between the Lilly Creek and Butler Ditch subwatershed. That divide is essentially located at the unpaved extension of El Rio Drive, which provides cover for an existing sanitary sewer. There are two north-south culverts running through the embankment. The large-scale topographic map of the area shows the low point of the embankment to be at approximate elevation 778 feet NGVD29. The Lilly Creek 100-year recurrence interval flood elevation at the divide is 779.4 feet NGVD29 and the approximate Butler Ditch 100-year flood elevation at the divide is 778 feet NGVD29. Thus, during a 100-year flood, some flow from Lilly Creek may overtop the embankment at the divide and enter Butler Ditch. If El Rio Drive is extended by the Village along the alignment of the existing embankment, it is recommended that the minimum grade be raised to an elevation above 779.4 feet NGVD29, effectively preventing the occurrence of overflow from Lilly Creek to Butler Ditch.

The Village's staff has reported stormwater drainage problems at houses in the Butler Ditch subwatershed near the Lilly Creek-Butler Ditch subwatershed divide. The source of those problems is the lack of adequate drainage outlets from those areas due to the flat slopes in the large forest/wetland complex located at the divide. Although consideration of that situation is beyond the scope of this report, it was addressed under SEWRPC Community Assistance Planning Report No. 152, A Stormwater Drainage and Flood Control System Plan for the Milwaukee Metropolitan Sewerage District, December 1990. That system plan recommends deepening of the 0.6-mile-long reach of Butler Ditch upstream of Lisbon Road in order to provide an adequate outlet for stormwater drainage from adjacent subdivisions.

Effects of Implementation of the Recommended Flood Control Plan Element in Conjunction with the Recommended Stormwater Drainage Plan Element

Full implementation of the recommended stormwater drainage and flood control plan elements would serve to eliminate flood damages due to direct overland flooding along Lilly Creek for floods up to and including the 100-year recurrence interval flood event under planned land use and channel conditions. The 100-year recurrence interval floodplain limits along Lilly Creek and its tributary streams under planned land use and existing and planned stormwater drainage and channel conditions are shown on Map 21. Some 33 residential, industrial, and commercial buildings would be removed from the 100-year floodplain and would be freed of the requirement of obtaining flood insurance when securing a mortgage for those properties. The one home remaining in the floodplain, which is recommended to be floodproofed to prevent flood damages, would not be freed of that requirement.

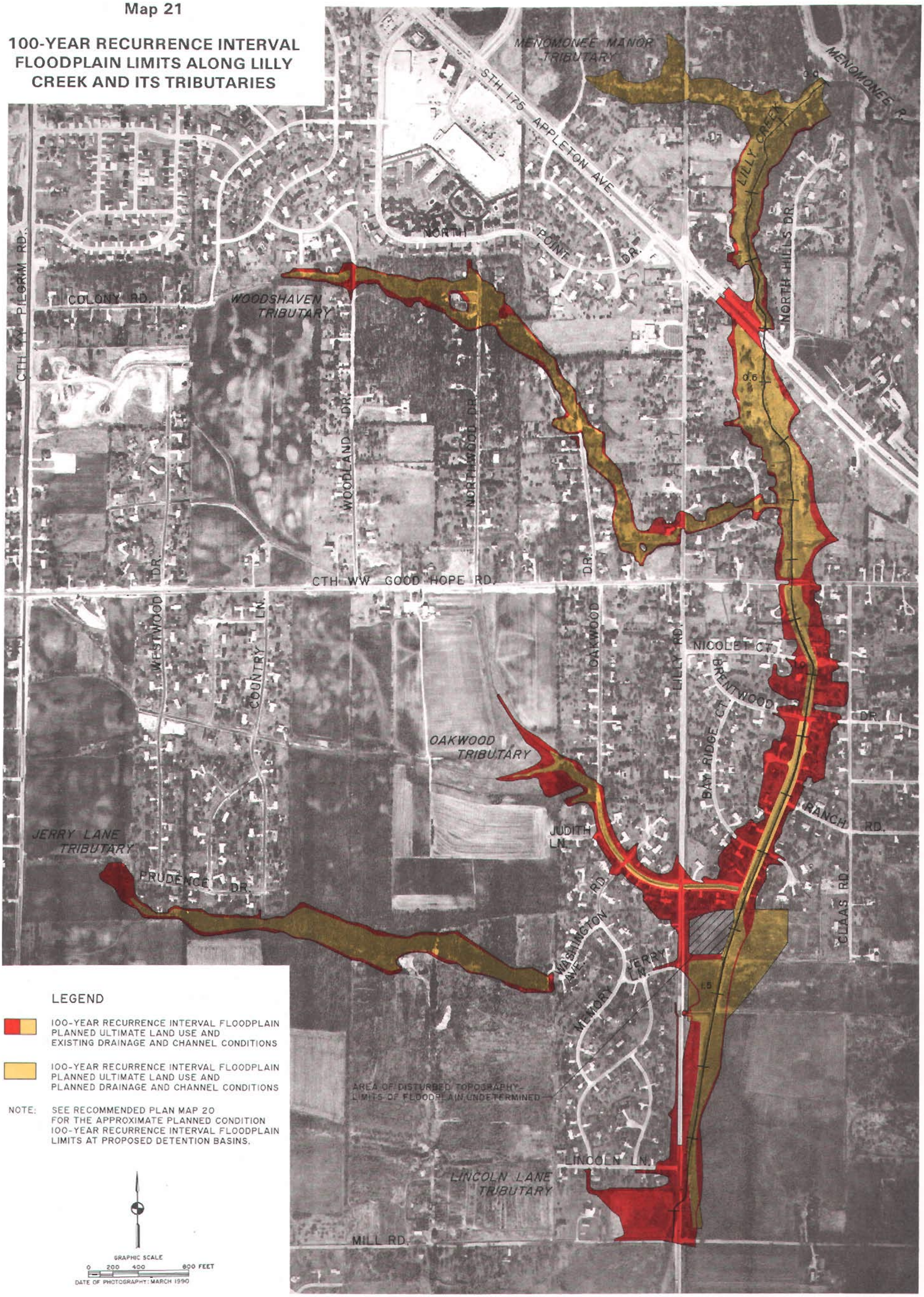
With complete implementation of the recommended flood control plan in conjunction with the recommended drainage plan, there would be no roadway overtopping during a 100-year flood under planned land use conditions at any of the bridges or culverts from W. Appleton Avenue through W. Silver Spring Drive.

With the exception of a localized area along both sides of Manor Hills Boulevard near its intersection with Manor Hill Court, flooding of streets due to overflow from Lilly Creek during a 100-year recurrence interval flood would be eliminated. The localized flooding of Manor Hills Boulevard would have a maximum depth of about 0.5 foot, assuming the approximate existing street grades are maintained when the street is relocated to accommodate the modified channel.

The recommended drainage and flood control plan would reduce Lilly Creek flood stages at Bobolink and Kaul Avenues, reducing submergence of storm sewer outfalls and facilitating more efficient drainage of lands tributary to those outfalls.

Map 21

100-YEAR RECURRENCE INTERVAL FLOODPLAIN LIMITS ALONG LILLY CREEK AND ITS TRIBUTARIES



LEGEND

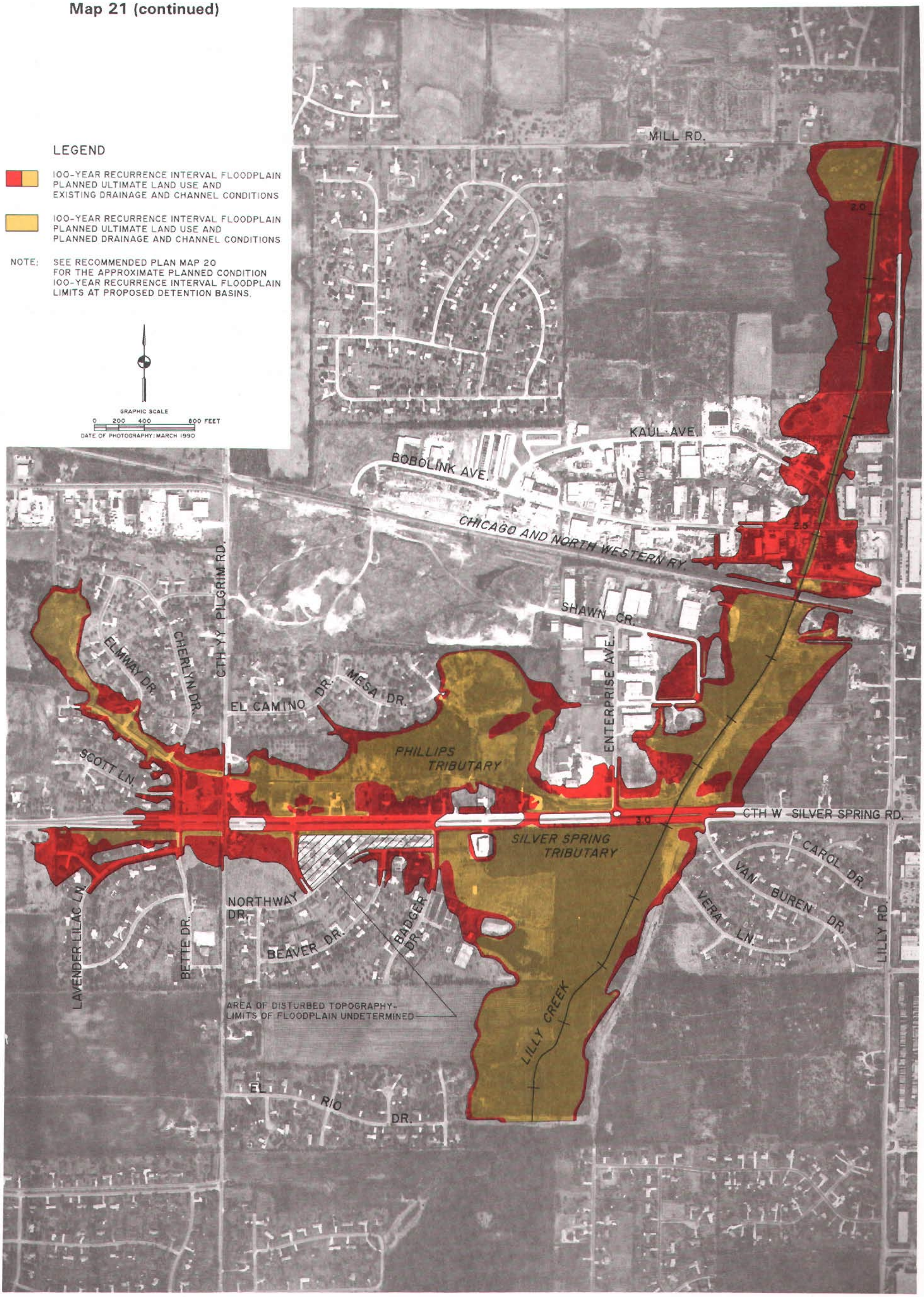
🟥🟨 100-YEAR RECURRENCE INTERVAL FLOODPLAIN PLANNED ULTIMATE LAND USE AND EXISTING DRAINAGE AND CHANNEL CONDITIONS

🟨 100-YEAR RECURRENCE INTERVAL FLOODPLAIN PLANNED ULTIMATE LAND USE AND PLANNED DRAINAGE AND CHANNEL CONDITIONS

NOTE: SEE RECOMMENDED PLAN MAP 20 FOR THE APPROXIMATE PLANNED CONDITION 100-YEAR RECURRENCE INTERVAL FLOODPLAIN LIMITS AT PROPOSED DETENTION BASINS.

DATE OF PHOTOGRAPHY: MARCH 1990

Map 21 (continued)



NOTE: SEE RECOMMENDED PLAN MAP 20 FOR THE APPROXIMATE PLANNED CONDITION 100-YEAR RECURRENCE INTERVAL FLOODPLAIN LIMITS AT PROPOSED DETENTION BASINS.

Source: SEWRPC.

Flood Control Plan Costs

The total capital cost of the recommended flood control plan is estimated to be $3,615,000. This cost includes $2,725,000 for construction of the widened and deepened channel, $415,000 for bridge removal and replacement, $250,000 for reconstruction and relocation of Manor Hills Boulevard, $25,000 for structure floodproofing, $20,000 for additional easements not already obtained by the Village, and $180,000 for stream restoration measures in reaches where the channel is to be modified. Utilizing an annual interest rate of 6 percent and a project life and amortization period of 50 years, the average annual cost of the alternative plan, including $5,000 annual operation and maintenance costs, is $234,600. The average annual flood damage abatement benefit, assuming full implementation of the refined preliminary recommended stormwater drainage plan, is estimated to be $64,700, yielding a benefit-cost ratio of 0.28. An additional cost of $158,000 is estimated for stream enhancement and recreational measures not directly related to the flood control plan.

EFFECTS OF THE FINAL RECOMMENDED STORMWATER MANAGEMENT AND FLOOD CONTROL PLAN

Hydraulic and Hydrologic Effects

The primary hydraulic and hydrologic effects of the implementation of the recommended system plan will be the safe and efficient conveyance and/or storage of runoff from all storm events up to and including the 100-year recurrence interval event and the removal of 33 residential, industrial, and commercial buildings from the 100-year recurrence interval floodplain of Lilly Creek.

As shown in Table 53, two-, 10-, and 100-year recurrence interval flood flows along Lilly Creek under planned land use, drainage, and channel conditions would generally be less than, or about equal to, flows under existing land use, drainage, and channel conditions. Because the 100-year recurrence interval flood flow at the confluence of Lilly Creek and the Menomonee River would be less under planned land use, drainage, and channel conditions than under existing land use, drainage, and channel conditions, it is not anticipated that the recommended stormwater management and flood control plan would create a significant increase in 100-year recurrence interval flood flows and stages along the Menomonee River.

In the first 0.6 mile of Lilly Creek upstream of its mouth, the peak planned condition two-year recurrence interval flood flow will be from 10 to 20 percent less than the existing condition peak two-year flow. Because the more frequently occurring floods with recurrence intervals of two years and less significantly influence the shape and alignment of the low-flow channel, the anticipated reduction in the peak two-year flood flow under recommended plan conditions should aid in the establishment of a natural channel which would be less susceptible to streambank erosion and streambed scour than the existing channel. In the unmodified reach of channel upstream of the Chicago & North Western Railway, peak two-year flood flows under recommended plan conditions would be expected to be no more than 17 percent greater than those under existing conditions. That increase in flows is much less than the possible 50 percent increase if the recommended stormwater management plan were not implemented. Thus, the recommended plan would limit changes in the morphology of the low-flow channel in the reach by controlling two-year flood flows to the greatest degree practicable.

Water Quality Effects

A primary benefit of the final recommended stormwater management and flood control plan would be improved water quality and biological conditions within Lilly Creek and its tributaries. Future loadings of suspended solids, phosphorus, and metals would be about 87, 60, and 65 percent lower, respectively, than if the plan recommendations were not implemented. Overall, the water quality of the surface waters may be expected to be better than under existing conditions, although loadings of some pollutants may be somewhat higher.

The Wisconsin Departments of Natural Resources and of Agriculture, Trade and Consumer Protection, in A Nonpoint Source Control Plan for the Menomonee River Priority Watershed Project, 1990, recommended that under planned year 2000 land use conditions suspended solids, phosphorus, and lead loadings to Lilly Creek and its tributaries be reduced to about 50 percent of the existing loadings. Such reductions would be expected to achieve the established water use objectives for the surface waters.

These high levels of pollutant loading reductions recommended within the priority watershed plan would be very difficult, and indeed, perhaps impracticable, to achieve for some pollutants within a rapidly urbanizing area such as the Lilly Creek subwatershed. However, analyses indicate that, although the final recommended plan may be expected to meet some of the priority watershed plan's pollutant loading reduction goals only partially, the desired water use objectives should be achieved.

The Lilly Creek subwatershed has been selected by the DNR as a priority watershed master monitoring site for which streamflow and biological characteristics will be monitored over a period of about 10 years. That monitoring program should provide an indication of the effectiveness of those recommended water quality management facilities which are implemented during the monitoring period.

The final recommended plan would fully meet the priority watershed plan's suspended solids loading objective. Primarily because of the high effectiveness of construction erosion controls and wet detention basins in controlling sediment losses, implementation of the plan may be expected to result in total annual suspended solids loadings which are about 73 percent lower than the existing suspended solids loadings. As a result, water clarity is expected to substantially improve, thereby enhancing the aesthetics of the water resource and increasing aquatic plant growth. With less deposition of sediment, the aquatic habitat and the suitability of the bottom substrate for fish feeding and reproduction would be expected to be significantly better than under existing conditions.

Under the final recommended plan, phosphorus loadings may be expected to be approximately 9 percent lower than under existing conditions, thereby only partially achieving the priority watershed plan's objective. However, the anticipated phosphorus levels are not expected to cause a serious water quality problem. While aquatic plant growth may be stimulated by the nutrient levels, such growth may actually be beneficial, improving the habitat conditions for fish and aquatic life. The Wisconsin DNR does not have a standard for phosphorus in its Administrative Code, although the Regional Planning Commission, in 1979,[1] recommended that the Department adopt a standard for phosphorus levels. For planning purposes, the Commission has a recommended maximum standard of 0.1 milligrams per liter, which applies only to streams recommended for full body-contact recreational uses. The Commission standard does not apply to surface waters recommended for only partial body-contact recreational uses, such as Lilly Creek or its tributaries.

Perhaps the most significant water quality concern is related to metal concentrations in the water and in the sediments. Because of the large amount of urbanization which is expected to occur within the Lilly Creek subwatershed, the anticipated loadings of lead, which was selected to be representative of metal contaminants, may be expected to be about 8 percent higher under the recommended plan than the existing lead loadings. The metals reduction objective set forth in the priority watershed plan, about 50 percent lower than the existing loadings, was based on the need to prevent the occurrence of acute toxic conditions at storm sewer outfalls, of chronic toxic conditions in the stream system, and of contaminated bottom sediments. These problems are not expected to be severe under the recommended plan. Because annual stormwater runoff volumes are expected to increase under the recommended plan by over 200 percent, the increased metals loadings would be diluted sufficiently to prevent the occurrence of toxic conditions during the vast majority of storm events. Furthermore, metals attached to sediment particles, which would be most likely to settle in the stream and contaminate the bottom substrate, also would be most readily controlled by the recommended nonpoint source abatement measures. Thus, although the metals loadings may increase somewhat, the occurrence of toxic conditions in either the water column or the bottom sediments is expected to be significantly diminished. Under the final recommended plan, it is anticipated that Lilly Creek would be able to support healthy resident populations of warmwater fish and aquatic life.

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[1] *SEWRPC Planning Report No. 30, A Regional Water Quality Management Plan for Southeastern Wisconsin: 2000, Volume Three, Recommended Plan, 1979.*

Recent research findings have indicated that copper and zinc frequently exceed toxic standards in urban runoff. The recommended plan may be less effective in reducing these metals than in reducing lead levels because a smaller portion of the copper and zinc is attached to the particulates, which could be readily removed by detention or street sweeping. Thus, additional source controls may be required to abate copper and zinc-related pollution problems in Lilly Creek fully.

Bacterial contamination of surface waters is another potential water quality problem which should be reduced by the recommended plan. Although street sweeping and construction erosion control measures are not effective in reducing bacterial loadings, wet detention ponds may provide moderate benefits, sometimes reducing bacterial levels by more than 50 percent. Source controls, such as pet waste controls, may also reduce bacterial loadings to surface waters. In addition, substantial completion of the program to retire existing septic systems and connect users to the recently constructed sanitary sewer system will also reduce bacterial levels in surface waters. It is therefore expected that implementation of the recommended plan, along with the discontinued use of septic systems and the implementation of additional source controls subsequently identified as needed, would allow achievement of the fecal coliform standards supporting partial body-contact recreational uses.

The water quality management plan element would provide numerous benefits in addition to water quality enhancement. Properly designed and managed, the 25 recommended wet detention basins would provide valuable habitats for wildlife, and, in some cases, fish. The basins would also be attractive landscape features, offering opportunities for aesthetic enjoyment and limited recreational use, such as ice skating and nature study. The recommended grassed swales and, to a lesser extent, the wet detention basins should help recharge the shallow groundwater aquifer, thereby helping to maintain the base flow of streams during dry weather periods. This increased base flow should, in turn, improve the ability of the streams to assimilate pollutant loads and generally enhance aquatic habitat conditions. The provision of extended detention storage in 15 of the recommended wet or dry detention basins would control frequently occurring flows, as well as larger floods, reducing the potential for streambank erosion and streambed scour and preserving aquatic and riparian habitat. The construction site erosion control measures, the streambank stabilization measures, and the increased street sweeping would help provide an overall cleaner and more attractive environment in the subwatershed, enriching the quality of life for its residents.

The water quality management recommendations could also have significant negative effects if the measures are not properly designed and managed. Wet detention basins must be carefully located to prevent impeding important fish migration and to avoid increasing the water temperature of ecologically sensitive headwater streams. Accumulated sediments in wet basins may contain toxic substances, especially metals. Sediment to be dredged should be tested to determine the appropriate means of disposal. The basins must also be maintained and cleaned to control the decomposition of accumulated organic matter which consumes dissolved oxygen needed to support fish and aquatic life. Proper basin maintenance can also minimize occasional aesthetic and odor nuisance problems caused by excessive macrophytes, algae, or debris. Those basins located in residential areas should also be designed to minimize safety hazards, especially to children. The recommended channel modifications to Lilly Creek and several tributaries would be a temporary source of increased sediment loadings to the streams in the subwatershed. It is recommended that the amount of the sediment loading be controlled through the enforcement of strict erosion control requirements during construction. Following construction and final restoration of all disturbed areas, potentially damaging sediment loadings would be eliminated.

COMPARISON OF RECOMMENDED PLAN TO THE WISCONSIN DEPARTMENT OF NATURAL RESOURCES NONPOINT SOURCE CONTROL PLAN

It should be noted that there are significant differences between the water quality analysis presented by the Wisconsin Departments of Natural Resources and of Agriculture, Trade and Consumer Protection in A Nonpoint Source Control Plan for the Menomonee River Priority Watershed Project, 1990, and the analysis presented in this report. In general, the priority

Table 54

COMPARISON OF THE FINAL RECOMMENDED WATER QUALITY MANAGEMENT PLAN ELEMENT TO STORMWATER DETENTION OF THE ENTIRE LILLY CREEK SUBWATERSHED

Plan	Percent Reduction in Existing Nonpoint Source Pollutant Loadings[a]			Cost		
	Suspended Solids	Phosphorus	Lead	Capital[b]	Annual Operation and Maintenance	Equivalent Annual
Final Recommended Plan	-73	-9	+8	$7,037,600	$112,000	$558,900
Stormwater Detention of Entire Subwatershed[c]	-90	-21	-9	9,240,400	180,800	767,500

[a] *The percent change refers to the change relative to the existing loading.*

[b] *Capital costs would be incurred over the 20-year planning period from 1990 through 2010.*

[c] *Detention of stormwater from essentially the entire subwatershed would require the construction of a total of 53 wet detention basins, rather than the 24 basins included in the final recommended plan. The cost estimate is based upon the assumption that suitable open land sites would be available for the additional 29 basins. However, a preliminary review indicates that at least 12 of the additional basins would need to be retrofitted in areas of existing urban development. This could require the construction of an underground storage facility or the purchase and demolition or relocation of existing buildings. Retrofitted detention facilities may be expected to entail a capital cost about 10 times higher than a detention facility located in an area of new urban development. If the entire subwatershed were tributary to detention facilities, the construction erosion control and grassed swale drainage, as presented in the recommended plan, would still be included. However, street sweeping would not be included because such sweeping would not provide benefits in an area which is tributary to a wet detention basin.*

Source: SEWRPC.

watershed plan concluded that higher reductions in existing pollutant loadings could be achieved, compared to those reductions estimated herein. These differences are related to the forecast planning periods used, and to the effectiveness of nonpoint source control measures in new urban development areas. The priority watershed plan analysis was based upon planned year 2000 land use conditions, while the analysis set forth in this report is based upon ultimate planned land use conditions. Only 75 percent of the ultimate planned new urban development is expected to occur by the year 2000. Thus, the urban runoff loadings estimated in the priority watershed plan were somewhat less than those presented in this report, and, as a result, the priority watershed plan reported that a larger reduction in existing pollutant loadings could be achieved.

With respect to the effectiveness of urban nonpoint source controls, the priority watershed plan analysis was based on the assumption that source controls, such as public education and improved "urban housekeeping" practices, would provide a significant additional reduction in metal loadings from new urban development beyond that achieved by wet detention basins and street sweeping, with the benefits of such source controls actually being quantified. While the Commission staff recognized the benefits of source controls, and indeed, while such controls are included in the recommended plan, it was concluded that the pollutant removal effectiveness of these controls could not be quantified at this time.

The priority watershed plan for the Menomonee River watershed concluded that a 50 percent reduction in existing metal loadings would be essentially achievable if stormwater runoff from all planned year 2000 new urban development and all existing commercial and industrial areas was detained in wet detention basins. In contrast, the water quality analyses set forth in this report, as summarized in Table 54, indicate that even if essentially the entire subwatershed drained to wet basins, the resulting metal load would represent only a 9 percent reduction over the existing loading. However, as noted above, the remaining metal loadings under the recommended plan are not expected to cause toxic problems in storm runoff or the stream system. Implementation of stormwater detention within essentially the entire subwatershed would entail an equivalent annual cost of about $767,500, or 37 percent higher than the annual cost of the final recommended water quality management plan element, assuming that suitable open land areas would be found for each of the necessary additional 29 wet detention basins. As indicated in Table 54, the cost would increase substantially if some of the basins had to be retrofitted into existing urban development, which could require the construction of underground storage facilities or the purchase and demolition or relocation of existing buildings.

RELATIONSHIP OF THE RECOMMENDED LILLY CREEK STORMWATER MANAGEMENT AND FLOOD CONTROL PLAN TO THE FLOOD CONTROL RECOMMENDATIONS OF THE MENOMONEE RIVER WATERSHED STUDY AND THE MILWAUKEE METROPOLITAN SEWERAGE DISTRICT STORMWATER DRAINAGE AND FLOOD CONTROL SYSTEM PLAN

Relationship to the Menomonee River Watershed Study

The Regional Planning Commission's Menomonee River watershed study, published in 1976, included flood control recommendations for Lilly Creek. Three flood control alternatives were considered in the conduct of the watershed study: floodproofing and removal of structures, locally proposed channel modifications, and bridge and culvert alteration and replacement. It was concluded in the watershed study that both the floodproofing and channel modification alternatives were technically feasible means of resolving existing and forecast flood problems along Lilly Creek. The benefit-cost ratio of the floodproofing alternative was estimated to be 1.38, as compared to a benefit-cost ratio of 0.69 for the channel modification alternative. Upon consideration of the technical, economic, and environmental issues concerned, the Regional Planning Commission staff recommended the adoption of the structure floodproofing and removal alternative. Upon further consideration of the Village of Menomonee Falls' commitment to channel modification as reflected by the location, size, and grades of existing and proposed storm sewers and storm sewer outfalls, the Menomonee River Watershed Committee opted to recommend the channel modification alternative to resolve the flooding problems along Lilly Creek.

The plan recommended by the Watershed Committee called for the construction of a concrete-lined, trapezoidal channel with a 12-foot-wide bottom and one vertical on three horizontal side slopes in the 1.55-mile-long channel reach from the confluence with the Menomonee River to Jerry Lane extended and a turf-lined, trapezoidal channel with a 20-foot-wide bottom and one vertical on three horizontal side slopes in the 1.42-mile-long reach from Jerry Lane extended to W. Silver Spring Road. The concrete lining was proposed to extend to the elevation of the 10-year recurrence interval flood. Along Manor Hills Boulevard, where the right-of-way is limited, vertical concrete retaining walls were called for along the upper portion of the modified channel. Clear-span bridges were recommended to be constructed at W. Good Hope Road, Brentwood Drive, Lilly Road, W. Mill Road, Kaul Avenue, Bobolink Avenue, and the Chicago & North Western Railway right-of-way. Two private road bridges and one pedestrian bridge between W. Mill Road and Kaul Avenue and the private road bridge between Bobolink Avenue and the Chicago & North Western Railway right-of-way were recommended to be removed and not replaced.

The Wisconsin DNR was represented on the Watershed Committee that oversaw the development of the watershed plan, and even though the Department supported the plan in Committee, the Department subsequently objected to the implementation of the recommended Lilly Creek channel improvements on environmental grounds. Thus, under this stormwater management and flood control plan the various alternatives were reconsidered and an amended channel improvement recommended. The amended improvement contains the following refinements of the original plan:

1. In general, similar modified channel shape and dimensions would be maintained under this plan as under the original watershed plan. The flood control channel

bottom width would be 12 feet, except at transitions to the existing grade, at bridges, and in reaches where additional in-channel flood storage is to be provided. Channel side slopes would average one vertical on three horizontal, as also recommended under the watershed study, except at localized transitions and in storage reaches where one vertical on four horizontal slopes would be used.

2. A small, riprap-lined low-flow channel would be provided within the flood control channel in order to restore aquatic habitat. Alternating pools and riffles would be provided in the low-flow channel.

3. The entire modified channel would be lined with natural vegetation designed to minimize channel roughness while shading the low-flow channel to improve habitat.

4. To avoid destruction of valuable aquatic habitat as identified in Appendix C of this report, channel modification would be limited to the 2.08-mile-long reach of the channel extending from a location 0.17 mile downstream of W. Good Hope Road to a location 0.15 mile upstream of the Chicago & North Western Railway. A comparison of the recommended streambed and 100-year recurrence interval flood profiles under the Menomonee River watershed study with those under this study is shown graphically on Figure 25.

5. This plan also calls for the creation of three on-channel overbank storage areas to reduce flood peaks, in contrast to the watershed study plan, which relied solely on channel conveyance of flood flows. Two of those storage areas would be wetlands constructed to enhance riparian and aquatic habitat values.

6. Because of limitations on the extent of channel modification necessary to protect valuable habitat reaches, as identified in Appendix C of this report, this plan calls for the floodproofing of one house located along one of those reaches, while the watershed study recommended no floodproofing.

The estimated peak 100-year recurrence interval flood flow at the mouth of Lilly Creek under planned land use and channel conditions as set forth in the watershed study was 2,600 cubic feet per second (cfs). Under this stormwater drainage and flood control system plan, which calls for additional on-channel and off-channel detention storage of floodwaters, the estimated peak 100-year flood flow at the mouth is about 1,900 cfs.

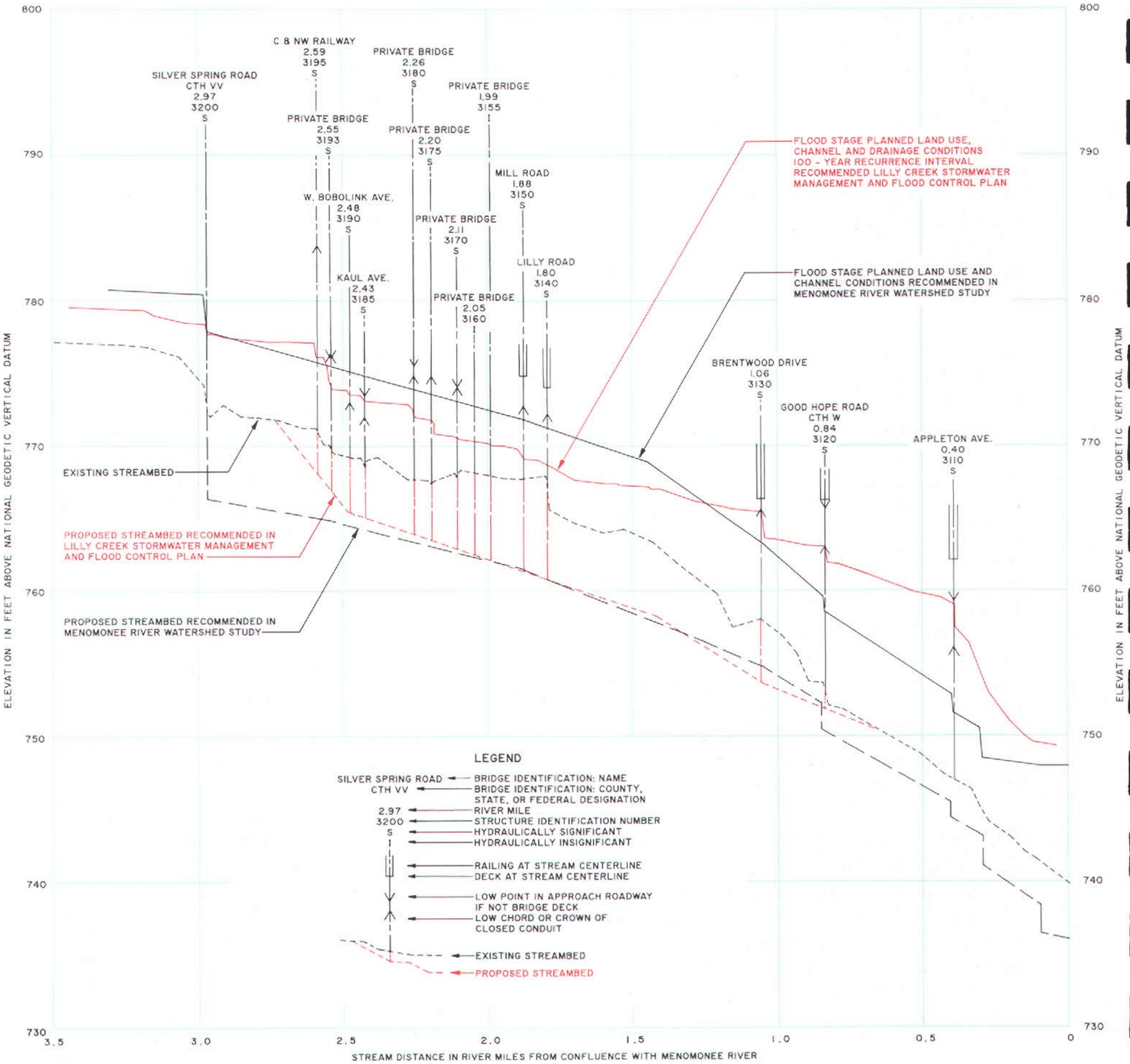
Relationship to the Stormwater Drainage and Flood Control System Plan for the Milwaukee Metropolitan Sewerage District

Because Lilly Creek did not meet the criteria for inclusion under the District's jurisdiction as set forth in the 1986 SEWRPC Community Assistance Planning Report No. 130, A Stormwater Drainage and Flood Control Policy Plan for the Milwaukee Metropolitan Sewerage District, flood control recommendations for the Creek were not made in the 1990 SEWRPC Community Assistance Planning Report No. 152, A Stormwater Drainage and Flood Control System Plan for the Milwaukee Metropolitan Sewerage District. However, the reach of the Menomonee River to which Lilly Creek discharges was studied under the District's system plan and the flood control recommendations for the Menomonee River were refined under that plan. The recommendations of the District's plan regarding the Menomonee River were reviewed to determine whether they could be affected by the recommendations contained herein for Lilly Creek.

Since the Lilly Creek plan was not completed at the time that the District's system plan was being developed, the recommended features for Lilly Creek could not be directly incorporated into the hydrologic model used under the District's study. That hydrologic-hydraulic simulation model was originally developed under the Menomonee River watershed study and was refined under the District's study. The Lilly Creek recommendations were, however, approximated under the District's study, wherein the 100-year peak flood flow at the mouth under planned land use conditions was estimated to be 1,400 cfs. The 100-year peak flood flow for the Menomonee River in the reach downstream of its confluence with Lilly Creek was estimated to be 3,300 cfs under the District's study. As set forth in Table 53 of this chapter, the Lilly Creek 100-year peak flood flow under planned land use, drainage, and channel conditions is estimated to be 1,900 cfs.

Figure 25

COMPARISON OF RECOMMENDED STREAMBED AND FLOOD PROFILES: 1976 MENOMONEE RIVER WATERSHED STUDY AND 1992 LILLY CREEK STORMWATER MANAGEMENT AND FLOOD CONTROL PLAN



Source: SEWRPC.

The difference in 100-year flood flows for planned land use and channel conditions between the District's study and this stormwater management and flood control plan would not be expected to have a significant impact on the conclusions and recommendations made under the District's study of the Menomonee River for the following reasons:

1. Flood flows were estimated under the District's study using the Hydrocomp HSP-X continuous simulation hydrologic model, which is well suited to performing hydrologic analyses at the level of detail and watershed discretization required to analyze relatively large watersheds such as that of the Menomonee River. That model was calibrated to historic flood events, 49 years of historical climatological data were used to develop annual peak flood flows, and those flows were subjected to statistical analysis to determine flood frequencies. With this type of hydrologic model, differences in flood flows which may occur at the subbasin or subwatershed level are often insignificant when considered at the watershed level. The plan presented here for the Lilly Creek subwatershed is based on flood flows developed using the U. S. Army Corps of Engineers HEC-1 flood hydrograph package. As applied in this study, the model used design storms based on theoretical rainfall distributions and rainfall-intensity-duration data developed by the Commission from observed storms in the Region. The HEC-1 model utilizing a design storm concept is well suited to application for a detailed stormwater management plan because it is consistent with commonly accepted and widely applied stormwater drainage design techniques at the project design level and because it is more readily applied at the greater level of detail required for stormwater drainage analysis than is the HSP-X model. Because of the differing levels of detail employed to execute the two models, the difference in flows computed by the two approaches is considered to be within a reasonable range consistent with the objectives of each model and study.

2. The increased peak flood flows on Lilly Creek, from the 1,400 cfs estimated under the District's flood control planning work to the 1,900 cfs estimated under the modified plan for the Lilly Creek subwatershed, would not be expected to directly affect the peak flood flows on the Menomonee River main stem downstream from the confluence with Lilly Creek because of differences in the timing of the peaks on the two stream systems. During two of the largest flood events of record, one on March 17 and 18, 1964, and one on September 10 and 11, 1986, the peak flow at the mouth of Lilly Creek occurred about one hour prior to the occurrence of the peak flow on the Menomonee River main stem at the confluence with Lilly Creek. Thus, the flood flows on Lilly Creek may be expected to have begun to abate before the flows peak on the Menomonee River.

3. Modest increases in flood flows on the Menomonee River downstream of Lilly Creek would not, in any case, result in significant changes in the horizontal extent of the floodplain in that area. For example, an increase in the 100-year flood flow from 3,300 to 3,800 cfs, or 16 percent, in the reach of the Menomonee River immediately downstream of Lilly Creek would result in an increase in stage of 0.6 foot. Such an increase would not significantly change the horizontal extent of the floodplain in that downstream reach.

4. The 12-mile-long reach of the Menomonee River downstream of its confluence with Lilly Creek is a reach of low flood damage potential under planned land use and existing channel conditions. The District's study recommends floodproofing of several buildings located along this reach, but no channel modifications. The closest reach where flooding-related problems occur on the Menomonee River is four miles downstream of the confluence with Lilly Creek. There are several major tributaries flowing into the Menomonee River mainstem in the 12-mile reach and the tributary area and peak 100-year flood flows more than double in the reach. Thus, the influence of changes in Lilly Creek flood flows on flood flows along the mainstem of the Menomonee River may be expected to be relatively localized.

On the basis of these factors, it may be concluded that the difference in the Lilly Creek peak 100-year flood flow as estimated under the District's flood control study and under the Lilly Creek stormwater management and flood control study is not significant enough to alter any of the conclusions or flood control recommendations set forth in the District's study.

REVIEW OF PLAN COMPONENTS FOR COMPLIANCE WITH CHAPTER NR 103 OF THE WISCONSIN ADMINISTRATIVE CODE

Chapter NR 103 of the Wisconsin Administrative Code, effective August 1, 1991, establishes water quality standards for wetlands. The rules set forth in Chapter NR 103 consist of two parts: 1) a set of standards intended to protect water quality-related functions of wetlands including sediment and pollution control, stormwater and floodwater storage, hydrologic cycle maintenance, shoreline erosion protection, habitat protection for aquatic organisms and other wildlife species, and recreational uses and 2) implementation procedures for application of the water quality standards. The Wisconsin DNR is responsible for the review of proposed projects for compliance with Chapter NR 103.

The plan set forth in this report is intended to meet the multiple objectives of controlling nonpoint source pollution, protecting primary environmental corridors and wetlands, and providing adequate stormwater drainage and flood control facilities to meet the needs of existing and new development. Those objectives are generally consistent with the intent of the standards set forth in Chapter NR 103; however, fully meeting each of the objectives may not be possible in all instances because the objectives may conflict. In such cases, it may be most desirable for a certain objective to only be partially met in order to insure that other equally important objectives can be fully met.

In general, the recommendations of this stormwater management plan are intended to preserve or enhance the quality of receiving streams and wetlands wherever practicable through the control of frequently occurring flows and through the control of nonpoint source pollution. In some instances, the provisions of such controls may involve locating a stormwater management facility in a wetland. In those cases, the proposed facility must be evaluated for conformance with the requirements of Chapter NR 103. Map 22 shows the shoreland and nonshoreland wetlands in the subwatershed.

A project would not be in compliance with the provisions of Chapter NR 103 if it is not surface water- or wetland-dependent, meaning that it does not necessarily require "location in or adjacent to surface waters or wetlands to fulfill its basic purpose," and if a practicable alternative to the project exists.[2] Under a practicable alternatives analysis, the proposed project would be compared to the practicable alternatives considering relative monetary costs, logistical limitations, technological limitations, and other pertinent positive or negative aspects of the alternatives. If there is an alternative to the project which is practicable, will not adversely impact wetlands, and will not have other significant adverse environmental consequences, the alternative would be selected.

If, following the practicable alternatives analysis, no suitable alternative is identified, an assessment of the impacts of the project on the functional values of the wetland must be made. That assessment should provide details of the impacts to the wetland relative to the categories set forth in the standards and listed above. Those impacts would then be considered by the Department in making a determination that the requirements of Chapter NR 103 are satisfied.

The detailed permit application procedure set forth above would be initiated following the planning stage at such time that a given project is to be implemented. For the purposes of the stormwater management plan documented in this report, a practicable alternatives analysis was provided in Appendix D in each instance where a component of the recommended plan could result in wetland disturbance. If the analysis indicated that an alternative to the component included in the preliminary recommendation could be provided without significantly compromising the overall plan objectives, that alternative was selected. If no such alternative were judged to be practicable, the preliminary recommendation was maintained and a general assessment of the impact of the recommendation on the functional values of the wetland was made. That assessment was based in part on determinations by Commission's staff

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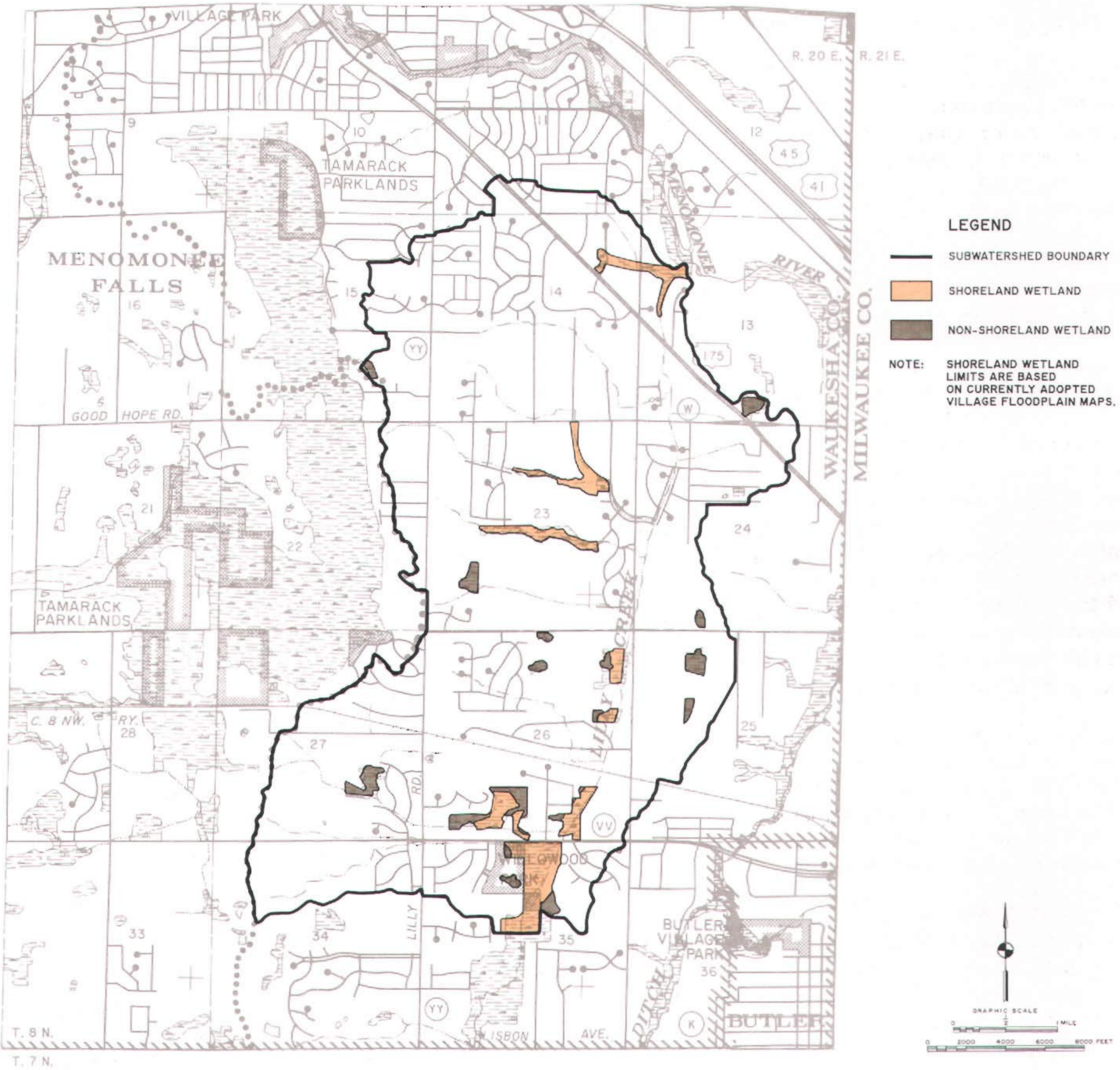
[2] *The staff of the Department of Natural Resources have determined that wet detention basins for control of nonpoint source pollution are not surface water- or wetland-dependent and would, therefore, not be in compliance with Chapter NR 103 if practicable alternatives exist which "will not adversely impact wetlands and will not result in other significant adverse environmental consequences."*

Map 22

WETLANDS IN THE LILLY CREEK SUBWATERSHED



Source: SEWRPC.

biologists of the existing functional value of each affected wetland and the potential for enhancement or degradation of the wetland.

AUXILIARY PLAN RECOMMENDATIONS

The foregoing recommendations address primarily stormwater drainage system improvements, water quality management measures, flood control, and stream restoration and enhancement. To provide a comprehensive stormwater management and flood control plan, however, these recommendations must be supplemented by plan elements relating to natural resource and open space protection, and by the continued proper maintenance of the stormwater management system.

Natural Resource and Open Space Preservation

A land use plan has been prepared and adopted by the Village that provides for the preservation of the primary environmental corridor lands within the Village and environs, including associated floodlands and wetlands, in essentially natural, open uses.[3] The protection of floodlands and wetlands from intrusion by urban land uses has important implications for stormwater management since these lands can provide needed capacity for the storage, infiltration, and transport of stormwater runoff.

Floodplain Map Revisions

It is recommended that the Village amend its floodplain zoning ordinance and request revision of the Federal Emergency Management Agency Flood Hazard Boundary Maps by the Federal Insurance Administration in two steps.

1. Immediately upon adoption of this system plan, the Village should amend those portions of its floodplain zoning ordinance pertaining to Lilly Creek to reflect the 100-year recurrence interval water surface profiles set forth in this plan for the existing channel and drainage system under planned ultimate land use conditions. At that time, the Village should also submit its proposed floodplain revisions to the Wisconsin DNR requesting revision of the Flood Hazard Boundary Maps by the Federal Insurance Administration.
2. As the drainage and flood control improvements herein recommended are constructed and become operational, the Village should again amend its floodplain zoning ordinance accordingly and request revision of the Flood Hazard Boundary Maps. Numerous citizens whose homes can be removed from the floodplain would thereby benefit from decreased insurance costs.

Maintenance of Stormwater Management Facilities

The effectiveness of the stormwater conveyance and detention facilities, once developed, can be sustained only if proper operation, repair, and maintenance procedures are carefully followed. The Village has a program of sewer, culvert, catch basin, and channel cleaning; sewer inspection; street sweeping; leaf collection; and minor repair work on sewers, manholes, catch basins, and inlets. Important additional maintenance activities include the periodic clearing of sewer obstructions, maintenance of open-channel vegetative lining, maintenance of detention facility inlets and outlets, maintenance of detention basin vegetative cover, periodic removal of sediment accumulated in detention basins, and sweeping of streets in commercial and industrial areas not served by wet detention basins. These maintenance activities are recommended to be carried out on a continuous basis to maximize the effectiveness of the stormwater management facilities and measures, and to protect the capital investment in the facilities. Cost estimates of the recommended maintenance activities are included in the total plan costs.

In the past, the Village has entered into legal agreements for the maintenance of detention basins constructed to control runoff from new development under the Village's stormwater detention guidelines as set forth in Chapter II of this report. Those agreements have covered situations where the detention basin lies within easements on individual lots in a subdivision, when the basin lies within outlots owned by a homeowners' association, or when the Village owns part of a basin with the remainder lying within an easement on a lot within a subdivision. The agreements in these cases call for the individual property owner or homeowners'

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[3] *SEWRPC Community Assistance Planning Report No. 162, A Land Use and Transportation System Plan for the Village of Menomonee Falls: 2010, April 1990.*

Table 55

COSTS OF THE RECOMMENDED STORMWATER MANAGEMENT AND FLOOD CONTROL PLAN FOR THE LILLY CREEK SUBWATERSHED

Plan Element	Capital	Annual Operation and Maintenance	Equivalent Annual Cost[a]
Stormwater Drainage System	$ 8,505,000	$ 61,800	$ 601,900
Water Quality Management Measures[b]	7,037,600	112,000	558,900
Flood Control and Stream Restoration Measures[c]	3,615,000	5,000	234,600
Total	$19,157,600	$178,800	$1,395,400

[a] *Equivalent annual cost computations assume a 50-year life and 6 percent annual interest.*

[b] *This amount includes $2,742,000 for construction erosion control as required by village ordinance. That amount would be spent over the 20-year planning period from 1990 through 2010.*

[c] *An additional expenditure of $158,300 is recommended for recreational measures and for stream habitat enhancement in reaches of Lilly Creek where no channel modification is recommended.*

Source: SEWRPC.

association to maintain the design elevations and slopes within the detention basins, to construct no structures within the detention basin, and to obtain the approval of the Village's staff for all trees and shrubs planted within the basin. When a homeowners' association was established, that association was responsible for maintenance and repairs to the basin and the Village was empowered to recover the costs of maintenance and repairs not performed by the association through special assessments against each subdivision lot owner.

It is recommended that the detention basins called for under the system plan set forth in this report be owned and maintained by the Village. Such an arrangement is desirable because the basins are generally centrally located to receive runoff from several different existing and/or planned upstream developments. The Village may establish procedures whereby the capital cost of construction of the basin is incurred by the Village and then, as development proceeds, the costs are recovered through charges to individual property owners or developers in the tributary area in proportion to their runoff contribution. In cases where a single development is the primary source of increased runoff to the basin, the cost of the basin may be paid by the developer during construction of the subdivision improvements.

STORMWATER MANAGEMENT SYSTEM COSTS

The capital costs and the operation and maintenance costs of the recommended stormwater management and flood control system plan are presented in Table 55. The capital cost of the recommended plan is estimated to be $19.2 million. The annual operation and maintenance cost increase of the recommended plan is estimated to be $178,800, or $31,600 per square mile for the 5.65-square-mile Lilly Creek subwatershed. Of the total capital cost of the recommended plan, about $8.5 million, or 44 percent, is for the stormwater drainage plan element; about $7.1 million, or 37 percent, is for the water quality management plan element; and the remaining $3.6 million, or 19 percent, is for the flood control plan element. Of the total annual operation and maintenance cost, about $61,800, or 34 percent, is for the stormwater drainage plan element; about $112,000, or 63 percent, is for the water quality management plan element; and $5,000, or 3 percent, is for the flood control plan element.

These costs are based upon planned ultimate development of the Lilly Creek subwatershed and do not include the cost of minimum-diameter collector sewers, roadside swale collectors, and road culverts that may be required to drain collector and land access roadways, the alignment of which has not as yet been determined, or the cost of roadway sections in newly developing areas that have been designated to function as a component of the major drainage system.

SUMMARY

The recommended stormwater management and flood control plan is shown in graphic form on Map 20. The recommended plan consists of three elements: 1) a stormwater drainage plan, which utilizes centralized detention storage, open channels and roadside swales, and storm sewers, and which attempts to preserve to the greatest extent possible the existing system of streams tributary to Lilly Creek, 2) a nonpoint source control plan, which calls for the construction of wet detention basins, the use of grassed swales in areas of new suburban and low-density residential development, accelerated street sweeping in industrial and commercial areas which are not tributary to wet detention basins, and the continued enforcement of the Village's construction erosion control ordinance, and 3) a flood control plan, which calls for the widening and deepening of 2.08 miles of the Lilly Creek channel, the replacement and/or removal of certain bridges and culverts along Lilly Creek, the floodproofing of one house, the creation of two wetland/overbank storage areas and one dry overbank storage area, and the implementation of stream restoration and enhancement measures to improve aquatic and terrestrial habitat conditions along the stream corridor. The estimated cost of the recommended stormwater management and flood control plan is set forth in Table 55.

Chapter VII

PLAN IMPLEMENTATION

INTRODUCTION

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

The first section of this chapter describes the relationship of land use development and redevelopment to the effectiveness of the planned stormwater management measures. The second section discusses the importance to implementation of the plan of more detailed engineering. The third section sets forth the specific actions required to implement the plan. A preliminary plan implementation schedule is presented in the fourth section. The fifth section presents regulatory considerations. The sixth section discusses the need for periodic reevaluation and updating of the plan itself.

RELATION TO FUTURE LAND USE DEVELOPMENT

Coordination with land use development and redevelopment is fundamental to successful implementation of a sound stormwater management plan. Ultimate planned land use conditions for the Lilly Creek subwatershed were presented in Chapter II of this report. To a large extent, the effectiveness of the recommended stormwater management measures will depend upon the degree to which future land use development and redevelopment conform to the planned land use pattern and the degree to which the land use and stormwater management plans properly complement each other.

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

RELATION OF DETAILED ENGINEERING DESIGN TO SYSTEM PLANNING

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

Chapter IV of this report presented the engineering design criteria and analytic procedures used in the preparation and evaluation of the alternative stormwater management and flood control system plans. These criteria and procedures, firmly based in current engineering practice, provided the means for quantitatively sizing and for quantitatively analyzing the performance of both the minor and major stormwater drainage system components and the flood control components. These criteria and procedures should also serve as a basis for the more detailed design of stormwater management and flood control system components in the implementation of the recommended plan. It is important that such criteria and procedures be applied uniformly and consistently in all phases of implementation of the plan if the resulting system is to perform as envisioned in the plan.

Table 56 sets forth the design criteria and analytic procedures recommended to be followed in the engineering design of the recommended plan components. Criteria and procedures are presented in the table for estimating stormwater flows; calculating hydraulic capacities of conveyance facilities; designing street cross-sections and related site grading; locating and designing storm sewer inlets; designing storm sewers; designing roadside swales, open channels, and culverts; and designing storage facilities. In this respect, it is recognized that over time new design techniques may be developed and become available for use in the design of stormwater management and flood control system components. Such techniques should, however, be carefully reviewed for consistency with the criteria and procedures set forth in the recommended system plan before they are adopted.

PLAN IMPLEMENTATION

Plan Adoption

An important first step in plan implementation is the formal adoption of the recommended stormwater management and flood control plan, as documented herein, by the Village Stormwater Management and Flood Control Advisory Committee, the Village Plan Commission, the Transportation and General Government Committees of the Village Board, and the Village Board. In addition, the plan should be endorsed by the Wisconsin Department of Natural Resources (DNR).

Upon such adoption, the stormwater management and flood control plan becomes the official guide for officials of the Village in making stormwater management decisions. Such formal adoption serves to signify agreement with, and official support of, the recommendations contained in the plan and enables the Village's staff to begin integrating the plan recommendations into the ongoing land use control, public works development planning and programming, and subdivision plat review processes of the Village.

Implementation Procedures

It is recommended that the plan be implemented, over time, using the Village's existing procedures for land subdivision plat and certified survey map approval, capital improvement programming, and public works construction and operation and maintenance. Implementation of the plan through a stormwater utility was considered and rejected. The administration of the stormwater management and flood control program through a utility would duplicate an administrative and review function already performed satisfactorily by the Village's staff and committees. Also, the time required to establish the utility and to resolve possible problems regarding the legal authority for such a utility could unduly delay implementation of the stormwater management plan.

In reviewing subdivision plats and certified survey maps, the Village Plan Commission should make a determination that the plats or maps are consistent with the land use plan utilized in the design of the recommended stormwater management and flood control plan. Any proposed departures from that land use plan should be carefully considered in light of the stormwater management needs of the proposed development and the impacts on upstream and downstream areas. If the proposed departures are permitted, the developers should be required to pay for the incremental costs entailed by modifying the stormwater management and flood control systems.

Capital improvements programming would be a particularly important tool for implementing the recommended stormwater management and flood control plan. Typically, a capital improvements program is a five-year program for timing and financing a priority list of capital improvement projects. Such a program is based upon the projected financial capability of the community and is formulated from an analysis of planned capital improvement costs and of municipal revenues, debt service obligations, financing procedures, and external funding potentials. Once formulated, the program should be reevaluated, refined, and extended on an annual basis. Under this option, the Village's well-developed procedure for capital improvement financing

Table 56

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

Design Function	Recommended Criteria and Procedures
Storm Runoff Flows	Minor system components should be designed to accommodate flows expected from a 10-year recurrence interval storm event. Major system components should be designed to accommodate flows expected from a 100-year recurrence interval storm event. To determine peak rates of flow for the design of pure conveyance facilities with no significant upstream storage, the Rational Method, as described in SEWRPC Technical Record, Vol. 2, No. 4, April-May 1965, "Determination of Runoff for Urban Stormwater Drainage System Design," or the U.S. Soil Conservation Service Method, as described in SCS Technical Release 55, June 1986, "Urban Hydrology for Small Watersheds," should be used. The rainfall intensity, duration, and frequency curves suitable for use with the Rational Method are provided in Figure 9 in Chapter IV of this report. When storage is to be included in the facilities and estimates of runoff volumes as well as peak rates of discharge are required, the TR55 Method for sizing detention basins or a suitable hydrologic-hydraulic simulation model should be used
Conveyance Facilities	The sizes of recommended conveyance facilities are set forth in Table 49 and on Map 20 of Chapter VI of this report. Manning's formula should be used to determine the hydraulic capacities of conveyance facilities where flow conditions approximate uniform conditions. The use of Kutter's formula is also acceptable for uniform pipe flow computations. Storm sewers should be designed to flow full during the design storm event. Flow velocities should not be less than 2.5 feet per second in storm sewers. The chart set forth in Figure 16, Chapter IV, of this report should be used to determine the hydraulic elements of the storm sewers. Manning's "n" values for roadside swales should be selected using retardance levels C or D, as shown in Figure 13 of Chapter IV. Flow velocities should not exceed six feet per second in turf-lined channels. Where pipe flow does not approach uniform conditions, backwater, drawdown, or inlet control conditions should be determined mathematically or by use of appropriate nomographs. Where open-channel flow does not approach uniform conditions, as along Lilly Creek and segments of its tributaries, the U. S. Army Corps of Engineers HEC-2 model or another comparable model should be used to compute water surface profiles
Street Cross-Sections and Related Site Grading	Except in areas specifically recommended to have rural cross-sections, streets should be designed with urban cross-sections. Typical street cross-sections are shown in Figure 2 of Chapter III. Slopes away from all buildings, as well as the slopes of interior drainage swales, should be at least one-quarter inch per foot to provide positive drainage
Storm Sewer Inlets	Storm sewer inlet location and capacity should be dictated by the allowable stormwater spread and depth of flow in streets. Combination inlets should be used in most instances. Uncontrolled flow across streets should not be allowed when the streets are functioning as a part of the minor stormwater drainage system. At locations where storm sewers function as a part of the major drainage system and are sized to convey design flows resulting from storms with recurrence intervals greater than 10 years, and at locations where a storm sewer is intended to divert a specific design flow to an offline detention basin, sufficient inlet hydraulic capacity should be provided to permit the design capacity of the storm sewer to be developed
Culverts	The length and size of recommended culverts are set forth in Table 49 and on Map 20. Culvert capacities should be determined by using appropriate nomographs and charts or by using the HEC-2 model or a comparable substitute where the culvert is a component of an open-channel system along Lilly Creek or its tributaries. Where appropriate, culverts should be designed to permit fish passage
Storage Facilities	The size and design outflows of recommended storage facilities are set forth in Tables 49 and 50 of Chapter VI. The effects of storage facilities on the frequency, duration, and magnitude of downstream flows under future conditions as compared to existing conditions should be carefully examined

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

Source: SEWRPC.

should incorporate the stormwater management and flood control plan components in a manner consistent with the construction prioritization set forth below.

Implementation of the plan through the Village's zoning ordinance provides another important means of ensuring that land use development takes place in accordance with the land use assumptions underlying the stormwater management and flood control plan. Unlike subdivision control, which operates on a plat-by-plat basis, the zoning ordinance operates over the entire Village in advance of development proposals, serving to increase public acceptance of the plan recommendations and improving coordination between upstream development and downstream stormwater management and flood control. As in the case of subdivision plat review, any zoning changes should consider the potential impacts on the facilities included in the stormwater management and flood control plan.

A common stormwater management problem facing municipalities is a lack of a continuing maintenance program for stormwater facilities, including periodic inspection and routine preventive maintenance. This problem is commonly caused by the absence of an assured source of funding and incomplete records to justify budgeting for such funding. Stormwater and flood control facility maintenance can be easily deferred for a limited period of time, and many public officials and citizens alike incorrectly perceive that certain components, such as open channels or sewers, are self-maintaining, or that no hazards will result if such facilities are allowed to deteriorate. A sound, continuing, preventive maintenance program should be given a high priority for funding, particularly for a stormwater management and flood control system which includes various types of components, such as storm sewers, roadside swales, culverts, open channels, and onsite and centralized detention facilities which are interrelated and interconnected.

The Village does have a maintenance program for drainage facilities. It is recommended that the public works program of the Village continue to provide for the maintenance, as well as construction, of the needed stormwater management and flood control facilities, including periodic inspection of conveyance and detention facilities; timely repair of facilities; cleaning of storm sewers, open channels, and detention facility inlets and outlets; maintenance of open-channel and detention facility lining materials; and periodic removal of accumulated sediment from conveyance, detention, and sediment control facilities.

Financing

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

In addition to using current tax revenue sources such as property taxes, the Village may make use of such revenue sources as reserve funds, general obligation bonds, private developer contributions, and state grants.

The creation of a tax incremental financing district is another financing option available to the Village. When such a district is created, a "tax incremental base" is established; this base is the aggregate value of all taxable property in the district as of the date of creation as equalized by the Wisconsin Department of Revenue. Any subsequent growth in the tax incremental district base is then "captured," so that as property value increases, levies on this growth represent positive dollar increments used for financing development. The effect of the tax incremental law, then, is to delay the availability to general government of the revenues which result from the increase in values due to improvements in the tax incremental district until the public costs entailed in generating the development have been paid for. Tax incremental financing could be used to finance some of the recommended stormwater management and flood control system components. The Village has used this method to finance other public works projects.

Other than Wisconsin DNR nonpoint source pollution abatement program funds, state and federal grants are generally not available to finance stormwater management measures at this time. The Village may be able to obtain financial assistance from the Wisconsin Fund

Nonpoint Source Pollution Abatement Program, administered by the Wisconsin DNR, for construction of components of the water quality management plan element. Some state funds may also be available for recommended Lilly Creek stream enhancement measures beyond reaches for which channel modifications are recommended. It is also possible that the cost of certain components of the recommended stormwater drainage and flood control system could be shared between the Village and the Wisconsin Department of Transportation (DOT) as a part of highway construction or reconstruction projects.

To provide a dependable source of funds necessary to meet the operation and maintenance costs of implementation of the recommended plan, such costs should be funded out of the Village's general fund as part of the ongoing public works program. For new urban developments which encompass recommended stormwater management measures to be financed in whole or in part by the private sector, provision of the recommended facilities would ordinarily be a condition of plat approval imposed by the Village. Thus, the costs would ultimately be borne at least in part by the lot parcel purchasers. Needed stormwater management and flood control facilities may also be funded through the levy of special assessments of the costs entailed against the benefitted properties. Such special assessments, like direct municipal funding, may involve the use of bonding.

PRELIMINARY SCHEDULE FOR IMPLEMENTATION AND FINANCING

Schedule of Public-Sector and Private-Sector Costs

In general, the capital costs of each stormwater management or flood control component were assumed to be borne by the public sector if the components were designed to serve public property or if the general public, not just the owners of new development, would benefit from the component. Capital costs were assumed to be borne by the private sector if the primary benefit of the component would accrue to new development. Public-sector and private-sector expenditure requirements are listed in Table 57. The following criteria were applied to allocate capital costs to the public sector and private sector:

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

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

3. Recommended plan components intended to serve specific, new, private urban development which must be oversized to provide capacity for additional planned future or existing upstream urban development were assumed to be funded by both the public sector and the private sector. The private sector was assumed to finance the costs of serving the specific new urban development; the public sector was assumed to finance the cost of the oversizing required to serve the additional upstream urban development.

4. Consistent with the Village's current policy, component capital costs were assigned to the private sector for specific private urban development which was either underway or for which development plans had been submitted to the Village during the time this system plan was being prepared.

5. The capital costs of the recommended street sweeping were assigned to the public sector.

6. All channel modifications and culvert replacements for flood control purposes were assumed to be funded by the public sector.

7. All floodproofing measures were assumed to be funded by the private sector.

Funds may be available from the State of Wisconsin for the installation of best management practices which meet the nonpoint source pollution reduction objectives set forth in the Menomonee River Priority Watershed Study. The current policy of the Wisconsin DNR regarding the provision of funding for nonpoint source pollution control measures undertaken by local units of government provides for state funding of up to 70 percent of the capital cost of wet

Table 57

RECOMMENDED ASSIGNMENT OF PUBLIC-SECTOR AND PRIVATE-SECTOR COSTS FOR COMPONENTS OF THE RECOMMENDED LILLY CREEK STORMWATER MANAGEMENT AND FLOOD CONTROL PLAN

Hydrologic Unit Designation	Component Designation	Public Sector		Private Sector		Total	
		Capital^a	Operation and Maintenance	Capital^a	Operation and Maintenance	Capital^a	Operation and Maintenance
Stormwater Drainage Plan Element (refer to Table 49)							
A	1	$ - -	$ 200	$ 16,000	$ - -	$ 16,000	$ 200
	2	- -	100	9,000	- -	9,000	100
	3	- -	600	82,000	- -	82,000	600
	4	- -	300	48,000	- -	48,000	300
	5	- -	800	123,000	- -	123,000	800
	6	- -	100	21,000	- -	21,000	100
	7	- -	200	100,000	- -	100,000	200
	8	- -	200	123,000	- -	123,000	200
	9	- -	100	82,000	- -	82,000	100
	10	- -	0	35,000	- -	35,000	0
	11	52,00	1,000	- -	- -	52,000	1,000
	12	65,000	0	- -	- -	65,000	0
	13	23,000	0	- -	- -	23,000	0
	14	1,000	0	- -	- -	1,000	0
	15	- -	3,000	172,000	- -	172,000	3,000
Subtotal		$ 141,000	$ 6,600	$ 811,000	$ - -	$ 952,000	$ 6,600
B	1	$ 34,000	$ 300	$ - -	$ - -	$ 34,000	$ 300
	2	23,000	200	- -	- -	23,000	200
	3	- -	200	35,000	- -	35,000	200
	4	- -	200	40,000	- -	40,000	200
	5	- -	300	52,000	- -	52,000	300
	6	- -	100	75,000	- -	75,000	100
	7	29,000	200	- -	- -	29,000	200
	8	- -	0	10,000	- -	10,000	0
	9	66,000	0	- -	- -	66,000	0
	10	- -	3,100	150,000	- -	150,000	3,100
	11	83,000	900	- -	- -	83,000	900
	12	176,000	4,200	- -	- -	176,000	4,200
Subtotal		$ 411,000	$ 9,700	$ 362,000	$ - -	$ 773,000	$ 9,700
C	1	$ - -	$ 200	$ 25,000	$ - -	$ 25,000	$ 200
	2	- -	200	20,000	- -	20,000	200
	3	129,000	700	- -	- -	129,000	700
	4	133,000	600	- -	- -	133,000	600
	5	124,000	200	- -	- -	124,000	200
	6	43,000	100	- -	- -	43,000	100
	7	34,000	0	- -	- -	34,000	0
	8	- -	0	89,000	- -	89,000	0
	9	- -	0	37,000	- -	37,000	0
	10	62,000	100	- -	- -	62,000	100
	11	134,000	100	- -	- -	134,000	100
	12	150,000	200	- -	- -	150,000	200
	13	- -	200	6,000	- -	6,000	200
	14	- -	1,000	135,000	- -	135,000	1,000
	15	- -	300	14,000	- -	14,000	300
Subtotal		$ 809,000	$ 3,900	$ 326,000	$ - -	$ 1,135,000	$ 3,900
D	1	$ - -	$ 500	$ 50,000	$ - -	$ 50,000	$ 500
	2	22,000	300	21,000	- -	43,000	300
	3	27,000	600	62,000	- -	89,000	600
	4	- -	500	79,000	- -	79,000	500
	5	- -	100	20,000	- -	20,000	100
	6	26,000	100	- -	- -	26,000	100
	7	20,000	400	45,000	- -	65,000	400

Table 57 (continued)

Hydrologic Unit Designation	Component Designation	Public Sector		Private Sector		Total	
		Capital[a]	Operation and Maintenance	Capital[a]	Operation and Maintenance	Capital[a]	Operation and Maintenance
D (continued)	8	$ - -	$ 100	$ 69,000	$ - -	$ 69,000	$ 100
	9	332,000	500	- -	- -	332,000	500
	10	119,000	200	- -	- -	119,000	200
	11	- -	100	56,000	- -	56,000	100
	12	- -	100	73,000	- -	73,000	100
	13	- -	2,000	98,000	- -	98,000	2,000
Subtotal		$ 546,000	$ 5,500	$ 573,000	$ - -	$ 1,119,000	$ 5,500
E	1	$ - -	$ 100	$ 20,000	$ - -	$ 20,000	$ 100
	2	- -	300	48,000	- -	48,000	300
	3	- -	600	83,000	- -	83,000	600
	4	- -	200	34,000	- -	34,000	200
	5	- -	600	117,000	- -	117,000	600
	6	- -	400	193,000	- -	193,000	400
	7	- -	400	210,000	- -	210,000	400
	8	- -	200	162,000	- -	162,000	200
	9	- -	2,500	105,000	- -	105,000	2,500
	10	- -	1,400	52,000	- -	52,000	1,400
Subtotal		$ - -	$ 6,700	$1,024,000	$ - -	$ 1,024,000	$ 6,700
F	1	$ - -	$ 300	$ 41,000	$ - -	$ 41,000	$ 300
	2	- -	300	123,000	- -	123,000	300
	3	- -	500	355,000	- -	355,000	500
	4	- -	3,800	156,000	- -	156,000	3,800
Subtotal		$ - -	$ 4,900	$ 675,000	$ - -	$ 675,000	$ 4,900
G	1	$ 9,000	$ 100	$ 8,000	$ - -	$ 17,000	$ 100
	2	7,000	100	8,000	- -	15,000	100
	3	4,000	0	5,000	- -	9,000	0
	4	10,000	200	10,000	- -	20,000	200
	5	- -	3,300	219,000	- -	219,000	3,300
	6	37,000	1,600	36,000	- -	73,000	1,600
Subtotal		$ 67,000	$ 5,300	$ 286,000	$ - -	$ 353,000	$ 5,300
H	1	$ - -	$ 200	$ 41,000	$ - -	$ 41,000	$ 200
	2	- -	300	62,000	- -	62,000	300
	3	- -	100	61,000	- -	61,000	100
	4	151,000	200	- -	- -	151,000	200
	5	46,000	0	- -	- -	46,000	0
	6	- -	200	207,000	- -	207,000	200
	7	63,000	600	- -	- -	63,000	600
	8	10,000	0	- -	- -	10,000	0
	9	20,000	0	- -	- -	20,000	0
	10	4,000	0	- -	- -	4,000	0
	11	102,000	400	71,000	- -	173,000	400
	12	10,000	100	16,000	- -	26,000	100
	13	108,000	4,700	188,000	- -	296,000	4,700
	14	424,000	6,900	- -	- -	424,000	6,900
Subtotal		$ 938,000	$ 13,700	$ 646,000	$ - -	$ 1,584,000	$ 13,700
I	1	$ - -	$ 1,300	$ 44,000	$ - -	$ 44,000	$ 1,300
Subtotal		$ - -	$ 1,300	$ 44,000	$ - -	$ 44,000	$ 1,300
J	1	$ 40,000	$ 0	$ - -	$ - -	$ 40,000	$ 0
	2	7,000	0	- -	- -	7,000	0
	3	- -	100	44,000	- -	44,000	100
	4	- -	0	9,000	- -	9,000	0

Table 57 (continued)

Hydrologic Unit Designation	Component Designation	Public Sector		Private Sector		Total	
		Capital[a]	Operation and Maintenance	Capital[a]	Operation and Maintenance	Capital[a]	Operation and Maintenance
J (continued)	5	$ 39,000	$ 200	$ --	$ --	$ 39,000	$ 200
	6	13,000	300	--	--	13,000	300
	7	--	0	10,000	--	10,000	0
	8	--	2,800	123,000	--	123,000	2,800
Subtotal		$ 99,000	$ 3,400	$ 186,000	$ --	$ 285,000	$ 3,400
K	1	$ 71,000	$ 100	$ --	$ --	$ 71,000	$ 100
	2	10,000	0	--	--	10,000	0
	3	12,000	100	--	--	12,000	100
	4	--	200	74,000	--	74,000	200
	5	--	0	27,000	--	27,000	0
	6	--	0	16,000	--	16,000	0
	7	--	0	5,000	--	5,000	0
Subtotal		$ 93,000	$ 400	$ 122,000	$ --	$ 215,000	$ 400
L	1	$ 156,000	$ 200	$ --	$ --	$ 156,000	$ 200
	2	141,000	100	--	--	141,000	100
	3	43,000	0	--	--	43,000	0
	4	6,000	100	--	--	6,000	100
Subtotal		$ 346,000	$ 400	$ --	$ --	$ 346,000	$ 400
Drainage Subtotal		$3,450,000	$ 61,800	$5,055,000	$ --	$ 8,505,000	$ 61,800
Water Quality Management Plan Element (refer to Table 49)							
A	16	$ --	$ --	$ 396,000	$1,000	$ 396,000	$ 1,000
	17	200	400	--	--	200	400
	18	--	10,500	373,000	--	373,000	10,500
Subtotal		$ 200	$ 10,900	$ 769,000	$1,000	$ 769,200	$ 11,900
B	11	$ --	$ --	$ 426,000	$1,100	$ 426,000	$ 1,100
	12	700	1,600	--	--	700	1,600
	13	--	6,400	247,000	--	247,000	6,400
	14	138,000	2,700	--	--	138,000	2,700
	15	84,000	2,700	--	--	84,000	2,700
	16	126,000	3,600	--	--	126,000	3,600
	17	160,000	3,100	--	--	160,000	3,100
	18	--	2,300	71,000	--	71,000	2,300
	19	71,000	2,100	--	--	71,000	2,100
Subtotal		$ 579,700	$ 24,500	$ 744,000	$1,100	$ 1,323,700	$ 25,600
C	16	$ --	$ --	$ 138,000	$ 300	$ 138,000	$ 300
	17	100	300	--	--	100	300
	18	--	3,500	135,000	--	135,000	3,500
	19	89,000	4,400	--	--	589,000	4,400
	20	--	1,300	26,000	--	26,000	1,300
Subtotal		$ 589,100	$ 9,500	$ 299,000	$ 300	$ 888,100	$ 9,800
D	17	$ --	$ --	$ 183,000	$ 500	$ 183,000	$ 500
	18	500	1,200	--	--	500	1,200
	19	--	3,700	132,000	--	132,000	3,700
	20	--	2,600	81,000	--	81,000	2,600
Subtotal		$ 500	$ 7,500	$ 396,000	$ 500	$ 396,500	$ 8,000

Table 57 (continued)

Hydrologic Unit Designation	Component Designation	Public Sector		Private Sector		Total	
		Capital[a]	Operation and Maintenance	Capital[a]	Operation and Maintenance	Capital[a]	Operation and Maintenance
E	13	$ - -	$ - -	$ 315,000	$ 800	$ 315,000	$ 800
	14	- -	5,000	230,000	- -	230,000	5,000
	15	- -	1,800	51,000	- -	51,000	1,800
	16	- -	1,600	44,000	- -	44,000	1,600
Subtotal		$ - -	$ 8,400	$ 640,000	$ 800	$ 640,000	$ 9,200
F	6	$ - -	$ - -	$ 201,000	$ 500	$ 201,000	$ 500
	7	- -	3,000	107,000	- -	107,000	3,000
Subtotal		$ - -	$ 3,000	$ 308,000	$ 500	$ 308,000	$ 3,500
G	7	$ - -	$ - -	$ 225,000	$ 600	$ 225,000	$ 600
	8	- -	5,000	276,000	- -	276,000	5,000
	9	28,000	2,000	28,000	- -	56,000	2,000
Subtotal		$ 28,000	$ 7,000	$ 529,000	$ 600	$ 557,000	$ 7,600
H	15	$ - -	$ - -	$ 414,000	$1,000	$ 414,000	$ 1,000
	16	406,000	11,000	- -	- -	406,000	11,000
Subtotal		$ 406,000	$ 11,000	$ 414,000	$1,000	$ 820,000	$ 12,000
I	2	$ - -	$ - -	$ 36,000	$ 100	$ 36,000	$ 100
	3	- -	1,600	44,000	- -	44,000	1,600
Subtotal		$ - -	$ 1,600	$ 80,000	$ 100	$ 80,000	$ 1,700
J	9	$ - -	$ - -	$ 147,000	$ 400	$ 147,000	$ 400
	10	- -	4,200	148,000	- -	148,000	4,200
	11	35,000	1,400	- -	- -	35,000	1,400
Subtotal		$ 35,000	$ 5,600	$ 295,000	$ 400	$ 330,000	$ 6,000
K	8	$ - -	$ - -	$ 78,000	$ 200	$ 78,000	$ 200
	9	100	200	- -	- -	100	200
	10	55,000	2,500	54,000	- -	109,000	2,500
	11	- -	1,400	35,000	- -	35,000	1,400
Subtotal		$ 55,100	$ 4,100	$ 167,000	$ 200	$ 222,100	$ 4,300
L	5	$ - -	$ - -	$ 183,000	$ 500	$ 183,000	$ 500
	6	189,000	4,300	- -	- -	189,000	4,300
	7	331,000	7,600	- -	- -	331,000	7,600
Subtotal		$ 520,000	$ 11,900	$ 183,000	$ 500	$ 703,000	$ 12,400
Water Quality Subtotal		$2,213,600	$105,000	$4,824,000	$7,000	$ 7,037,600	$112,000
Flood Control Plan Element (refer to Table 52)							
- -	A	$2,725,000	$ - -	$ - -	$ - -	$ 2,725,000	$ - -
	B	326,000	- -	- -	- -	326,000	- -
	C	89,000	- -	- -	- -	89,000	- -
	D	250,000	- -	- -	- -	250,000	- -
	E	20,000	- -	- -	- -	20,000	- -
	F	- -	- -	25,000	- -	25,000	- -
	G	180,000	- -	- -	- -	0	- -
Flood Control Subtotal		$3,590,000	$ 5,000	$ 25,000	$ - -	$ 3,615,000	$ 5,000
Total		$9,253,600	$171,800	$9,904,000	$7,000	$19,157,600	$178,800

[a] Includes engineering, administration, and contingencies. Costs based on 1989 Engineering News-Record Construction Cost Index of 4,725.

Source: SEWRPC.

detention basins in areas of existing urban development. Department funding may also be available for up to 50 percent of the land acquisition cost, up to 50 percent of the cost of the conveyance components required to divert runoff into treatment facilities, and up to 100 percent of the design and engineering costs for structural best management practices which serve existing urban development.

Under current policy, state funds are not available for the construction of nonpoint source control measures in areas of new development or for the operation and maintenance of any nonpoint source control measures. Although present DNR policy does not provide cost-sharing for these items, such cost sharing is not prohibited by Chapter NR 120 of the Wisconsin Administrative Code, which details the administrative procedures of the state nonpoint source water pollution abatement program. Chapter NR 120, however, expressly forbids provision of state funds for construction site erosion control. State funds may be provided for accelerated street sweeping above the current levels practiced by the Village. The funds would cover the costs of accelerated sweeping, as defined in the Menomonee River Priority Watershed Study, for a five-year period, after which the Village would be required to maintain the accelerated sweeping schedule for 10 years. Tables 58 and 59 provide possible allocations of costs between the Village, the State, and the private sector on the basis of current state cost-sharing policy.

The Menomonee River Priority Watershed Study was completed by the Wisconsin DNR in 1991. Best management practices constructed with state funds provided under the Wisconsin Fund Nonpoint Source Pollution Abatement Program, which is administered by the Department, must be completed by October 1999. In addition to funds provided by the Wisconsin DNR, it is also possible that the cost of certain recommended components of the stormwater drainage system may be shared between the Village and the Wisconsin DOT as a part of future highway construction or reconstruction projects. Because the division of costs for such measures is presently unknown, this plan assigns all such costs to the Village.

All operation and maintenance costs, except those for construction erosion control, were assumed to be financed by the public sector regardless of whether public sector or private sector funds were used to construct the facilities. It may be desirable for the operation and maintenance costs of some stormwater drainage and nonpoint source pollution control measures to be borne by the private sector, depending on the specific nature of individual projects. If operation and maintenance costs for a specific project are financed by the private sector, it would be necessary for the Village and the party responsible for operation and maintenance to execute a legal agreement which details both the responsibility of the private party for providing operation and maintenance and the degree of maintenance to be provided. Those stormwater management facilities which are constructed with private funds, but maintained by the Village, would be dedicated to the Village following construction.

Prioritization of Capital Improvements

A preliminary prioritization of the recommended capital improvements is given in Table 60. This prioritization is provided to identify those projects which should be implemented to alleviate the most pressing stormwater management and flood control problems and to identify a necessary sequence for implementation of certain interdependent components of the total system. For this prioritization, a project is defined as a set of stormwater management or flood control components which should be constructed in concert in order for the set to function properly by itself and within the context of the larger total system of which it is a part. In some instances, several projects in the same localized area were grouped together as one larger project for the purposes of prioritization. An economy of scale may be possible by constructing several small projects in the same area at the same time.

The projects are classified as high-, intermediate-, or low-priority. The high-priority projects are those which address significant existing problems, or those which are required to serve new development which is actually occurring. The intermediate-priority projects are those required to serve new development anticipated to occur in the near future based on development proposals which have been submitted to the Village. The low-priority projects are those required to serve and promote development in the more distant future. The storm frequency for which certain projects are to be designed and the consequences of exceeding the capacity of the

Table 58

ASSIGNMENT OF VILLAGE, STATE, AND PRIVATE CAPITAL COSTS OF THE RECOMMENDED LILLY CREEK WATER QUALITY MANAGEMENT PLAN ELEMENT BASED ON CURRENT STATE COST-SHARING POLICY[a]

Hydrologic Unit Designation	Component Designation	Village of Menomonee Falls	Private Sector	State of Wisconsin	Total Capital Cost
		Capital Cost	Capital Cost	Capital Cost	
Water Quality Management Plan Element (refer to Table 49)					
A	16	$ - -	$ 396,000	$ - -	$ 396,000
	17	200	- -	- -	200
	18	- -	373,000	- -	373,000
Subtotal		$ 200	$ 769,000	$ - -	$ 769,200
B	13	$ - -	$ 426,000	$ - -	$ 426,000
	14	700	- -	- -	700
	15	- -	247,000	- -	247,000
	16	41,000	- -	97,000	138,000
	17	25,000	- -	59,000	84,000
	18	38,000	- -	88,000	126,000
	19	48,000	- -	112,000	160,000
	20	- -	71,000	- -	71,000
	21	21,000	- -	50,000	71,000
Subtotal		$173,700	$ 744,000	$ 406,000	$1,323,700
C	16	$ - -	$ 138,000	$ - -	$ 138,000
	17	100	- -	- -	100
	18	- -	78,000	57,000	135,000
	19	194,000	- -	395,000	589,000
	20	- -	26,000	- -	26,000
Subtotal		$194,100	$ 242,000	$ 452,000	$ 888,100
D	14	$ - -	$ 183,000	$ - -	$ 183,000
	15	500	- -	- -	500
	16	- -	40,000	92,000	132,000
	17	- -	58,000	23,000	81,000
Subtotal		$ 500	$ 281,000	$ 115,000	$ 396,500
E	11	$ - -	$ 315,000	$ - -	$ 315,000
	12	- -	230,000	- -	230,000
	13	- -	51,000	- -	51,000
	14	- -	44,000	- -	44,000
Subtotal		$ - -	$ 640,000	$ - -	$ 640,000
F	6	$ - -	$ 201,000	$ - -	$ 201,000
	7	- -	107,000	- -	107,000
Subtotal		$ - -	$ 308,000	$ - -	$ 308,000
G	7	$ - -	$ 225,000	$ - -	$ 225,000
	8	- -	276,000	- -	276,000
	9	18,000	18,000	20,000	56,000
Subtotal		$ 18,000	$ 519,000	$ 20,000	$ 557,000
H	15	$ - -	$ 414,000	$ - -	$ 414,000
	16	292,000	- -	114,000	406,000
Subtotal		$292,000	$ 414,000	$ 114,000	$ 820,000

Table 58 (continued)

Hydrologic Unit Designation	Component Designation	Village of Menomonee Falls	Private Sector	State of Wisconsin	Total Capital Cost
		Capital Cost	Capital Cost	Capital Cost	
I	2	$ - -	$ 36,000	$ - -	$ 36,000
	3	- -	44,000	- -	44,000
Subtotal		$ - -	$ 80,000	$ - -	$ 80,000
J	9	$ - -	$ 147,000	$ - -	$ 147,000
	10	- -	44,000	104,000	148,000
	11	10,000	- -	25,000	35,000
Subtotal		$ 10,000	$ 191,000	$ 129,000	$ 330,000
K	8	$ - -	$ 78,000	$ - -	$ 78,000
	9	100	- -	- -	100
	10	31,000	32,000	46,000	109,000
	11	- -	35,000	- -	35,000
Subtotal		$ 31,100	$ 145,000	$ 46,000	$ 222,100
L	5	$ - -	$ 183,000	$ - -	$ 183,000
	6	64,000	- -	125,000	189,000
	7	105,000	- -	226,000	331,000
Subtotal		$169,000	$ 183,000	$ 351,000	$ 703,000
Total		$888,600	$4,516,000	$1,633,000	$7,037,600

[a] *Cost assignment assumes 70 percent of the capital cost of each eligible best management practice and 50 percent of land acquisition cost for eligible components are funded by the State of Wisconsin.*

Source: SEWRPC.

Table 59

POSSIBLE APPORTIONMENT OF TOTAL VILLAGE OF MENOMONEE FALLS, STATE OF WISCONSIN, AND PRIVATE-SECTOR COSTS FOR THE LILLY CREEK STORMWATER MANAGEMENT PLAN BASED ON CURRENT STATE COST-SHARING POLICY

Plan Element	Village of Menomonee Falls			State of Wisconsin			Private Sector		
	Capital[a]	Annual Operation and Maintenance	Present Value[b]	Capital[a]	Annual Operation and Maintenance	Present Value[b]	Capital[a]	Annual Operation and Maintenance	Present Value[b]
Stormwater Drainage System	$3,450,000	$ 61,800	$ 4,424,100	$ - -	$- -	$ - -	$5,055,000	$ - -	$5,055,000
Water Quality Management Measures	888,600	105,000	2,543,600	1,633,000	- -	1,633,000	4,516,000	7,000	4,626,300
Flood Control Measures	3,590,000	5,000	3,668,800	- -	- -	- -	25,000	- -	25,000
Recreational and Stream Habitat Enhancement Measures in Reaches Where Channel Modification is Not Recommended	156,000	- -	156,000	2,000	- -	2,000	- -	- -	- -
Total	$8,084,600	$171,800	$10,792,500	$1,635,000	$- -	$1,635,000	$9,596,000	$7,000	$9,706,300

[a] *Includes engineering, administration, and contingencies. Costs based on 1989 Engineering News-Record Construction Cost Index of 4,725.*

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

Source: SEWRPC.

Table 60

PRIORITIZATION OF RECOMMENDED STORMWATER MANAGEMENT AND FLOOD CONTROL PROJECTS FOR THE LILLY CREEK SUBWATERSHED

Project Number and Designation	Hydrologic Unit	Project Components	Village of Menomonee Falls Capital Cost[a,b]	State of Wisconsin Capital Cost[a,b]	Private Sector Capital Cost[a,b]	Total Capital Cost of Components[a,b]
High-Priority Projects 1. Construction of detention basin WD16	H	Table 49 H14 and H16	$ 716,000	$ 114,000	$ - -	$ 830,000
2. Lilly Creek flood control and stream restoration	B, C, D, E, I, K, and L	Table 59	3,590,000	- -	- -	3,590,000
3. Floodproofing of one single-family residence located at River Mile 0.65	K	Table 59	- -	- -	25,000	25,000
4. Construction of detention basin WD7 associated tributary storm sewers, and the Bowling Green Tributary diversion pipe	C	Table 49 C1, 2, 3, (partial), C4 (partial), C8 through C12, C14, and C18	455,000	57,000	380,000	892,000
5. Modification of Oakwood Tributary and associated culvert replacement	H	Table 49 H7, H8, H9, and H10	95,000	- -	- -	95,000
6. Construction of detention basins WD21 and WD24	L	Table 49 L6 and L7	169,000	351,000	- -	520,000
7. Relief storm sewer between Manor Drive and W. Appleton Avenue	L	Table 49 L1, L2, and L3	340,000	- -	- -	340,000
8. Bowling Green Industrial Park storm sewers and detention basin WD8	C	Table 49 C3 (partial), C4 (partial), C5 through C7, and C19	540,000	395,000	- -	935,000
9. Construction of detention basins DD1, WD3, and WD4	B	Table 49 B11, B12, B16, B17, and B18	363,000	244,000	- -	607,000
10. West Appleton Avenue storm sewer replacement	K	Table 49 K1 through K3	93,000	- -	- -	93,000
11. Floodproofing of two houses along Phillips Tributary, two houses along Woodshaven Tributary, and one house south of the intersection of W. Appleton Avenue and North Point Drive	B, J, and K	Table 49 B8, J7, and K7	- -	- -	25,000	25,000
12. Modification of Phillips Tributary in the reach west of Pilgrim Road and replacement of the Pilgrim Road culverts	B	Table 49 B7 and B9	95,000	- -	- -	95,000
13. Recommended limited modification of the Woodshaven Tributary channel and associated culvert replacement	Table 49 J1, J2, and J5	Table 49 J1, J2, and J5	86,000	- -	- -	86,000
Intermediate-Priority Projects 14. Storm sewers and detention basin WD11 for that portion of the Mill Ridge subdivision located in Hydrologic Unit D	D	Table 49 D1, D2, (partial), D3 (partial), D4 (partial), D5, D8, and D17	$ - -	$ 23,000	$ 323,000	$ 346,000
15. Hidden Meadows subdivision including detention basin WD19	J	Table 49 J3, J4, J8 and J10	- -	104,000	220,000	324,000
16. Detention basin WD15 in the proposed addition to Cedar Ridge subdivision	G	Table 49 G5 and G8	- -	- -	495,000	495,000
17. Detention basin WD2 and associated outlet located in the proposed Wildlife Ridge subdivision	B	Table 49 B4 (partial), B5 (partial), B10 and B15	- -	- -	440,000	440,000

Table 60 (continued)

Project Number and Designation	Hydrologic Unit	Project Components	Village of Menomonee Falls Capital Cost[a,b]	State of Wisconsin Capital Cost[a,b]	Private Sector Capital Cost[a,b]	Total Capital Cost of Components[a,b]
Intermediate-Priority Projects (continued) 18. Storm sewers in the area west of recommended detention basin WD2	B	Table 49 B3, B4 (partial), B5 (partial), and B6	$ - -	$ - -	$ 162,000	$ 162,000
19. Mill Road storm sewer	D	Table 49 D2 (partial), D3 (partial), D6 and D7 (partial), D9, and D10	555,000	- -	- -	555,000
20. Phase II of the Single Tree subdivision and the upstream storm sewers proposed for the existing Country Lanes subdivision	H	Table 49 H4 through H6	196,000	- -	207,000	403,000
21. Lincoln Lane Tributary storm sewers	F	Table 49 F1, F2, and F3 (partial)	- -	- -	334,000	334,000
22. Modification of Silver Spring Tributary channel and associated culvert replacements at Village Fire Station and Badger Drive	A	Table 49 A11 through A14	141,000	- -	- -	141,000
23. Construction of detention basin WD1 and storm sewers in the area north and south of W. Silver Spring Drive and west of Pilgrim Road	A	Table 49 A1 through A10, A15 and A18	- -	- -	1,184,000	1,184,000
24. Construction of detention basin WD18 and associated storm sewers	K	Table 49 K4, K5, K6, and K10	31,000	46,000	149,000	226,000
25. Construction of wet detention basins WD5, WD6, and WD26 to control nonpoint source pollution from areas of existing development	B and J	Table 49 B19, B21, and J11	79,000	187,000	- -	266,000
26. Detention basin WD23 and associated upstream storm sewers	H	Table 49 H11 (partial), H12, and H13	129,000	- -	193,000	322,000
27. W. Good Hope Road storm sewers	H	Table 49 H11 (partial)	102,000	- -	70,000	172,000
28. Storm Sewers to drain El Camino Drive, Mesa Drive, and Vista Lane	B	Table 49 B1 and B2	57,000	- -	- -	57,000
Low-Priority Projects 29. Storm sewers west of existing Country Lanes subdivision	H	Table 49 H1 through H3	$ - -	$ - -	$ 164,000	$ 164,000
30. Detention basin WD9 and associated storm sewers	D	Table 49 D4 (partial), D7 (partial), D11, D12, D13, and D16	- -	92,000	350,000	442,000
31. Detention basin WD22 and associated storm sewers and modified channel	G	Table 49 G1 through G4, G6, and G9	85,000	20,000	85,000	190,000
32. Recommended dry detention basin DD3 and associated open channel	C	Table 49 C13, C15, and C20	- -	- -	46,000	46,000
33. Detention basin WD27 to control nonpoint source pollution from an area of planned new development	K	Table 49 K11	- -	- -	35,000	35,000
34. Detention basin WD17 and associated roadside swales	I	Table 49 I1 and I3	- -	- -	88,000	88,000

Table 60 (continued)

Project Number and Designation	Hydrologic Unit	Project Components	Village of Menomonee Falls Capital Cost[a,b]	State of Wisconsin Capital Cost[a,b]	Private Sector Capital Cost[a,b]	Total Capital Cost of Components[a,b]
Low-Priority Projects (continued)						
35. Detention basins WD12 and DD5 and associated storm sewers	E	Table 49 E1 through E10, E12, and E13	$ --	$ --	$1,305,000	$ 1,305,000
36. Detention basin WD14 to control nonpoint source pollution from an area of planned new development	E	Table 49 E14	--	--	44,000	44,000
37. Detention basin WD25 to control nonpoint source pollution from an area of planned new development	B	Table 49 B20	--	--	71,000	71,000
38. Provide riprap along 0.3 mile of the Woodshaven Tributary	J	Table 49 J6	13,000	--	--	13,000
39. Provide riprap along Menomonee Manor Tributary	L	Table 49 L4	6,000	--	--	6,000
Recommended Components Which Have Been Constructed						
40. Cedar Ridge subdivision along Lincoln Lane Tributary	F	Table 49 F3 (partial), F4, and F7	$ --	$ --	$ 450,000	$ 450,000
Recommended Accelerated Sweeping of Commercial and Industrial Streets	A, B, C, D, and K	Table 49 A17, B14, C17, D15, and K9	$ 1,600	$ --	$ --	$ 1,600
Construction Erosion Control to be Accomplished as Development Proceeds	All	Table 49 A16, B13, C16, D14, E11, F6, G7, H15, I2, J9, K8, and L5	$ --	$ --	$2,742,000	$ 2,742,000
Total	--	--	$7,937,600[c]	$1,633,000[c]	$9,587,000[c]	$19,157,600

[a] *City, private, and state costs are apportioned according to current state cost-sharing policy.*

[b] *Includes engineering, administration, and contingencies. Costs based on 1989 Engineering News-Record Construction Cost Index of 4,725.*

[c] *Public- and private-sector totals differ from those set forth in Table 57 because Table 57 does not account for State of Wisconsin cost-sharing, which could reduce both the Village's and private-sector costs.*

Source: SEWRPC.

existing stormwater management system were also considered in the prioritization.

The sequence in which projects are actually implemented and the time at which they are implemented will ultimately depend on a number of factors including, importantly, the sequence in which the facilities must be constructed to avoid the creation of new problems or the intensification of existing problems. Not all the factors to be considered are related solely to stormwater management. Factors related to other considerations include budgetary constraints; the need to implement other projects not related to stormwater management in the Village's capital improvements program; and variations in future development patterns as determined by the urban land market. As a result, some intermediate-priority projects may actually be constructed before some high-priority projects. However, where a specific implementation sequence for a series of components comprising a unified stormwater management or flood control project is required, that sequence should be followed to ensure the proper functioning of the system.

Identification of Critical Implementation Sequences

The following section identifies projects for which the implementation sequence of the project components is critical. The project numbers are those assigned in Table 60.

Project 1—Detention Basin WD16: Detention basin WD16 should be constructed prior to the Lilly Creek channel modification because the basin would provide a significant reduction in flood flows under both existing and planned land use conditions. That reduction in flows must be achieved in order to avoid exceeding the capacity of the recommended Lilly Creek channel modification.

Project 2—Lilly Creek Flood Control and Stream Restoration: Because of the interrelationships between the flood storage and conveyance components of this project, it should ideally be constructed in its entirety over a relatively short period of time, rather than in phases extending over a longer period. The channel modifications and bridge replacements called for under this project should proceed from downstream to upstream to ensure that the existing upstream floodplain storage capacity is maintained as much as possible, thereby limiting flood flows as construction proceeds. Also, the three recommended flood storage areas to be provided in the west overbank should be constructed at the same time as, or before, the adjacent channel modifications in the respective reaches.

Projects 1, 4, 14 through 17, 23, 24, 26, and 31 through 37—Detention Basins to Serve New Upstream Development: Detention basins intended to collect runoff from new upstream development or from a mix of new and existing upstream development should be constructed before the new upstream development. Detention basins in this category include WD1, 2, 7, 11, 12, 14 through 19, 22, 23, 25, 27, DD3, and DD5. In some cases, the new development which contributes runoff to the basin may not be located adjacent to the basin. In such instances, the Village could collect funds from the developer to cover the portion of the cost of the basin which is attributable to the runoff from the new development and the remainder of the basin cost should be provided by the Village, with those costs being recovered from other developers as development proceeds over time.

Project 4 and Projects 26 and 27—Detention Facilities Enabling Downsizing of Downstream Stormwater Drainage Conveyance Facilities: Detention basins which enable the construction of smaller downstream conveyance facilities should be constructed before the conveyance facilities. Thus, under Project 4, basin WD7 should be in place before construction of the Bowling Green Tributary diversion pipe. Also, basin WD23 as called for under Project 26, should be constructed before the W. Good Hope Road storm sewers, called for under Project 27.

Projects 5, 7, 8, 10, 12, 13, 20, and 22—Channel Modifications, Including Culvert Replacements or Storm Sewer Projects in Areas of Existing Development: Each one of these projects should proceed from downstream to upstream to ensure that the downstream channels, storm sewers, and/or culverts which are a part of that project have adequate capacity to pass the increased flows resulting from the provision of increased upstream hydraulic capacity.

REGULATORY CONSIDERATIONS

Implementation of some of the drainage improvements recommended in this system plan may require the prior approval of certain regulatory agencies other than the Village, including the Wisconsin DNR and the U. S. Army Corps of Engineers. Because the regulatory process involved is complex, the Village should seek legal counsel before proceeding with stormwater management and flood control improvements which involve the construction or improvement of artificial waterways connecting to navigable waters, the alteration or enclosure of navigable watercourses, the removal of material from the beds of navigable watercourses, or the disturbance of wetlands.

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

State regulatory authority relates to the construction or improvement of artificial waterways, canals, or ponds connecting to, or located within 500 feet of, a navigable waterway; the alteration of navigable waterways; the placement of deposits or structures in the bed of navigable waterways or the enclosure of navigable waterways; the removal of material from navigable waterways; and also to activities affecting the water quality of wetlands. This authority is contained the Sections 30.12, 30.195, 30.20 and 144.025 of the Wisconsin Statutes. The administering agency is the Wisconsin DNR.

Chapters of the Wisconsin Administrative Code which are pertinent to activities called for under the recommended plan include Chapter NR 103, "Water Quality Standards for Wetlands"; Chapter NR 116, "Wisconsin's Floodplain Management Program"; and Chapter NR 117, "Wisconsin's City and Village Shoreland-Wetland Protection Program." Because of the importance of the relatively new Chapter NR 103 regulations, special analyses have been conducted under this planning effort to address the requirements of this Chapter of the Code as set forth in Chapter VI and Appendix D of this report.

Implementation of the plan will allow the Federal Emergency Management Agency, upon the request of the Village, to revise the floodplain boundary maps following submittal of substantiating information. Such revisions should be requested immediately upon adoption of this plan and also as the recommended stormwater management and flood control measures are constructed. Revision will ultimately eliminate the need for many property owners in the Village to purchase flood insurance.

PLAN REEVALUATION AND UPDATING

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

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Chapter VIII

SUMMARY

The recommended stormwater management and flood control plan for the Lilly Creek subwatershed consists of a stormwater drainage plan element, a water quality management plan element, and a flood control plan element. Several alternative plans were developed and evaluated under each of the three plan elements. The recommended plan integrates the preferred alternative plan elements into an overall system for stormwater management and flood control in the subwatershed under planned ultimate development conditions.

The recommended plan consists of minor and major stormwater drainage system components; components for the control of nonpoint source pollution, stream-bank erosion, and streambed scour; and components for flood control, all as shown on Map 20 in Chapter VI of this report. The minor drainage system components were designed for a 10-year recurrence interval peak flow, while the major system components and the flood control components were designed for a 100-year recurrence interval peak flow. The water quality management components are intended to function most efficiently during storms with recurrence intervals of two years or less.

The stormwater drainage plan element calls for approximately 9.9 miles of new reinforced concrete storm sewers, ranging in diameter from 12 to 66 inches; 800 lineal feet of replacement storm sewers; 1,200 lineal feet of 36-inch-diameter reinforced concrete diversion pipe; 2,100 lineal feet of roadside swales; the preservation of much of the existing system of streams tributary to Lilly Creek; culvert replacements at eight road crossings; floodproofing of five houses; and four dry detention basins. Implementation of the stormwater drainage element of the plan would be expected to eliminate all structural monetary flood damages along tributaries to Lilly Creek during floods with recurrence intervals up to, and including, 100 years. Implementation would also be expected generally to reduce the exposure of people to drainage-related inconvenience by providing minor system conveyance facilities designed for floods with recurrence intervals up to, and including, 10 years.

The water quality management plan element calls for 13 wet detention basins solely for stormwater quality control, the use of grassed swales in areas of new suburban- and low-density residential development, accelerated street sweeping in industrial and commercial areas which are not tributary to wet detention basins, and the enforcement of the Village's construction erosion control ordinance. In addition, 11 dual-purpose detention basins are recommended for the control of both quantity and quality of stormwater. The recommended water quality management measures may be expected to reduce uncontrolled nonpoint source pollutant loadings from the area by 87 percent for sediment, 60 percent for phosphorus, and 65 percent for heavy metals. Those measures should be adequate to achieve the desired water use objectives for Lilly Creek and its tributaries. These objectives include limited recreational use and the maintenance of warmwater forage fish and aquatic life along Lilly Creek from Mill Road to the Menomonee River and limited recreational use and maintenance of limited fish and aquatic life along Lilly Creek upstream of Mill Road and along the tributaries to Lilly Creek.

The flood control plan element calls for the widening and deepening of 2.08 miles of the Lilly Creek channel; the replacement and/or removal of the bridges and culverts at Brentwood Drive, Lilly Road and W. Mill Road, Kaul Avenue, Bobolink Drive, and at several private drives; the floodproofing of one house; the creation of two wetland-overbank storage areas and one dry overbank storage area; and the implementation of stream restoration and enhancement measures to improve aquatic and terrestrial habitat conditions along the stream corridor. Implementation of the flood control element of the plan would be expected to eliminate all structural monetary flood damages during floods with recurrence intervals up to, and including, 100 years.

The total capital cost of the recommended plan, excluding recreational measures and stream habitat enhancement measures in reaches where channel modification is not recommended, approximates $19.2 million. Of that total cost, about $8.5 million, or 44 percent, is for the stormwater drainage plan element; about $7.1 million, or 37 percent, is for the water quality management plan element; and the remaining $3.6 million, or 19 percent, is for the flood control plan element. Of the total capital cost of the plan, about $7.9 million, or 41 percent, is recommended to be borne by the Village of Menomonee Falls; about $1.7 million, or 9 percent, is recommended to be borne by the State of Wisconsin; and about $9.6 million, or 50 percent, is recommended to be financed by the private sector, primarily land developers and land parcel purchasers. About $2.7 million, or 39 percent of the total cost of the water quality management element, is estimated to be required for construction erosion control under the Village's existing ordinance.

Of the total annual additional operation and maintenance cost of $178,800, about $171,800, or 96 percent, would be paid by the public sector, with the remaining $7,000, or 4 percent, paid by the private sector for maintenance of construction erosion control measures. About $61,800, or 34 percent of the total operation and maintenance cost, is for the stormwater drainage plan element; about $112,000, or 63 percent, is for the water quality management plan element; and about $5,000, or 3 percent, is for the flood control element.

The first step toward plan implementation is formal adoption of the plan by the Village Stormwater Management and Flood Control Advisory Committee, the Village Public Works and Plan Commissions, and the Village Board. Importantly, to avoid further state impediments to implementation, the plan should be endorsed by the Wisconsin Department of Natural Resources. Following such endorsement, Department review of permit applications for the construction of individual components of the plan should be routine, given the crucial interrelationships between the plan's multiple objectives of controlling stormwater runoff and floods, reducing nonpoint source pollution, and restoring and enhancing aquatic and riparian habitat.

Plan implementation should proceed under the Village's existing structure for the review, administration, and financing of stormwater management and flood control projects. The recommended plan should be integrated into the Village's public works and capital improvement programs to initiate construction of the recommended facilities and to ensure reliable and stable operation and maintenance of both the existing and new facilities. With respect to regulatory actions, the Village should review subdivision plats to determine conformance between the recommended plan and future land uses, street and lot layouts, and any required dedication of land for stormwater management purposes.

Upon adoption by the Village, the Lilly Creek Stormwater Management and Flood Control Plan should be adopted by the Regional Planning Commission as an amendment to the Menomonee River watershed plan. The recommendations contained in the Lilly Creek plan should have no significant effect on the conclusions and recommendations concerning flood flows and stages on the Menomonee River as developed under the stormwater drainage and flood control system plan prepared by the Commission for the Milwaukee Metropolitan Sewerage District. The Lilly Creek plan also represents a step toward implementation of the Wisconsin Department of Natural Resources nonpoint source priority watershed plan.

The plan identifies the most cost-effective and environmentally sound means of resolving existing and probable future stormwater management and flooding problems in the Lilly Creek subwatershed. Implementation of the recommended plan would provide protection against substantial inconvenience to residents during minor storm events and against major property damage or a significant hazard to human health and safety during major storm events. Implementation of the plan would improve water quality and aquatic habitat conditions in the subwatershed, thereby enhancing the potential use of the surface waters. Importantly, implementation of the plan would support continued sound land use development and redevelopment within the Village, enriching the quality of life for all its residents.

APPENDICES

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Appendix A

INSTREAM HABITAT MITIGATION CRITERIA TO BE CONSIDERED DURING FACILITIES DESIGN

Table A-1

HABITAT VARIABLES AND OPTIMUM INSTREAM HABITAT MITIGATION FOR SELECTED FISH SPECIES CURRENTLY MANAGED IN THE LILLY CREEK SUBWATERSHED

Life Requisites	Habitat Variables	Blacknose Dace (Rhinichythys atratulus)		Common Shiner (Notropis cornutus)		Creek Chub (Semotilus atromaculatus)		White Sucker (Catostomus commersoni)		Overall Optimum Habitat Criteria and Comments
		Optimum Range of Values	Suitability Index[a]	Optimum Range of Values	Suitability Index[a]	Optimum Range of Values	Suitability Index[a]	Optimum Range of Values	Suitability Index[a]	
Food/Cover	Total lengths of pools, expressed as a percent of total stream length[b]	40-85	(1.0)	45-55	(1.0)	40-60	(1.0)	40-60	(1.0)	50 percent pool[c]
	Pool depth (feet)	N/A	N/A	N/A	N/A	N/A	N/A	1.5-2.3 2.3-3.3	(0.7-1.0)	2.5-foot pool depth
	Average stream depth (feet)	N/A	N/A	N/A	N/A	1.6-3.3	N/A	N/A	N/A	2.0 feet for pools and runs
	Average stream width (feet)	1-18	(1.0)	N/A	N/A	6-20 3-6 1.5-3	(1.0) (0.8-1.0) (0.5-0.8)	N/A	N/A	3 to 4 feet for low-flow channel
	Pool Class (percent cover)[d]	N/A	N/A	Class B Class A	(1.0) (0.4)	Class A Class B	(1.0) (0.6)	35-75[e]	(1.0)	50 percent cover in pools to comprise woody debris, boulder, undercut, and overhead bank cover
	Percent stream surface area shaded	50-85	(1.0)	N/A	N/A	>70	(1.0)[f]	>50	(1.0)	75 percent shaded. To be dominated by grasses, trees, and shrubs
	Stream bank vegetation (food cover)	N/A	N/A	N/A	N/A	>70[g]	(1.0)	N/A	N/A	70 to 100 percent bank cover
	Stream gradient (fall/distance)	0.01-0.025	(1.0)	N/A	N/A	0.007-0.013	(1.0)	0.002-0.008	(1.0)	0.008
	Average velocity									
	Adult in pools	<1.0 feet per second	(1.0)	0-1.0 feet per second	(0.7-1.0)	0.3-1.3 feet per second	(1.0)	0.1-0.65 feet per second	(0.7-1.0)	0.5 feet per second
	Juvenile in riffle	0.5-1.0 feet per second	(1.0)	0-1.0 feet per second	(0.7-1.0)	0.3-1.3 feet per second	(1.0)	0.1-0.65 feet per second	(0.7-1.0)	1.0 feet per second
	Fry at stream margin	0-0.3 feet per second	(1.0)	N/A	N/A	0-0.3 feet per second	(1.0)	N/A	N/A	<0.3 feet per second
	Substrate									
	Adult in pools/runs	Gravel/cobble	(1.0)	Sand/gravel Rubble	(1.0) (0.8)	Rubble/gravel Boulder/sand	(1.0)[h] (0.7)	N/A	N/A	50 percent cobble/rubble (2.5- to 10-inch diameter) and 50 percent gravel (0.1- to 2.5-inch diameter)
	Juveniles in riffles	Pebble/gravel	(1.0)	Sand/gravel	(1.0)	Rubble/gravel	(1.0)[h]	N/A	N/A	100 percent gravel (0.1-to 2.5-inch diameter)
	Fry at stream margin	Silt/sand/debris	(1.0)	Sand/gravel	(1.0)	Rubble/gravel	(1.0)[h]	N/A	N/A	100 percent gravel (0.1- to 2.5-inch diameter); sufficient amounts of fines should be available
Reproduction	Substrate (in riffles)	Gravel	(1.0)	Sand/gravel	(1.0)	Gravel	(1.0)	Gravel Gravel/pebble	(1.0) (0.8)	100 percent gravel (0.1- to 2.5-inch diameter)
	Riffle depth	0-1 foot	(1.0)	N/A	N/A	N/A	N/A	0.5-0.8 feet	(1.0)	Average 0.5 foot during spring period May through June. May not be achievable due to low-flow conditions. May be mitigated through use of run areas for spawning purposes
	Average velocity in riffles	0.65-1.5 feet per second	(1.0)	0.5-.65 feet per second	(1.0)	0.65-1.3 feet per second	(1.0)	1.0-2.0 feet per second	(1.0)	1.0 feet per second
	Stream temperature[i]	15-23°C (May-June)	(1.0)	15-18°C (May-July)	(1.0)	12-20°C (May-June)	(1.0)	21-23°C weekly average (May-June) fry	(1.0)	Highly variable. 15-22°C May through June
Water Quality[j]	Temperature	12-22°C summer maximum	(1.0)	22°C one week maximum	(1.0)	18-22°C summer average	(1.0)	18-24°C summer average	(1.0)	25°C or cooler if feasible
		22-26°C summer maximum	(0.5)	25°C one week maximum	(0.5)	29°C summer average	(0.5)	29°C summer average	(0.5)	
		≥30°C summer maximum	(0.0)	≥30°C one week maximum	(0.0)	≥32°C summer average	(0.0)	≥34°C summer average	(0.0)	
	Dissolved oxygen									
	Warmwater, or full, fish and aquatic life	N/A	N/A	N/A	N/A	≥5 mg/l all life stages	(1.0)	≥6 mg/l summer all life stages	(1.0)	5.0 mg/l
	Limited, or intermediate, fish and aquatic life	- -	- -	- -	- -	3-5 mg/l	(0.5-1.0)	3-5 mg/l	(0.5-0.9)	3.0 mg/l
	pH	- -	- -	- -	- -	≤3 mg/l	(0.0-0.5)	≤1.5-3 mg/l	(0.0-0.5)	6 to 9 standard units

[a] *Habitat suitability index (HSI) is given in parentheses for each variable and species. The HSI values can range from 0.0 to 1.0, with 1.0 being optimum.*

[b] *Optimum percent pool or run during summer low-flow periods for adult, juvenile, and fry life stages.*

[c] *Use 70 percent as an optimum value for pools plus deep runs in order to provide improved deep water habitat under low-flow conditions.*

[d] *Pool classes:*

- *A. "Large and deep" to provide sufficient low-velocity resting area. More than 30 percent of bottom is obscured by debris, surface turbulence, boulders, and overhanging vegetation and bank. Optimum depth identified for white sucker is sufficient.*
- *B. "Moderate size and depth" to provide low-velocity resting area. Five to 30 percent of bottom is obscured by debris, surface turbulence, boulders, and overhanging vegetation and bank. Optimum depth identified for white sucker is sufficient.*

[e] *Optimum pool cover: shade and instream cover as root systems, undercut banks, and woody debris are important cover for adult white sucker. Optimum instream and shoreline cover for all stream reaches in recommended.*

[f] *Optimum shading for water quality life requisite for maintenance of preferred water temperature.*

[g] *Optimum bank cover for food source. Suitability index (SI) is calculated from weighted stream bank vegetation types as follows: SI + [2(% shrubs) + 1.5 (% grasses) + (% trees) + 0(% bare soil)]/-100. Optimize through use of shrub and grass cover along banks.*

[h] *Optimum for food production to include more than 30 percent aquatic vegetation.*

[i] *Optimum temperature through range of spawning period for southeastern Wisconsin (from George Becker, Fishes of Wisconsin, University of Wisconsin Press, Madison, Wisconsin, 1983).*

[j] *The values given here indicate the sensitivity of different species to temperature and dissolved oxygen levels. The Commission standards for temperature and dissolved oxygen are appropriate for establishing overall use classifications for streams.*

Source: Wisconsin Department of Natural Resources.

Table A-2

ADDITIONAL OPTIMUM INSTREAM HABITAT MITIGATION CRITERIA TO BE CONSIDERED IN THE FACILITIES DESIGN FOR THE LILLY CREEK SUBWATERSHED

1. The formation of pool and riffle sections should be promoted through the construction of channel meanders in the low-flow channel.[a]
2. Where practicable, the low-flow channel side slope on the inside curve of a meander should be about one vertical on three horizontal and the side slope on the outside curve should be about one vertical on two horizontal.[b]
3. Pools should be spaced at interval ranging from five to seven times the channel width for a one-year recurrence interval flow.[c]
4. Pool lengths should be between one and three channel widths.[c]
5. Riffle lengths should be from one-half to two-thirds the length of pools.[c]
6. Sills constructed for instream habitat purposes should be provided with a notch to permit the passage of fish.[d]
7. Wing deflectors should be located so as to alternate between the right and left stream banks and should be oriented at an angle of 45 degrees, or less, with the stream bank.[c,e]
8. Erosion protection should be provided on the banks adjacent to, and opposite from, wing deflectors.[e]
9. The top of wing deflectors must be set low enough to allow overtopping during the mean annual flood.[f]
10. Low-flow channels in box culverts should be at least eight inches deep.[d]

[a] *Nelson R. Nunnally, "Stream Renovation: An Alternative to Channelization", Environmental Management, Vol. 2, No. 5, 1978.*

[b] *Linda Jewell, "Alternatives to Channelization," Landscape Architecture, July 1981.*

[c] *U. S. Army Corps of Engineers, EM 1110-2-1205, Environmental Engineering for Flood Control Channels, draft; January 1987.*

[d] *James R. Barton and Parley V. Winger, Stream Rehabilitation Concepts, Research Report submitted to Utah State Department of Highways, January 1974.*

[e] *Government of Canada, Province of British Columbia, Ministry of Environment, Stream Enhancement Guide, March 1980.*

[f] *Neal T. O'Reilly, Wisconsin Department of Natural Resources, personal communication, May 5, 1989.*

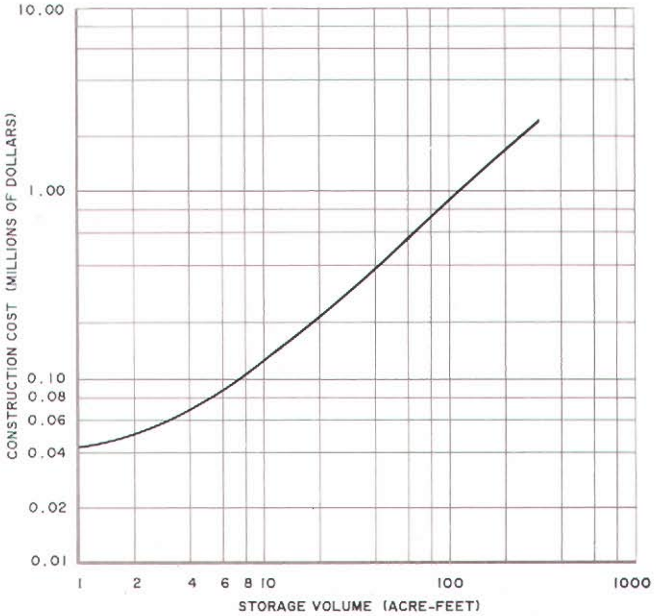
Source: SEWRPC.

Appendix B

STORMWATER MANAGEMENT AND FLOOD CONTROL PLAN FOR THE LILLY CREEK SUBWATERSHED IN THE VILLAGE OF MENOMONEE FALLS

Figure B-1

COST DATA FOR STORMWATER MANAGEMENT AND FLOOD CONTROL FACILITIES: SURFACE STORAGE FACILITY COST CURVE[a]

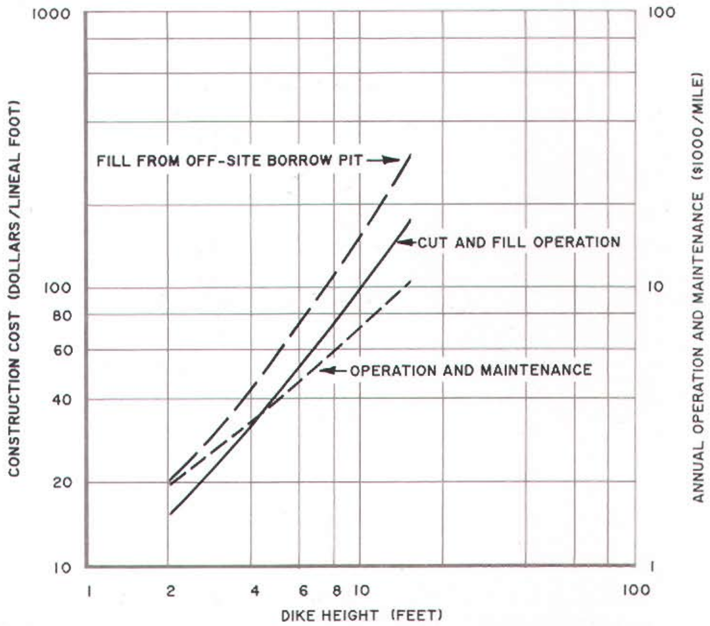


[a] *ENR CCI = 4,725 (1989). Does not include land acquisition, engineering, administration, and contingencies. For operation and maintenance costs, see SEWRPC Technical Report No. 31, Costs of Urban Nonpoint Source Water Pollution Control Measures, June 1991.*

Source: SEWRPC.

Figure B-2

COST DATA FOR STORMWATER MANAGEMENT AND FLOOD CONTROL FACILITIES: DIKE COST CURVES[a]

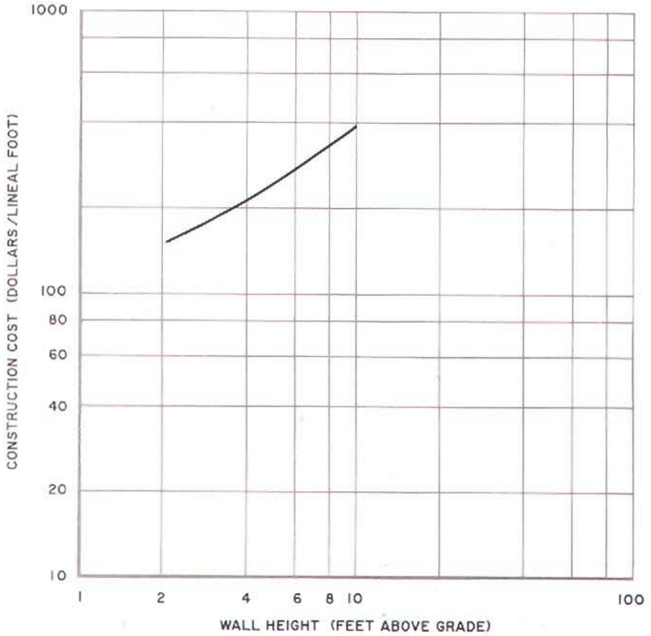


[a] *ENR CCI = 4,725 (1989). Does not include land acquisition, engineering, administration, and contingencies.*

Source: SEWRPC.

Figure B-3

FLOODWALL COST CURVES[a]

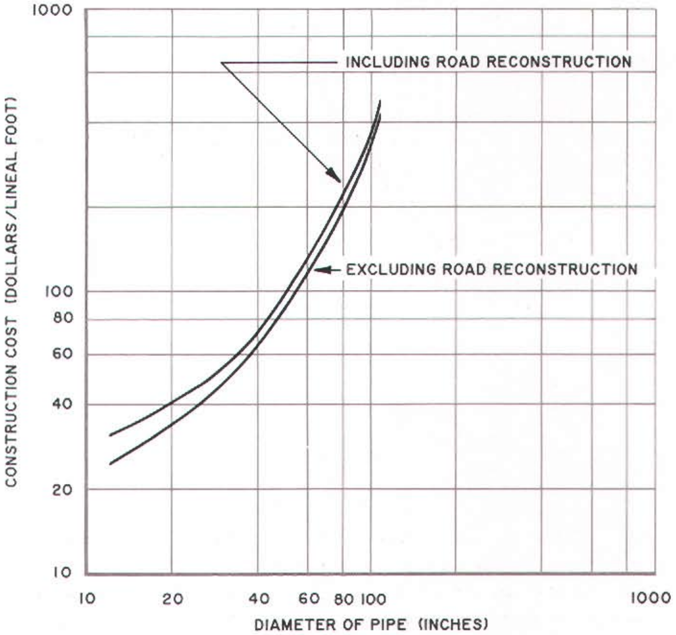


[a] *ENR CCI = 4,725 (1989). Does not include land acquisition, engineering, administration, and contingencies. Annual operation and maintenance costs equal $3,100 per year.*

Source: SEWRPC.

Figure B-4

REINFORCED CONCRETE PIPE COST CURVES[a]

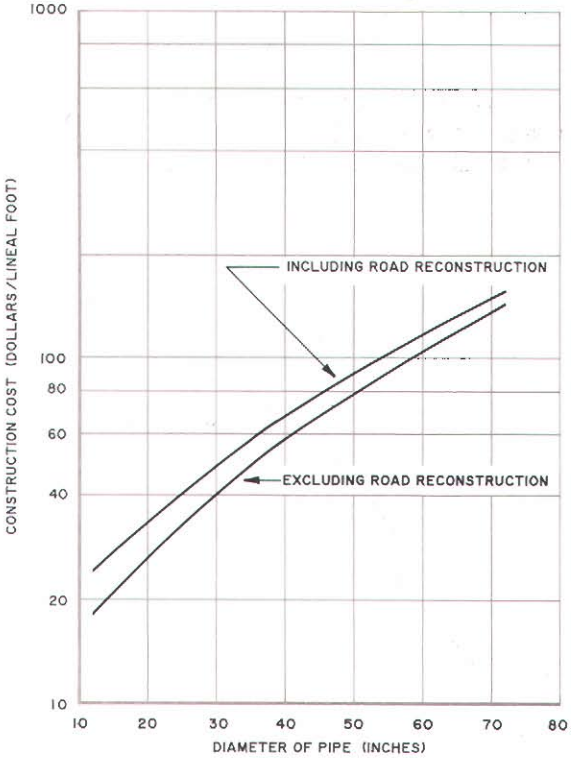


[a] *ENR CCI = 4,725 (1989). Does not include easements, engineering, administration, and contingencies..*

Source: SEWRPC.

Figure B-5

CORRUGATED METAL PIPE COST CURVES[a]

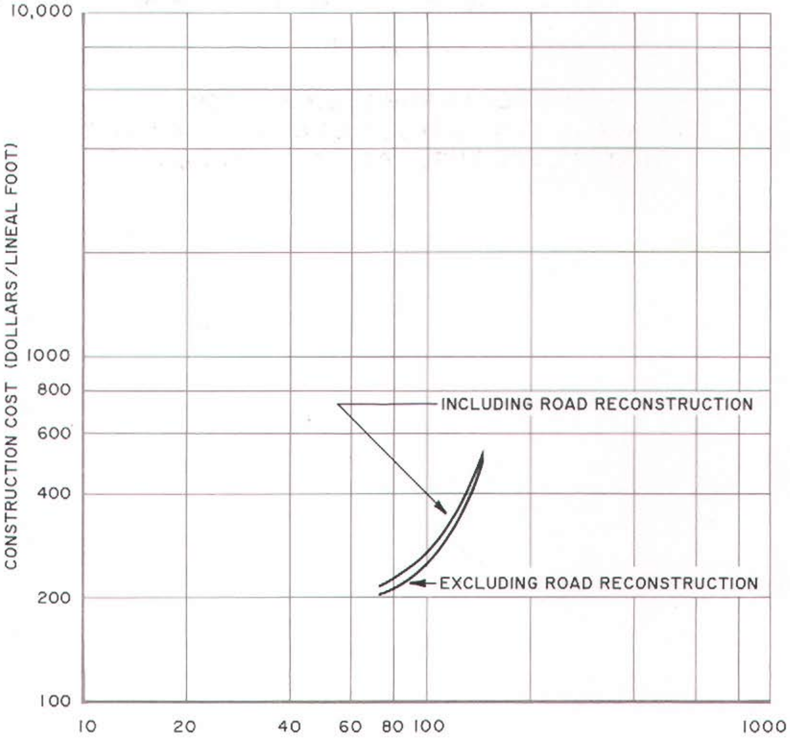


[a]ENR CCI = 4,725 (1989). Does not include easements, engineering, administration, and contingencies..

Source: SEWRPC.

Figure B-6

STRUCTURAL PLATE PIPE COST CURVES[a]

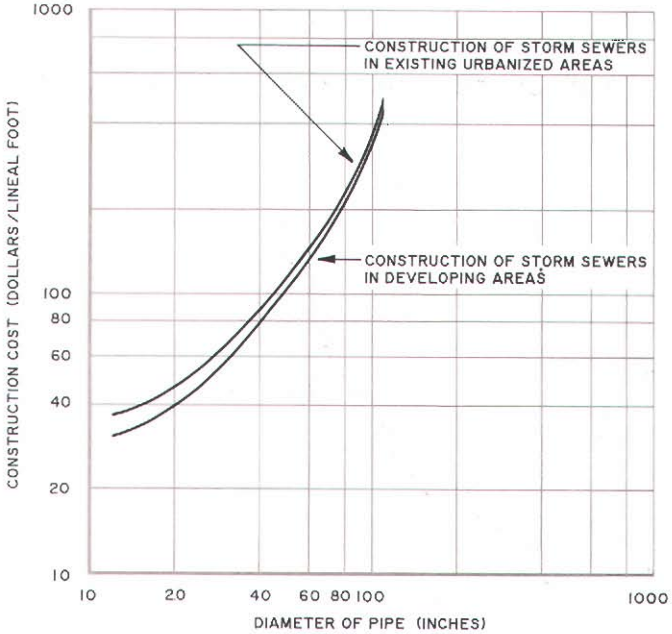


[a]ENR CCI = 4,725 (1989). Does not include easements, engineering, administration, and contingencies..

Source: SEWRPC.

Figure B-7

REINFORCED CONCRETE PIPE STORM SEWER COST CURVES[a,b]



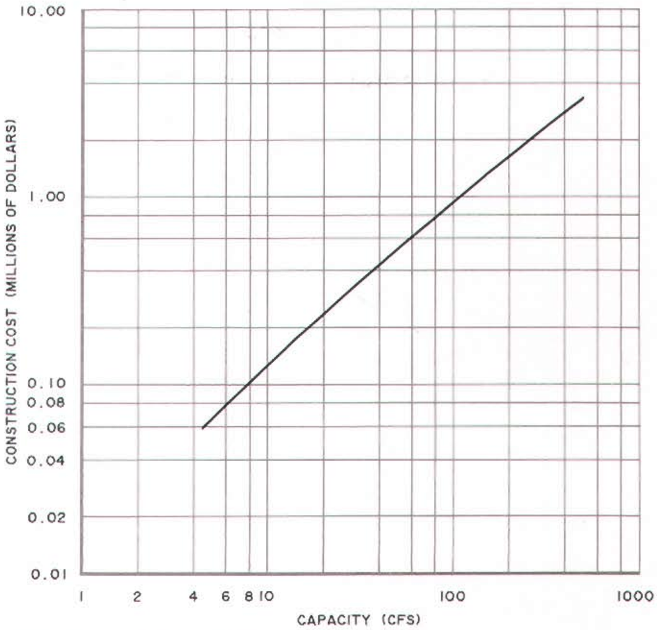
[a]ENR CCI = 4,725 (1989). Does not include easements, engineering, administration, and contingencies. Annual operation and maintenance costs equal $1,000 per mile for diameter greater than or equal to 36 inches and $2,100 per mile for diameter less than 36 inches.

[b]These curves are applicable for pipe invert depths up to 12 feet. For depths greater than 12 feet, site-specific cost adjustments should be made.

Source: Village of Menomonee Falls and SEWRPC.

Figure B-8

PUMPING STATION COST CURVES[a]



[a]ENR CCI = 4,725 (1989). Does not include land acquisition, engineering, administration, and contingencies. Annual operation and maintenance costs equal $6,300 per year.

Source: SEWRPC.

Table B-1

UNIT COSTS FOR CHANNEL MODIFICATION COMPONENTS

Component	Unit Cost[a]
Clearing and Grubbing	$3,700 per acre
Excavation	$3 to $20 per cubic yard[b]
Concrete	$170 per cubic yard
Riprap	$40 per cubic yard
Gabions	$105 per cubic yard
Landscaping	$3,600 per acre

[a] ENR CCI = 4,725 (1989). Annual channel maintenance cost = $2,100 per mile.

[b] Cost dependent on haul distance to disposal site, disposal site tipping fees, and whether excavated material includes toxic substances requiring special disposal methods.

Source: SEWRPC.

Table B-2

UNIT COSTS FOR BRIDGE REMOVAL AND REPLACEMENT

Type of Bridge	Unit Cost[a,b,c] (per square foot)
Street	$80
Pedestrian	90

[a] ENR CCI = 4,725 (1989).

[b] Based on bridge deck area including street, curbs, sidewalks, and parapets.

[c] Includes cost for contingencies, construction engineering, and administration. Add 10 percent for design engineering.

Source: SEWRPC.

Table B-3

UNIT COSTS FOR RAILWAY BRIDGE REMOVAL AND REPLACEMENT

Number of Tracks	Unit Cost[a] (per lineal foot of span)
1	$ 5,100
2	9,100
3	13,100

[a] ENR CCI = 4,725 (1989).

Source: SEWRPC.

Table B-4

UNIT COSTS FOR CONCRETE BOX CULVERTS

Culvert Size (feet)	Unit Cost[a,b] (per lineal foot)
4 x 2	$130
5 x 3	190
7 x 3	330
8 x 4	390
8 x 6	420
8 x 8	480
10 x 3	370
10 x 4	460
10 x 6	510
10 x 8	610
10 x 10	690
12 x 6	670
12 x 8	700
12 x 10	860
12 x 12	940
16 x 6	630

[a] ENR CCI = 4,725 (1989).

[b] Add $20 per lineal foot of pipe to account for road reconstruction.

Source: SEWRPC.

Table B-5

UNIT COSTS FOR CORRUGATED METAL PIPE ARCHES

Pipe Size, Span x Rise (inches)	Unit Cost[a] (per lineal foot)	
	Excluding Road Reconstruction	Including Road Reconstruction
36 x 22	$ 54	$ 64
43 x 27	63	73
50 x 31	69	79
58 x 36	83	93
65 x 40	104	114
72 x 44	110	120

[a]ENR CCI = 4,725 (1989).

Source: SEWRPC.

Table B-6

UNIT COSTS FOR STRUCTURAL PLATE PIPE ARCHES

Pipe Size, Span x Rise (inches)	Unit Cost[a] (per lineal foot)	
	Excluding Road Reconstruction	Including Road Reconstruction
73 x 55	$280	$290
84 x 61	300	310
98 x 69	340	350
114 x 77	410	420
131 x 85	500	510
148 x 93	540	560
161 x 101	600	620
178 x 109	640	660
190 x 118	700	730
199 x 121	720	750

[a]ENR CCI = 4,725 (1989).

Source: Dodge Guide and SEWRPC.

Table B-7

UNIT COSTS FOR REINFORCED CONCRETE PIPE ARCH (RCPA) AND HORIZONTAL ELLIPTICAL (HE) STORM SEWERS

Pipe Size, Span x Rise (inches)		Unit Cost[a] (per lineal foot)	
		Replacement of Existing Storm Sewers in Urbanized Areas	Construction of New Storm Sewers in Developing Areas
RCPA	HE		
22 x 14	23 x 14	$ 49	$ 42
29 x 18	30 x 19	62	54
36 x 23	38 x 24	80	72
44 x 27	45 x 29	89	80
51 x 31	53 x 34	110	100
58 x 36	60 x 38	133	123
65 x 40	68 x 43	153	141
73 x 45	76 x 48	183	171
--	83 x 53	200	188
88 x 54	91 x 58	235	223

[a]ENR CCI = 4,725 (1989).

Source: SEWRPC.

Table B-8

STRUCTURE FLOODPROOFING COSTS[a]

Structure Type	Cost per Structure
Single-Family Home	$5,500
Two-Family Residence	7,100
Industrial/Commercial Building with Basement	Market Value x (0.07 + 0.05 x height, in feet, of floodproofing above first floor)

[a]ENR CCI = 4,725 (1989). Costs include administration and contingencies.

Source: SEWRPC.

Table B-9

SINGLE-FAMILY HOME ELEVATION COSTS

Cost = $30,000 + $4,300 x Number of Feet Raised

[a]ENR CCI = 4,725 (1989). Costs include administration and contingencies.

Source: SEWRPC.

Table B-10

SINGLE-FAMILY HOME REMOVAL COSTS

Cost = $18,000 + Structure and Site Acquisition Cost

[a]All item Consumer Price Index (W) = 357.74 (1988).

Source: SEWRPC.

Appendix C

LILLY CREEK ENVIRONMENTAL ENHANCEMENT REPORT PREPARED BY BRW, INC.

Prepared for:	Village of Menomonee Falls, in cooperation with the Wisconsin DNR and SEWRPC
Prepared by:	BRW, Inc.

FINAL DRAFT

November 24, 1992

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INTRODUCTION

The flood control alternative proposed by SEWRPC involves the deepening of the Lilly Creek streambed by an average of four feet through reaches 4 through 8, less in reaches 2, 3 and 9. As a result of this channel modification, much of the existing bank vegetation, streambed substrate, and fish and wildlife habitat along the existing Lilly Creek channel would be lost or disturbed. However, these natural features can be restored and enhanced during and after channel modifications are made.

The following report describes and recommends several different methods which can be used to restore, create, or enhance natural environmental features in coordination with the proposed modifications to the Lilly Creek channel.

The different environmental enhancement and restoration methods typically used for this purpose are described in the first section of this report. Each of the methods have been evaluated for the value they could contribute to this specific project. The last section of the report provides recommendations and associated costs for the most valuable and feasible methods of environmental enhancement and restoration for each reach of Lilly Creek.

I. General Method Descriptions

It is understood that implementation of the proposed stream modifications for flood management purposes would reduce or eliminate the biological, aesthetic and recreational characteristics of the existing Lilly Creek corridor. It has been assumed that restoration measures will be required by the Wisconsin DNR for these impacts. These restoration methods can be divided into two forms, channel restoration and corridor enhancement.

There are many different ways to restore or enhance a disturbed environment. The methods described herein can be used to provide multiple uses. This multi-objective approach is focused on developing restoration and enhancement methods which compensate for the adverse impacts on the biological, recreational and aquatic resources of Lilly Creek.

Emphasis has been placed on restoration of the in-stream components of the creek, such as recreating pools/riffles and replanting channel vegetation. Improvements to the existing streambed condition can also be made by adding more pool/riffle systems, adding a varied substrate, and developing a continuous low flow channel for fish passage. Additional enhancement methods, such as preservation of adjacent terrestrial habitat and resources, development of recreational and open space opportunities for Village residents, and enhancement of the stream and neighborhood aesthetic values of the project, were also considered.

Each of the methods described in this report could be used alone or in conjunction with one or more other methods to achieve the desired results. Some examples of how these methods can provide multiple functions include; the creation of wetland basins for storm water storage in addition to providing wildlife/fisheries habitat, channel cleaning to increase aesthetic value which also improves flood flow potential of the channel, and revegetation or preservation of existing vegetation to increase aesthetics while also reducing bank erosion.

Restoration Methods

1. Low Flow Channel

A low flow channel is a subchannel constructed inside a larger flood control channel which concentrates flow during low to moderate flow conditions. The emphasis of the proposed restoration and enhancement will be on the improvement of the instream fish habitat of which the most important feature is to maintain a low flow channel along the entire length of the creek, which allows for continuous fish passage. Additionally, creating resting pools for aquatic life during low flow periods is also important.

In addition to other enhancement methods, erosion control during the channel modification process will be key to the success of the environmental enhancement. The use of riprap is one of the most common methods of erosion control in stream areas. The use of gabions in areas of severe erosion is also common (Figure 2).

TABLE 1: Multi-objective Restoration and Environmental Enhancement Methods

Method	Fishery/ Wildlife	Recreation /Aesthetic	Flood Control	Erosion Control
Restoration Methods				
1) Low Flow Channel	X	X		X
2) Bank Preservation	X	X	X	X
3) Pools/Riffles	X			X
4) Revegetation (Lower)	X	X		X
5) Substrate Enhance	X	X		X
6) Meandering	X	X		X
Enhancement Methods				
7) Create Wetland	X	X	X	X
8) Habitat Preserve	X	X	X	
9) Recreation		X		
10) Channel Cleaning		X	X	
11) Revegetation (Upper)	X	X		X

Each of the restoration methods can be used in conjunction with the low flow channel to increase its fisheries value, as well as improve erosion control and aesthetic value.

2. Bank Preservation

Limiting channel modification impacts to one of the existing creek banks, leaving the opposite bank undisturbed, reduces the potential for erosion as well as preserve existing fish and wildlife habitat within the creek corridor. It is preferable to preserve the west bank to maintain vegetation which shades the stream. Vegetation on the working bank should also be left intact to the extent practicable.

3. Pools and Riffles

This instream habitat modification creates diversity within the creek channel by creating micro-environments within the stream that are needed by different fish species to fulfill various life requirements. These are to be constructed within the low flow channel. Series of Pool/Riffle systems should be placed strategically to coordinate with outfall structures, meanders, quick changes in elevation, and in other potentially erodible areas. A series should consists of two or more pools and riffles spaced with 5 to 7 channel widths between pools (Figure 3).

4. Revegetation (lower bank)

The main purpose of this method is to control low flow channel erosion. However, it can also enhance wildlife and fish habitat, recreation and aesthetics. This is done through the types and size of vegetation used, and where it is located in relation to the creek and surrounding land uses. Natural vegetation in and adjacent to the creek channel can provide important fish and wildlife habitat, reduce erosion potential, make the creek more aesthetically attractive and increase its value for recreation.

In order for revegetation to promote soil stabilization, disturbed areas should be seeded immediately after the disturbance is completed. Additional landscaping can provide fish and wildlife benefits. By planting trees or shrubs near the creek channel, shade is created for the fishery. Plants that produce abundant seeds or fruits, or nesting sites, will also support different species of wildlife (see Table 2 and Figure 4).

5. Substrate Enhancement

The existing substrate of the creek along much of its length is silty with some scattered rubble. The substrate can be enhanced by randomly adding gravel, cobbles and larger rocks to the creek bed. This will add diversity to the substrate and improve the fish habitat (Figure 2).

TABLE 2: Recommended Plant List

Common Name		Latin Name
Wetland Seed Mix		
A.	Switchgrass	Panicum virgatum
	Big Bluestem	Andropogon gerardi
	Joe-Pye Weed	Eupatorium maculatum
	Boneset	Eupatorium perfoliatum
	New England Aster	Aster novae-anglicae
	Blue Vervain	Verbena hastata
B.	Canada Bluejoint grass	Calamagrostis canadensis
	Fox Sedge	Carex vulpinoidea
	Dark Green Bulrush	Scirpus atrovirens
	Marsh Milkweed	Asclepias incarnata
	Boneset	Eupatorium perfoliatum
C.	Cattails	Typha latifolia
	Arrowhead	Sagittaria latifolia
	Pickerelweed	Pontederia cordata
Lower Bank Mix		
D.	Prairie Cordgrass	Spartina pectinata
	Canada Bluejoint grass	Calamagrostis canadensis
	Big Bluestem	Andropogon gerardi
	Switchgrass	Panicum vergatum
Upper Bank Mix		
E.	Birdsfoot Trefoil	Lotus corniculatus
	Cider Milkvetch	Astragalus spp.
	Bluegrass var.	Poa spp.
	Timothy	Pluem pratense
Landscaped (Shrubs and Trees)		
F.	Red-osier Dogwood	Cornus stolonifera
	Common Elderberry	Sambucus canadensis
	Nannyberry Viburnum	Viburnum lentago
	Staghorn Sumac	Rhus typhina
	River Birch	Betula nigra
	Silver Maple	Acer saccharinum
	Spruce spp.	Picea spp.

6. Channel Meanders

Natural creek channels curve and bend as a function of changes in substrate, stream velocity, and other factors. Creating meanders within a channel is aimed at recreating channel diversity (Figure 2).

Environmental Enhancement Methods

7. Creating Wetlands

Wetlands can provide all of the characteristics and functions that the DNR requests be restored to the creek channel after flood control modifications; wildlife and fish habitat, aesthetics, recreation, improved hydraulic capacity, and erosion control.

The creation of off-line wetland systems would increase the upstream storage capacity of Lilly Creek while at the same time providing aesthetic and wildlife habitat value to the community. The three main goals of creating wetland systems are:

- Provide additional mitigation of flood flows through detention storage.
- Provide biofiltration in wetlands to enhance instream water quality.
- Couple with adjacent woodland preservation to provide maximum habitat value.

In order to achieve these goals, the wetland design should include the following general components:

1) Meandering, irregular edges to provide maximum edge habitat and create a natural image on the landscape.
2) Adequate topographic buffer between wetland and adjacent properties to avoid creating additional flooding problems.
3) Shallow side slopes to allow growth of emergent vegetation (Figure 5).

8. Habitat Preservation

Preserving adjacent habitat, especially within the creek floodplain, in its existing condition will enhance the wildlife habitat value of the creek corridor as surrounding areas undergo development, as well as protect the existing floodplain area from further development.

9. Recreational Amenities

Opportunities for public pedestrian and bicycle trails were identified after reviewing the existing development, the Village's Outdoor Park and Recreation Plan, and the hydrologic plan for the creek. The following design criteria were developed (Figure 6):

1) Screening or sight barriers may be necessary between the trail and private properties.
2) Connect trail to existing and planned future trail systems.
3) Picnic and play areas should be incorporated near residential areas.
4) Design trail in accordance with Recreation Plan.

10. Channel Cleaning

The existing channel has had grass clippings, leaves, concrete rubble, bricks, branches and other debris placed within its banks. This debris can cause flood flow problems, decrease water quality, and reduce the aesthetic value of the creek corridor. Removal of this debris from all ten reaches will enhance flood flows, water quality, and visual integrity.

Debris removal should be limited to man-made debris and woody vegetation which is shown to have significant impacts on flood conveyance or bank erosion. Removal of "hydraulic significant" woody vegetation should be based on a site specific examination by a hydraulic engineer and fish/wildlife biologist to maintain maximum habitat features while providing easy flood water conveyance.

11. Revegetation (upper bank)

The upper banks of the channel must be revegetated with erosion resistant plant material to stabilize the steep slopes and protect them from the erosive forces of the high flow condition. Tree and shrub vegetation should be located at the top of the bank, rather than the upper bank, to minimize flow resistance within the channel.

II. Method Feasibility

1. Limitations

Through meetings and discussions with staff from the Village of Menomonee Falls, Wisconsin DNR, and SEWRPC, it was determined that some of the previously described methods would not be appropriate for use along the Lilly Creek channel. Several were ruled out due to conflicts with the proposed channel modifications and flood control objectives.

Creating Wetlands

Environmental enhancement methods should not restrict the flow of the creek, nor significantly affect the flood mitigation plan. Due to channel modifications for flood control, as proposed by SEWRPC, the placement of control structures within the creek channel to create on-line wetlands were perceived as negative. Additional structures within the creek could impede the flow of the creek during flood stage, they require maintenance, and are expensive to construct. Therefore, the addition of control structures to the channel as a means to create on-line wetland areas to provide flood storage, plus wildlife and fisheries benefits, would not be feasible.

However, the biological condition of the creek can be improved without the use of control structures. Wetlands can be created off-line or without the use of control structures. These off-line wetlands would simply become overflow areas within the floodplain that would be flooded during frequent rain events. Irregular bottom contours within the wetland basin would allow some standing water to remain within the wetland after flood levels subside in the creek.

Channel Meanders

The affects of creating channel meanders were also discussed with staff. Designing meanders into a modified channel would add diversity to the channel and reduce the flow of the channel. However, this was also perceived as a negative because the soils along Lilly Creek tend to be highly erodible when exposed, and during high flow periods these meanders will increase erosion potential by increasing flow velocity around the meander curves. This in turn will have a negative impact on the fishery and water quality of the creek.

It was concluded that meanders should only be used along Lilly Creek when used in conjunction with pool and riffle systems, which will help control erosional forces, and with use of notched guide structures that will guide and control the flow of the low flow channel, keeping it from cutting into the banks.

Use of Riprap

The most common factor in these restoration and enhancement methods is the importance of erosion control. Regardless of the objectives behind the different methods, if erosion is not properly controlled, the effectiveness of these methods is drastically reduced.

Unlimited use of riprap along the channel, however, would also have negative affects on the creek. Extensive use of riprap can be very unattractive and inhibit wildlife from using the creek banks. Care must also be taken when placing large riprap material along the bottom of the channel because the stream may become "lost" within the interstitial spaces of the material.

Random placement of riprap, on the other hand, is good for erosion control along the lower banks of the creek. Therefore, random placement of riprap should be used and limited to areas of extreme erodibility. For example in areas around storm sewer outlets and culvert, with the meander guide structures, at significant grade changes, and at the outside of meander curves.

2. Costs

Unit cost estimates have been developed for each of the methods described herein. These estimates were compiled through data collected from SEWRPC (1,10), the Village (8,9) and BRW. Costs do not include engineering, administration or contingency fees.

TABLE 3: Unit Costs for Restoration/Enhancement Methods

Restoration Method	Unit Cost
1) Low Flow Channel	$ 120/cu yd per gabion
	$ 45/cu yd per riprap
2) Bank Preservation	No cost
3) Pool/Riffle	
substrate	$ 50/ton per gravel/river rock
excavation	$ 50/pool for excavation
4) Revegetation (lower)	
lower bank seed	$1,000/acre
5) Substrate Enhancement	50/ton per gravel/river rock
6) Meander guides	$ 375/guide structure
Enhancement Method	Unit Cost
7) Wetland Creation	
wetland seed mixes	$ 900/acre
8) Habitat Preservation	$ 250/wetland acre
	$ 500/forested wetland
	$1,000/upland acre
9) Recreational Amenities	
trail	$ 7/linear foot
play equipment	$10,000 per unit
softball diamond	$10,000 per unit
picnic table/grill	$ 200 per unit
2 volleyball court	$ 1,000 per 2
Soccer/football	$ 2,000 per unit
Parking lot	$ 2,000 per unit
10) Channel Cleaning	$2,100 per mile
11) Revegetation (upper)	
upper bank seed	$ 700/acre
top of bank	$ 350/acre

Source: SEWRPC, Village, and BRW

III. Recommendations

Lilly Creek has been divided into 10 different reaches. Reaches 1 and 10 will not undergo any channel modifications and therefore were not included in the study area for replacement or enhancement measures.

The recommendations that follow are considered what will be necessary to regain the biological features of Lilly Creek that will be lost to flood control modification, and what will be needed to provide improved fisheries habitat and enhanced aesthetic values to the creek and community.

Since the proposed channel modifications are necessary for flood control, it is recommended that the opportunity be taken at this time to improve the existing fishery in conjunction with the proposed channel modifications.

Because the existing condition of Lilly Creek is one that has been previously channelled and altered, the amount of environmental enhancement that occurs will be a function of costs.

Reach 2

The primary fishery and spawning areas within Lilly Creek currently lie north of Good Hope Road in the first three reaches upstream of the confluence with the Menomonee River. SEWRPC's preferred flood mitigation plan would require channel modifications within reaches 2 and 3.

In order to maintain the existing fishery within reach 1, as well as the rest of the creek, it is recommended that channel modifications in reach 2, as in reach 1, also be avoided to protect and preserve the existing fishery.

Other enhancement measures that are recommended for this reach focus on the continued protection of the existing resource and stabilization of eroding streambanks. Erosion control is required in the north stretch of this reach north of Appleton Avenue. Gabions are suggested where steep banks are being undercut by the creek. Riprap is recommended along the outside meander curves south of Appleton Avenue. This reach also needs to have existing rubble, trash, and other debris removed from the channel to increase aesthetics. Fish habitat may be enhanced by randomly adding gravel to the streambed and in existing pool areas. Figures 7 and 8 illustrate these recommendations.

In reaches 3 through 9, there are five main enhancement measures that can be used within each reach. Those are; 1) create a low flow channel for fish passage, 2) limit channel impacts to one bank, 3) create pool/riffle systems within stream, 4) revegetate banks to reduce erosion potential and create upland habitats, and 5) provide random substrate enhancement within the streambed.

The extent to which these measures can be carried out will vary with the different limits of each reach. A brief description of which methods could be used follows. The recommended number and locations these methods should be used are identified in Figures 9 through 22.

Reach 3

The west bank of this reach can be preserved by deepening and widening the creek channel to the east. The east bank should then be revegetated with grasses on the lower bank to stabilize the soil, and patches of trees and shrubs on the top of the bank to replace the shading lost. Pools and riffles can also be created within low flow channel to mimic the existing channel. Random placement of gravel can be added to enhance the bottom substrate.

Reach 4

Reach 4 is a fully developed residential neighborhood wherein the creek has already been channelized between a pair of parallel roadways for much of its length. The existing channel has limited fish or wildlife habitat value due to roads. Preserving one bank in this reach does not provide enough benefit to make it feasible due to the constraints of the residential areas.

The important enhancement measures that should be implemented within this reach include providing shade over the channel with patches of trees and shrubs on the top of the bank and with tall grasses on the lower bank. Pools and riffles can also be incorporated into the low flow channel in several locations. Again, gravel should also be randomly placed along the channel bottom. This reach could be further enhanced through construction of a trail system to connect this reach with more scenic reaches.

Reach 5

In addition to the 5 main enhancement measures described for previous reaches, overflow wetland storage and picnic or play areas can also be created along this reach. The undeveloped area west of the creek and east of Lilly Road can be excavated down to create dry detention area that can be used as picnic grounds, play fields, volleyball courts and open space. A trail system can also be incorporated into this reach to connect it with other reaches.

Reach 6

The west bank of this reach should be preserved, if possible. A low flow channel with pools, riffles and gravel substrate should be added in this reach. Bank vegetation should shade at least fifty percent of the channel. Riprap will be needed at the new channel outlet under the Lilly Road/Mill Road intersection.

In addition to these measures, another park/open space area can be created west of the creek and Lilly Road in an existing open field. This play area can be connected to the play areas in reach 5, through reaches 3 and 4 to the St. Anthony's school on Appleton Avenue via a trail system.

Reach 7

As with reach 5, off-line wetland storage can be created within this reach in addition to the five main enhancement measures. The open field in the northwest portion of reach 7 can be excavated to create a shallow wetland basin.

The west bank of the creek, adjacent to the created wetland, could be preserved with the exception of a small area that would be excavated to allow overflow into the wetland from the creek during high flow periods.

South of this wetland area, the east bank of the channel would be preserved to avoid additional impacts on adjacent residences. The forested area west of the creek would have some trees removed adjacent to the existing channel. The remaining wooded vegetation should be preserved as important floodplain area and wildlife habitat.

Meanders, with pools, riffles and riprap, will be needed in areas where bank preservation switches from west to east.

Reach 8

This reach, as with reach 4, is limited in its fish and wildlife values. This is largely as a result of the adjacent industrial development. Large amounts of debris have been placed within the creek channel, affecting flows and aesthetics. This material should be removed during the channel deepening.

Banks preservation is not practical in this reach due to the existing land uses. However, other measures can be used, including placing riprap at bridge locations, adding pools/riffles, and landscaping for visual barriers at the tops of the banks.

Reach 9

The west channel bank should be preserve within this reach. As in reach 7, off-line wetland storage and habitat preservation should also be implemented within this reach (see Figure 21), as well as the additional of pools/riffles and substrate enhancement.

TABLE 4: Method Costs per Reach

Reach	Method	Number of Units	Subtotal	Total
2	1	3 gabion;5 riprap	$ 585	
	5	10 ton gravel/cobble	500	
	9	200 ft trail	1,400	
	10	0.3 miles	640	$ 3,125
3	1	5 riprap units	225	
	3	10 pool/riffle series	1,200	
	4	0.6 acres	600	
	5	6 tons gravel/cobble	300	
	6	3 meander guides	1,125	
	9	1600 ft trail	11,200	
	11	1.7 acres	1,190	$15,840
4	1	8 riprap units	360	
	3	12 pool/riffle series	1,800	
	4	1.3 acres	1,300	
	5	10 tons	500	
	6	3 meander guides	1,125	
	9	3000 ft trail	21,000	
	11	3.9 acres	2,730	$28,815

5	1	2 riprap units	90	
	3	11 pool/riffle series	1,650	
	4	0.8 acres	800	
	5	8 tons	400	
	9	1800 ft trail,land	70,600	
		Parking, volleyball, picnic/grill	4,600	
	11	2.5 acres upper,	1,750	
		12.8 acres top	4,480	$84,370
6	1	3 riprap units	135	
	3	2 pool/riffle series	300	
	4	0.4 acres	400	
	5	4 tons	200	
	9	1400 ft trail	9,800	
		4 acres upland, 2 softball, 4 picnic, parking, playground	41,800	
	11	1.2 acres	840	$53,475
7	1	7 riprap units	315	
	3	7 pools/riffle series	1,050	
	4	1.1 acres	1,100	
	5	8 tons	400	
	6	2 meander guides	750	
	7	5.2 acres upland	15,000	
		5.2 acres seed	4,680	
	8	5.5 acres floodplain	16,000	
	11	3.2 acres	2,240	$41,535
8	1	6 riprap units	270	
	3	6 pools/riffle series	900	
	4	0.6 acres	600	
	5	6 tons	300	
	10	0.3 miles	640	
	11	1.9 acres	1,330	$ 4,040
9	1	2 riprap units	90	
	3	3 pool/riffle series	450	
	4	0.3 acres	300	
	5	4 tons	200	
	7	3.5 ac seed	3,150	
		3.5 ac upland	4,100	
	8	2 acres floodplain	2,300	
	11	1 acre	700	$11,290
Total of proposed measures			$	242,490

* land costs based on assessed values (SEWRPC)

** costs do not include engineering, administration, or contingency fees.

TABLE 5: Summary of Proposed Method Costs

	Method	Cost
1	Riprap	$ 2,070
2	Bank Preservation	0
3	Pool/Riffle	7,350
4	Revegetation,lower	5,100
5	Bottom Substrate	2,800
6	Meander Guides	3,000
7	Wetland Creation	7,830
	Land	19,100
8	Habitat Preservation	18,300
9	Trails	56,000
	Amenities	37,400
	Land	67,000
10	Channel cleaning	1,280
11	Revegetation,upper	15,260
	Total	$ 242,490

To summarize, there are five general restoration measures that should be implemented within reaches 3 through 9 including creating a low flow channel, preserving one bank from modification, create pools and riffles, revegetate low flow channel banks with grasses, randomly add gravel to stream bottom, and provide meander guides where needed to prevent bank erosion. Other enhancement

TABLE 6: Summary of Proposed Methods per Reach

	Reach Number									
Method	1	2	3	4	5	6	7	8	9	10
1		X	X	X	X	X	X	X	X	
2		X	X				X		X	
3			X	X	X	X	X	X	X	
4			X	X	X	X	X	X	X	
5		X	X	X	X	X	X	X	X	
6			X	X			X			
7							X		X	
8							X		X	
9		X	X	X	X	X				
10		X						X		
11			X	X	X	X	X	X	X	

measures are incorporated where appropriate and feasible including creating wetlands, recreation areas, preserving significant upland vegetation within the floodplain, cleaning debris from channel, and providing vegetation for wildlife and aesthetics.

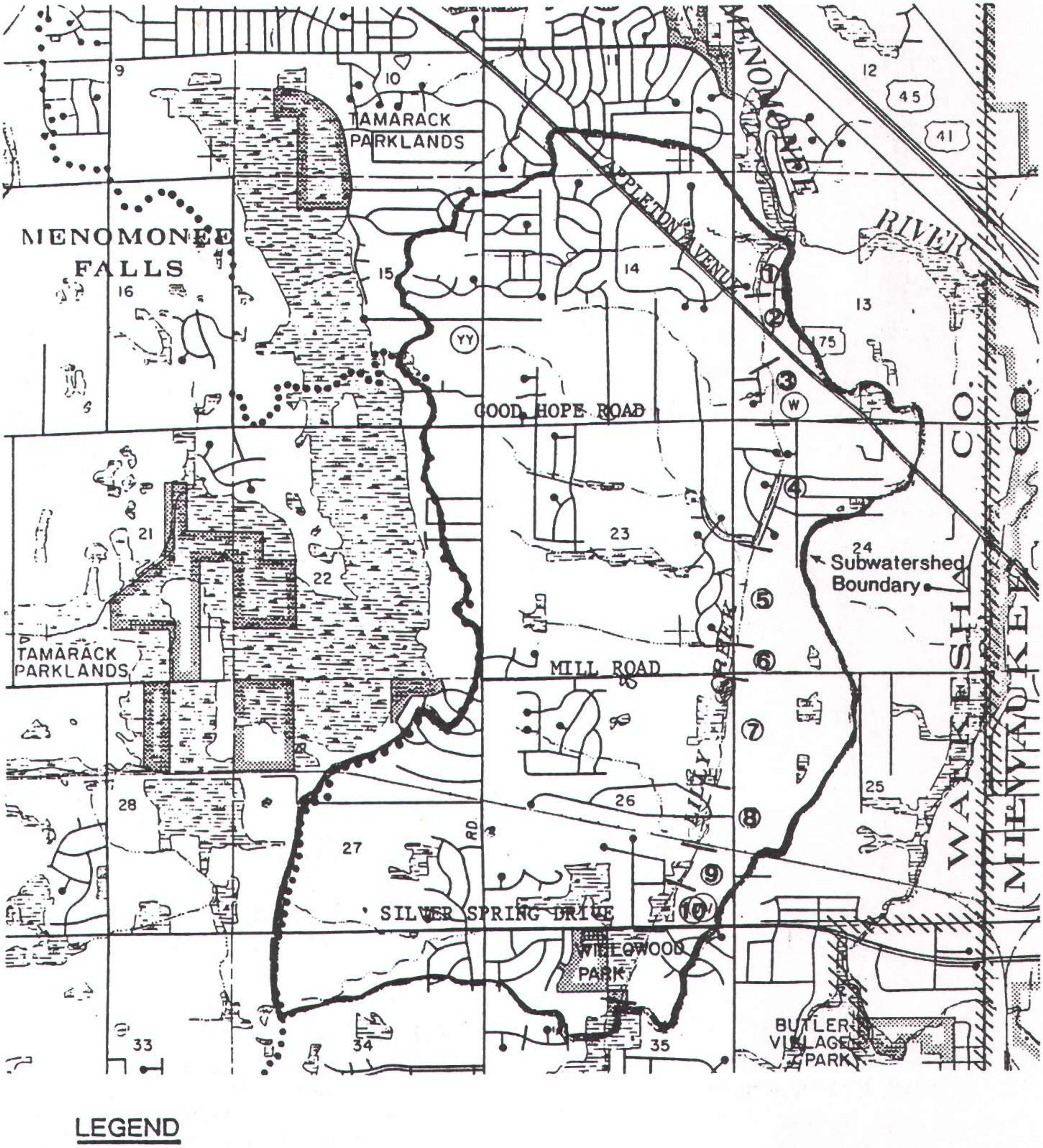
Reach 2 should be excluded from all channel modifications. This reach along with reach 8 should also have unsightly debris removed from channel.

A trail can be incorporated into the Lilly Creek corridor between Mill Road and Appleton Avenue connecting St. Anthony school with the new recreational areas in reaches 5 and 6. This method, as well as the proposed park amenities, are relatively costly compared to the other methods. The Village may eliminate the use of these two methods based on cost, as they do not directly affect the creek channel. The land area in reach 5 would, however, need to be acquired for overflow storage area.

The remaining methods proposed are recommended to meet the objectives of the Village, the DNR and SEWRPC at a reasonable cost. These recommendations may be modified to meet the Village budget and divided into appropriate phases.

Figure 1

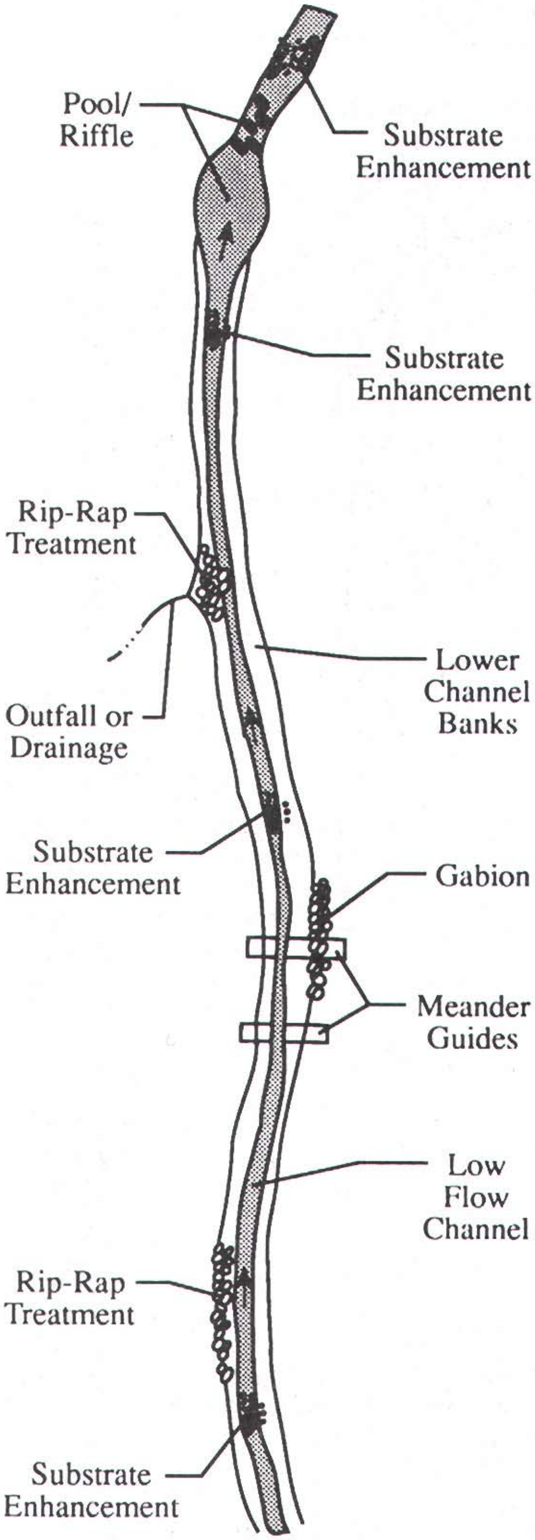
LILLY CREEK STREAM REACHES FOR WHICH ENVIRONMENTAL ENHANCEMENT MEASURES ARE TO BE PROVIDED



LEGEND

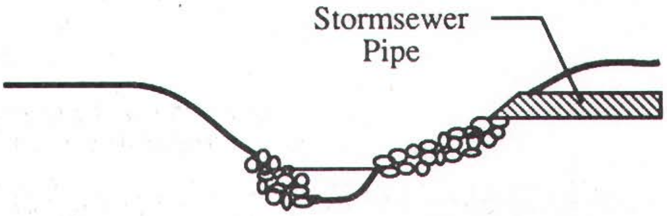
① Stream Reach Number

Source: SEWRPC.



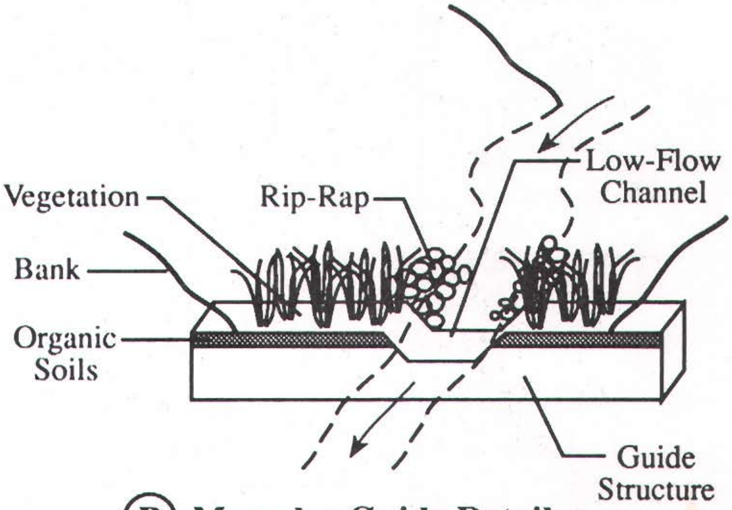
Ⓓ Substrate Enhancement

- Random Placement of Clean Gravel / Cobbel Material to Low Flow Channel Bottom.



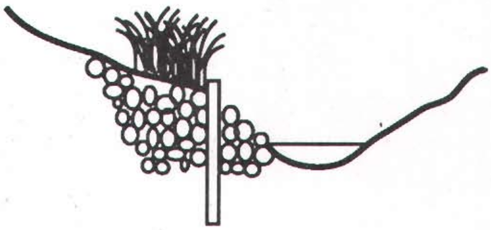
Ⓐ Rip-Rap Detail

- Place at Outfall Structures and Low Flow Channel Banks That May Have Erosion Problems.



Ⓑ Meander Guide Detail

- Notched Structure Reaching from Bank to Bank Which Guides Low Flow Channel.



Ⓒ Gabion Detail

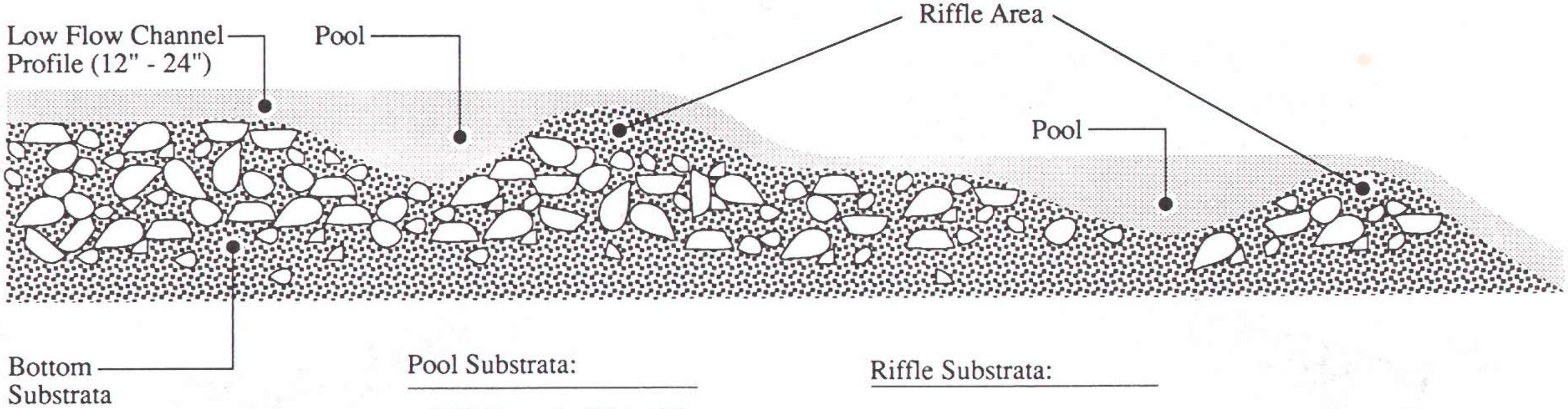
- Rip-Rap Contained by Wire Mesh to Correct Severe Erosion Problems in Steep Sloped Areas.

BRW

Lilly Creek Restoration & Enhancement Measures

Menomonee Falls

Figure 2
Erosion Control Detail for Low Flow Channel



Pool Substrata:

50% Gravel (1" to 3")
50% Cobble (3" to 10")

Pool Length:

3' to 9'

Pool Width:

1' to 10'

Pool Depth:

24" to 48"

Riffle Substrata:

50% Gravel
50% Cobble

Riffle Length:

1' to 6'

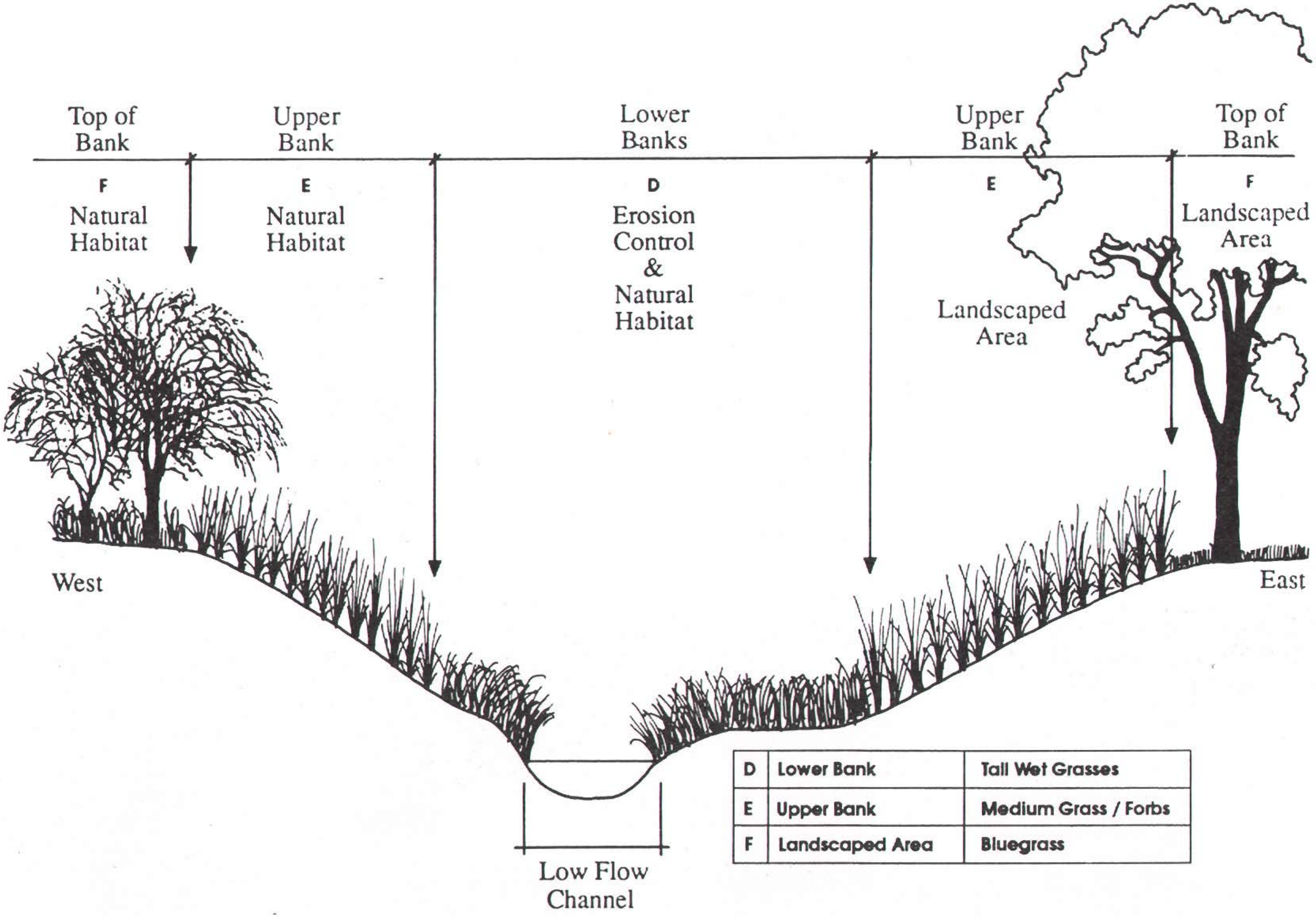
Riffle Width:

1' to 6'

Riffile / Pool Ratio:

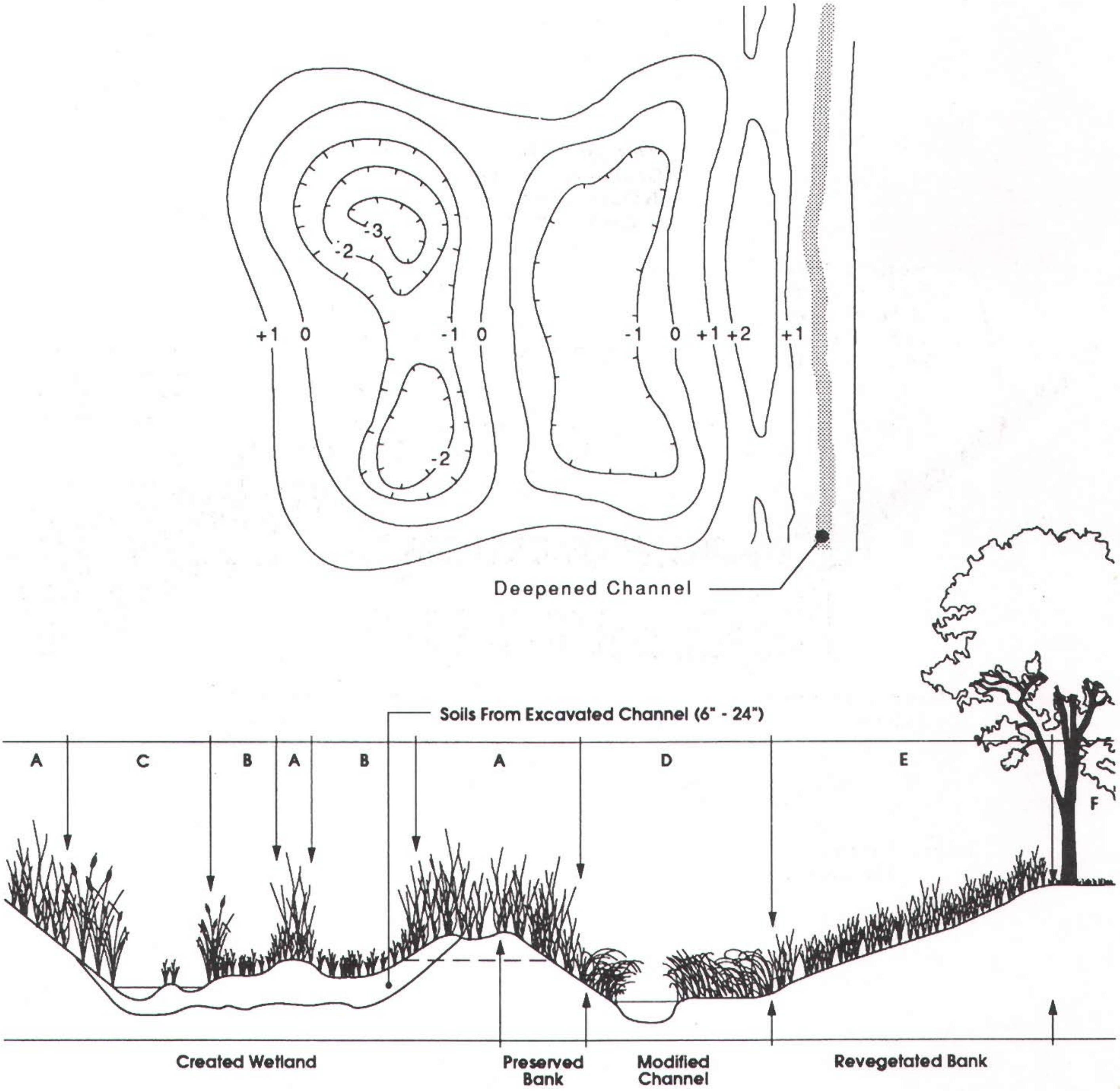
Approximately 15'-150' Between Riffiles

Figure 3 Detail of Typical Pool and Riffle Construction



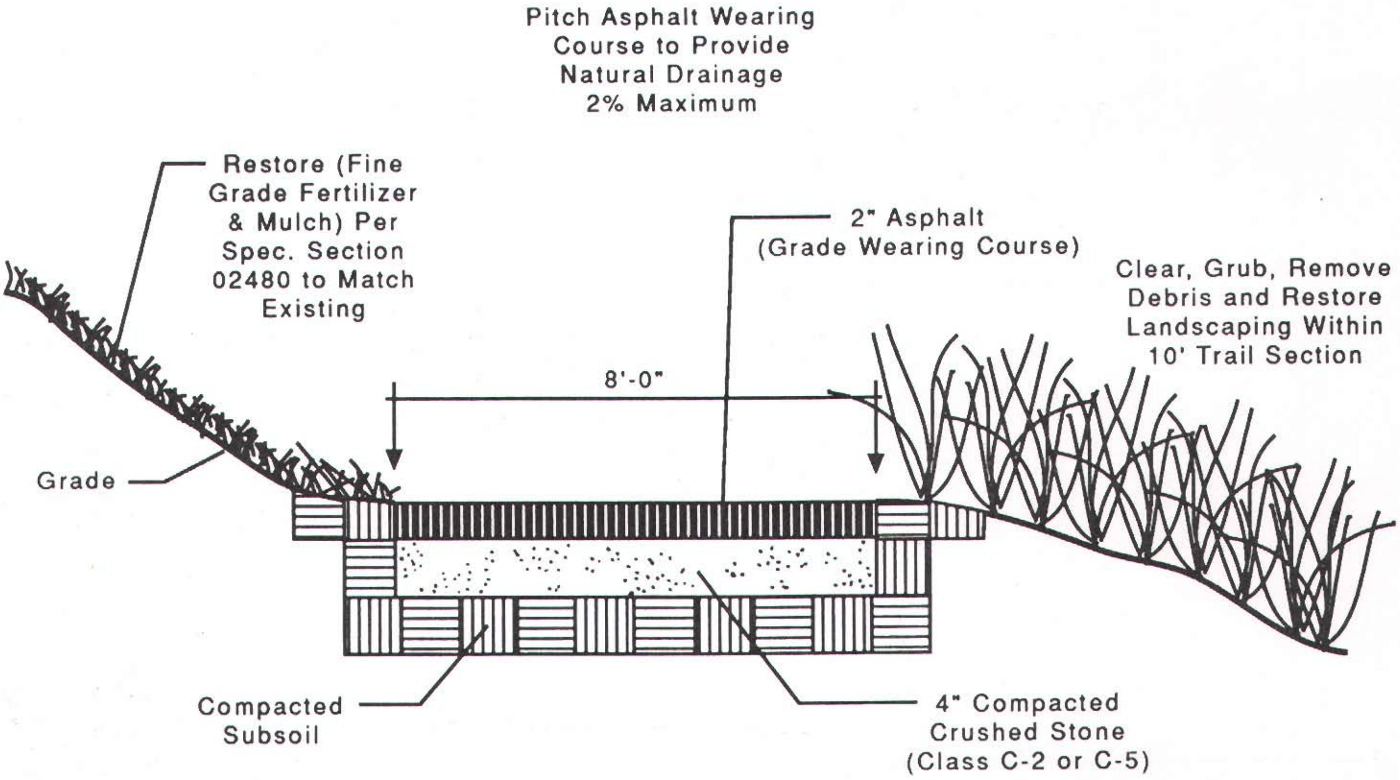
D	Lower Bank	Tall Wet Grasses
E	Upper Bank	Medium Grass / Forbs
F	Landscaped Area	Bluegrass

Figure 4
Revegetation Detail



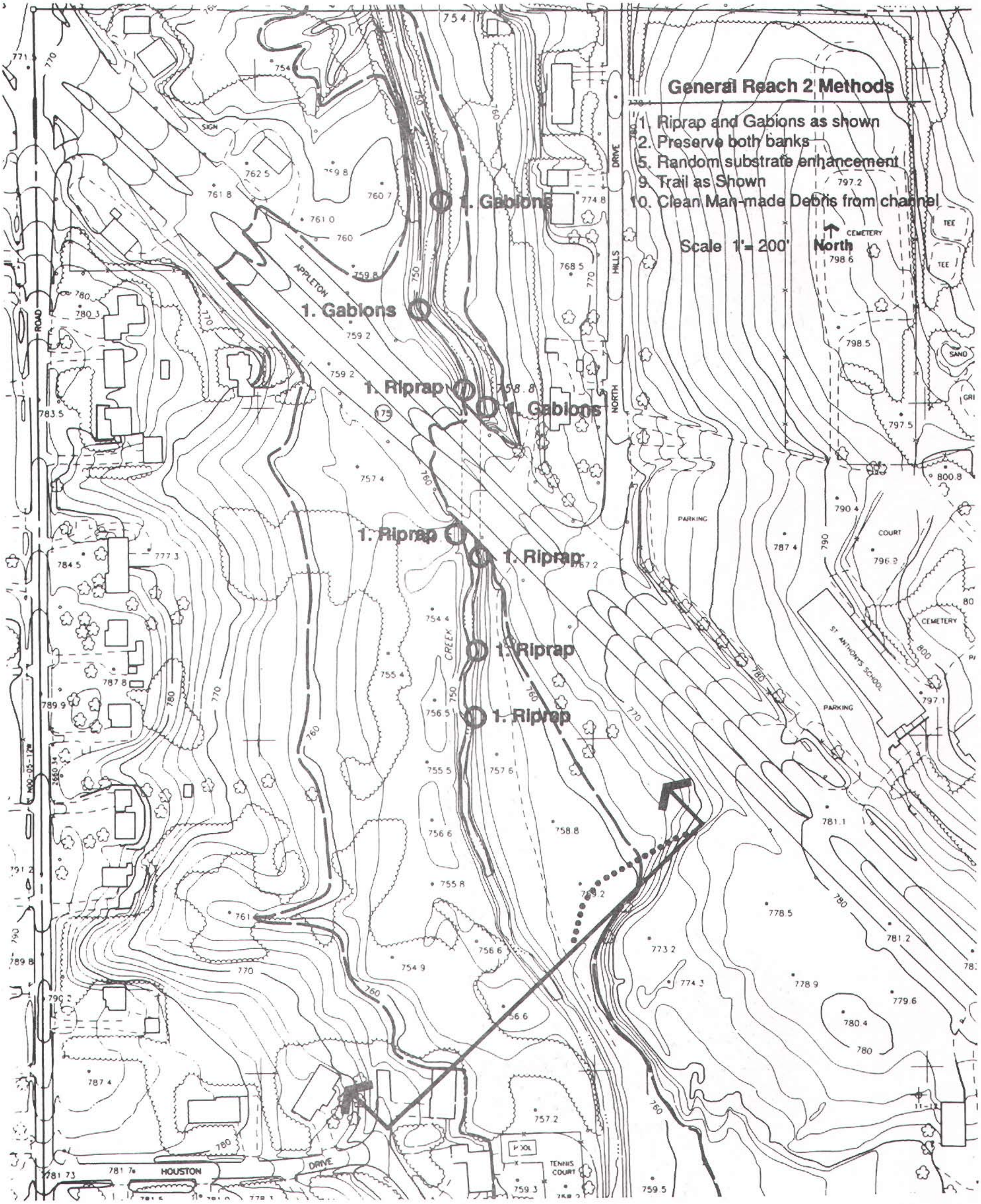
A	Wetland Vegetation	Bluestem / Switchgrass
B		Wet Meadow
C		Cattails / Rushes
D	Lower Bank	Tall Wet Grasses
E	Upper Bank	Medium Grass / Forbs
F	Landscaped Area	Bluegrass

Figure 5
Detail of Typical Wetland Creation & Revegetation



Source: Village of
Menomonee Falls

Figure 6
Detail of Typical Multi-Purpose Trail Section



General Reach 2 Methods

1. Riprap and Gabions as shown
2. Preserve both banks
5. Random substrate enhancement
9. Trail as Shown
10. Clean Man-made Debris from channel

Scale 1'= 200' North

B R W

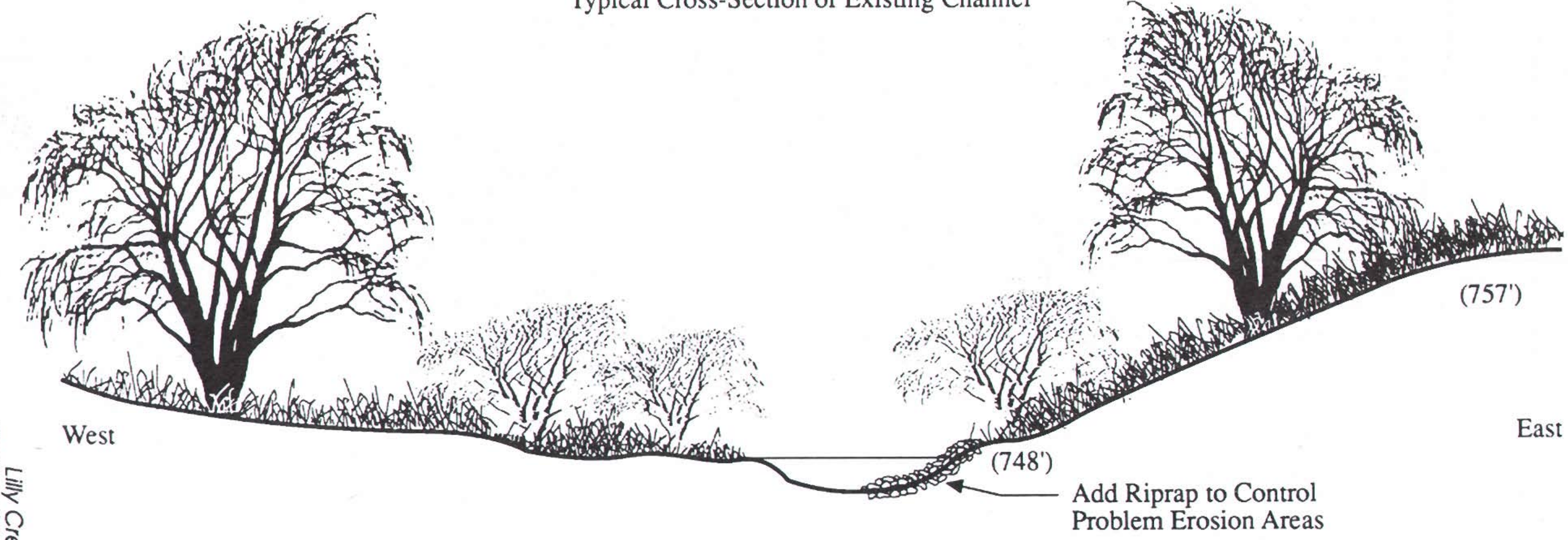
Lilly Creek
Restoration &
Enhancement
Measures

Menomonee Falls

Figure 7
Reach 2
Plan View

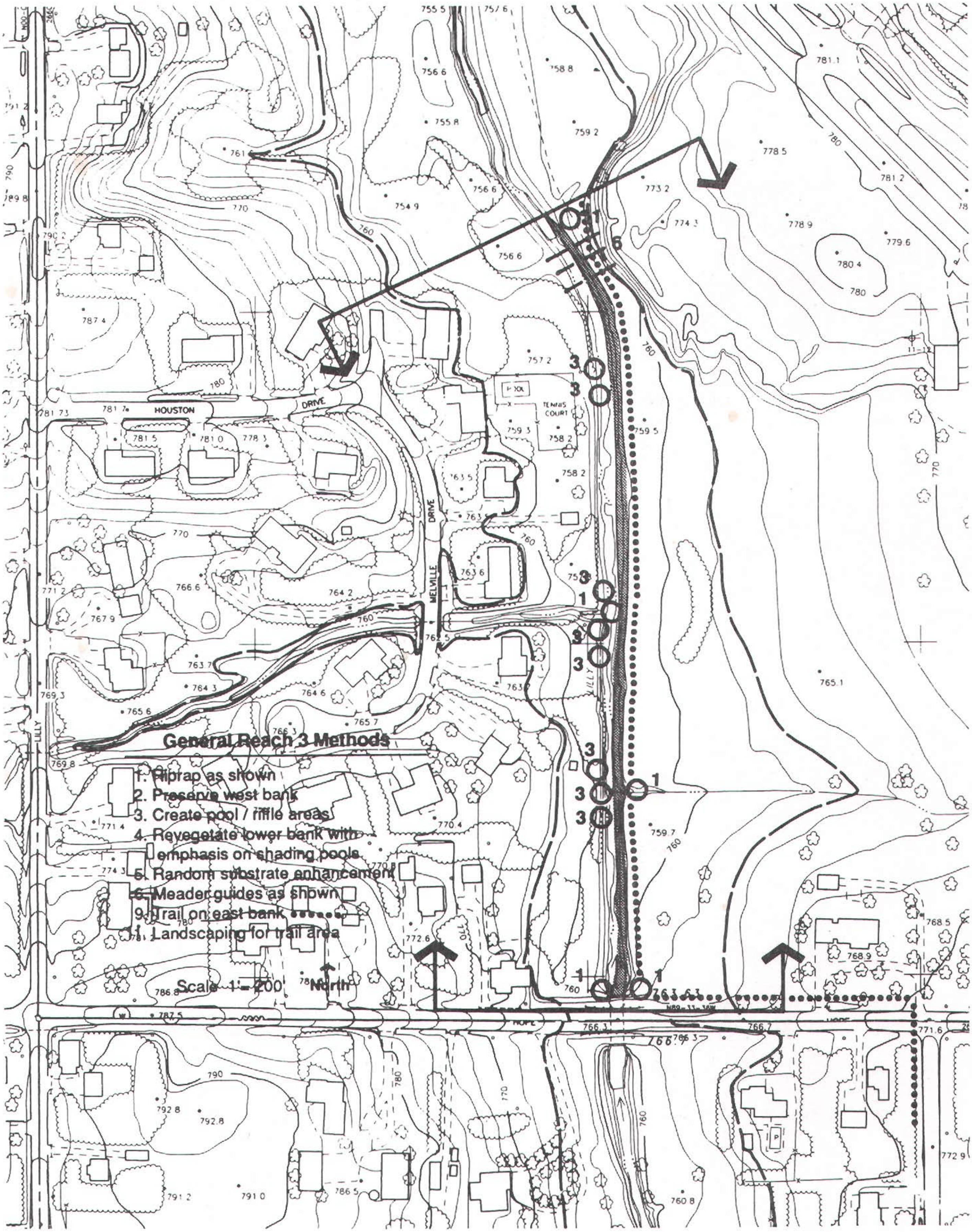
Reach #2

Typical Cross-Section of Existing Channel



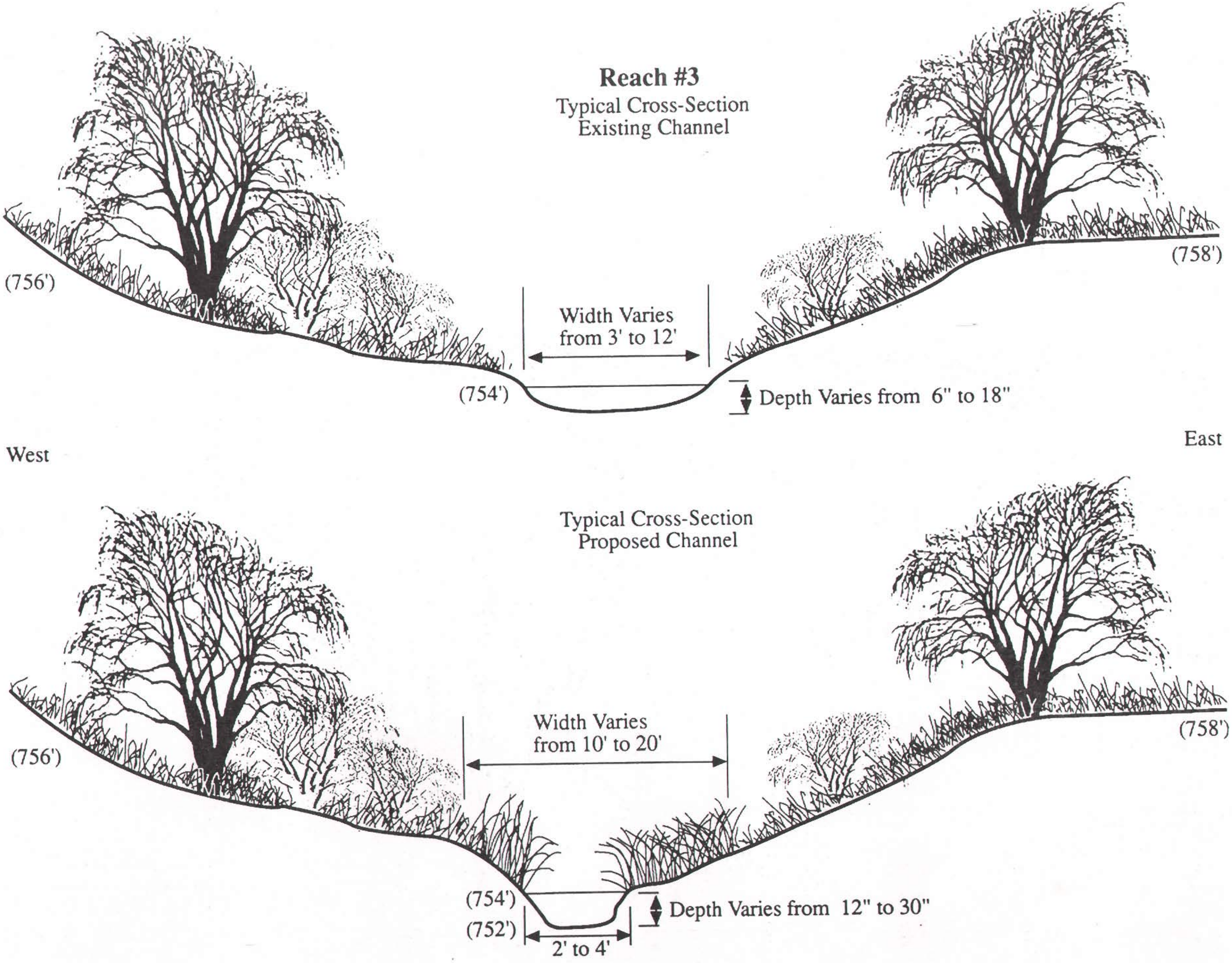
Lilly Creek
Restoration &
Enhancement
Measures

Figure 8
Reach 2
Cross Section



Lilly Creek
Restoration &
Enhancement
Measures

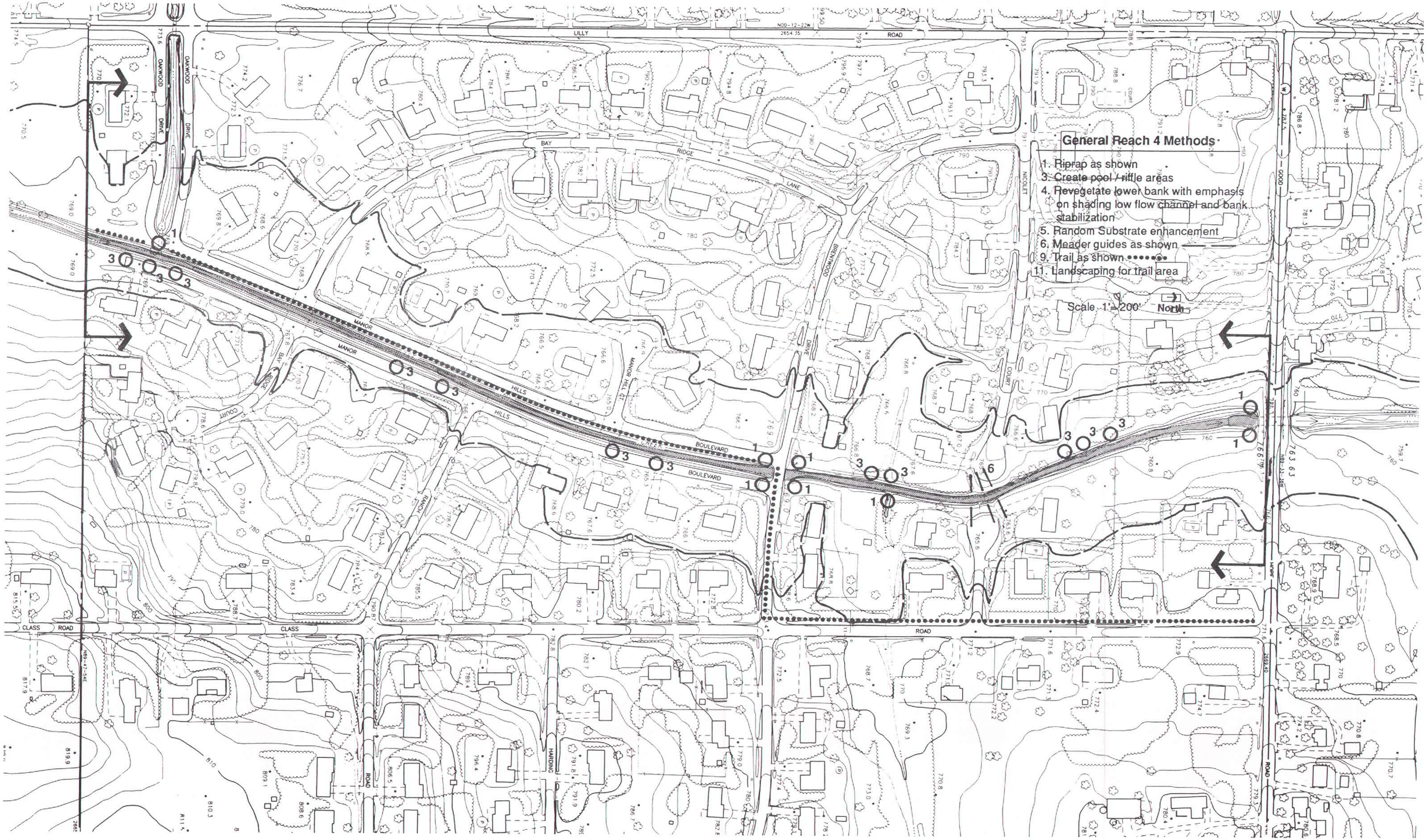
Figure 9
Reach 3
Plan View



Reach #3
Typical Cross-Section
Existing Channel

Typical Cross-Section
Proposed Channel

Figure 10
Reach 3
Cross Section



BRW

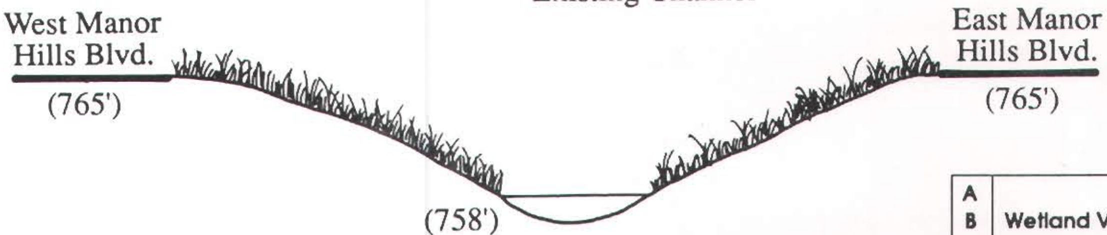
Lilly Creek Restoration & Enhancement Measures

Menomonee Falls

Figure 11
Reach 4
Plan View

Reach #4

Typical Cross-Section
Existing Channel



A B C	Wetland Vegetation	Bluestem / Switchgrass Wet Meadow Cattails / Rushes
D	Lower Bank	Tall Wet Grasses
E	Upper Bank	Medium Grass / Forbs
F	Landscaped Area	Bluegrass

Typical Cross-Section
Proposed Channel

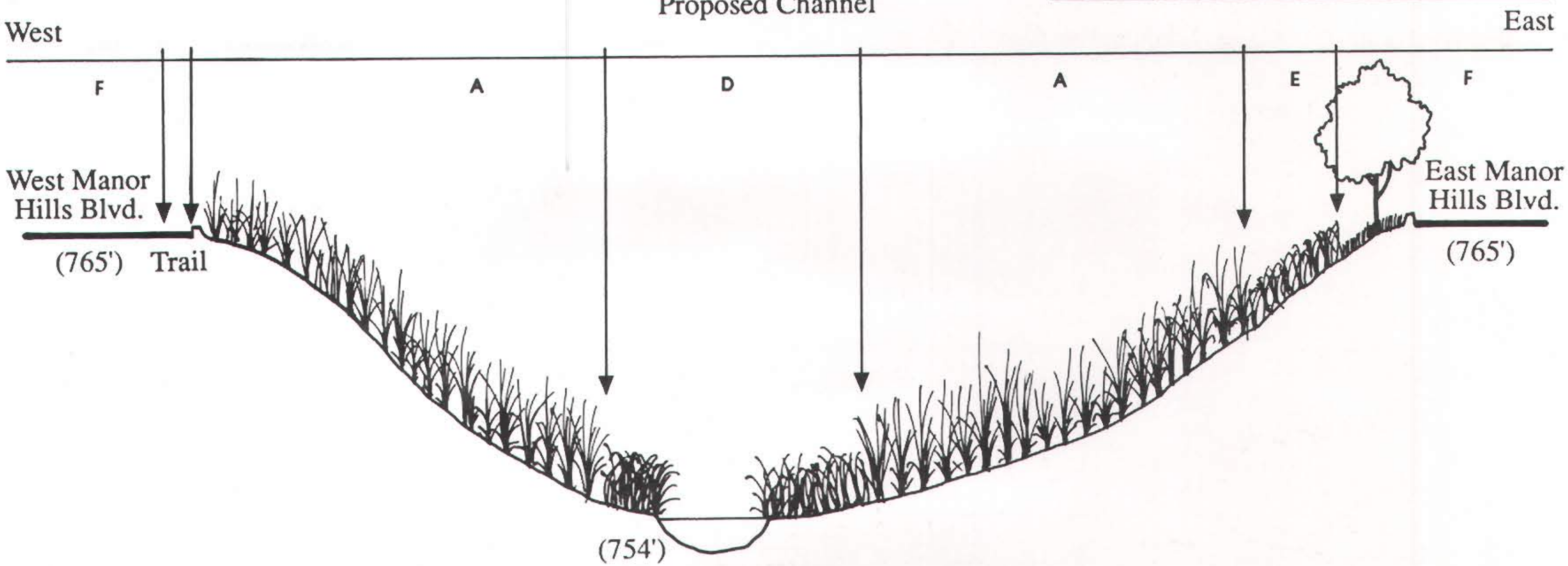
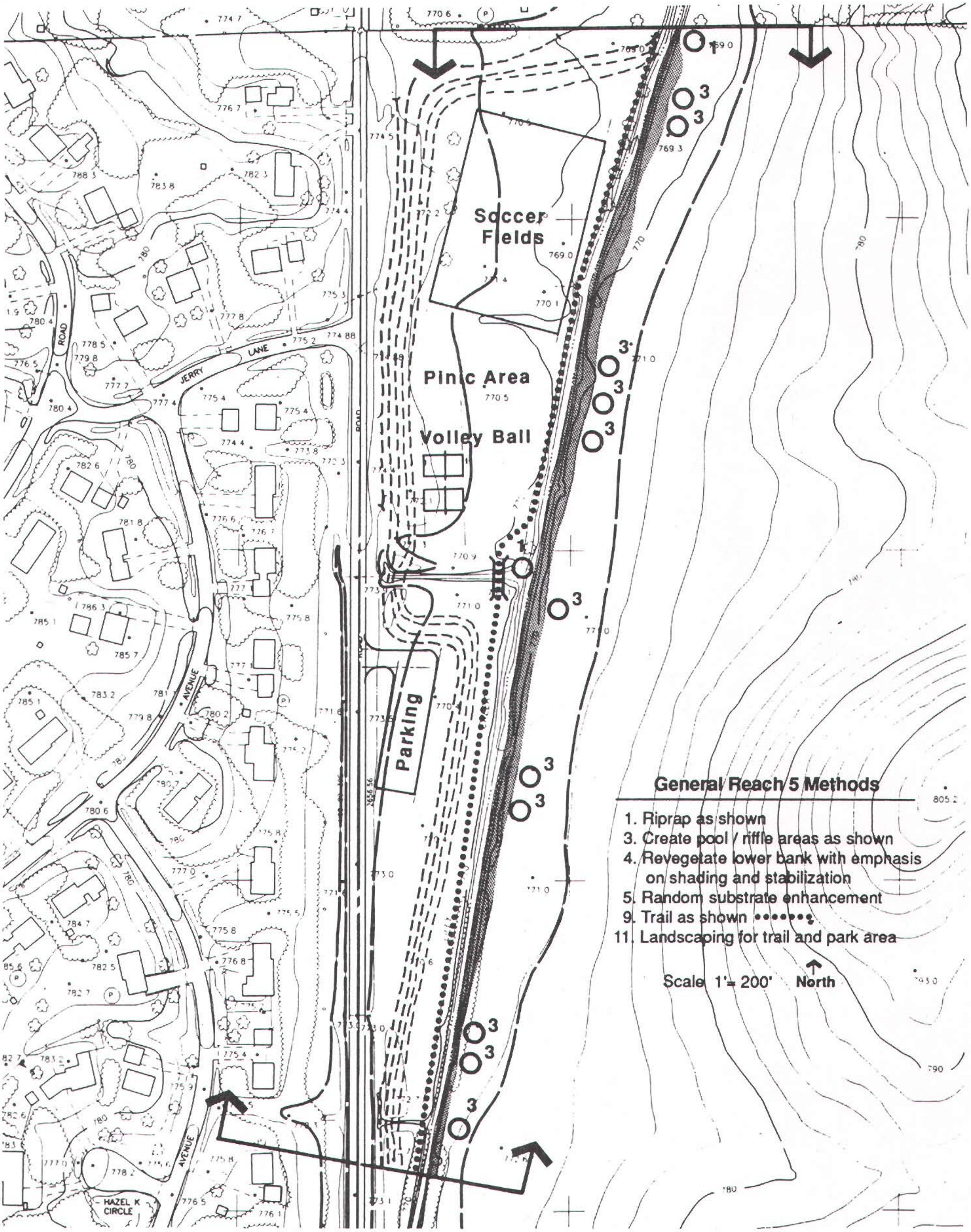


Figure 12
Reach 4
Cross Section



B R W

Lilly Creek
Restoration &
Enhancement
Measures

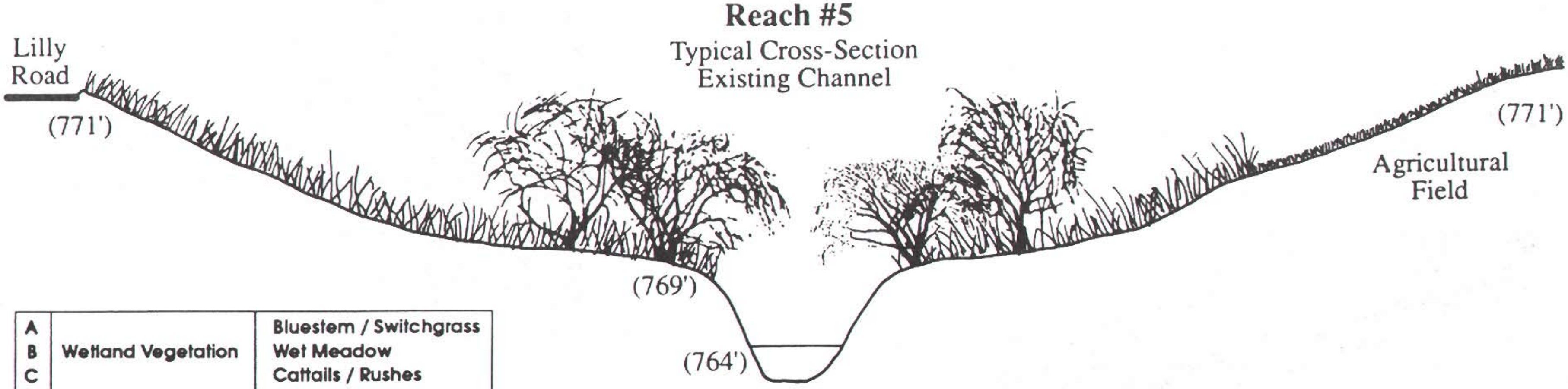
Menomonee Falls

Figure 13
Reach 5
Plan View

Lilly Creek Restoration & Enhancement Measures

Reach #5

Typical Cross-Section
Existing Channel



A B C	Wetland Vegetation	Bluestem / Switchgrass Wet Meadow Cattails / Rushes
D	Lower Bank	Tall Wet Grasses
E	Upper Bank	Medium Grass / Forbs
F	Landscaped Area	Bluegrass

Typical Cross-Section
Proposed Channel

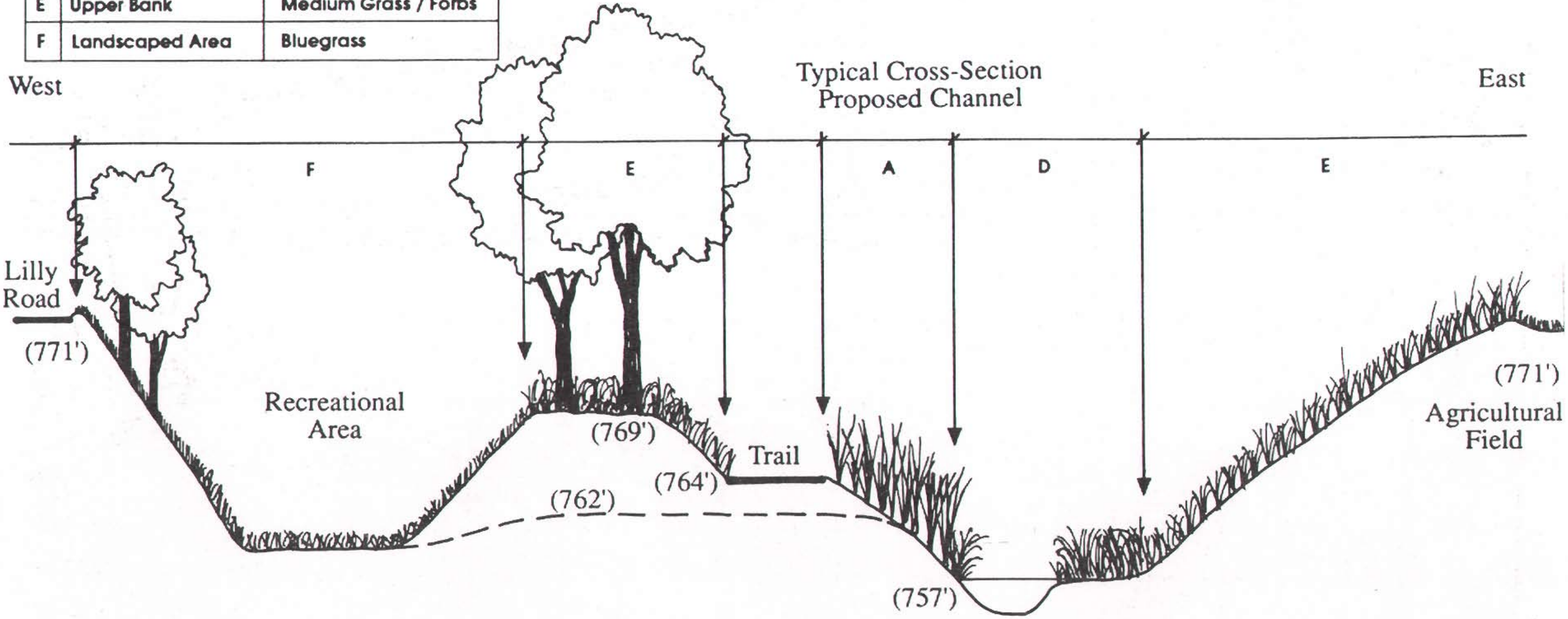
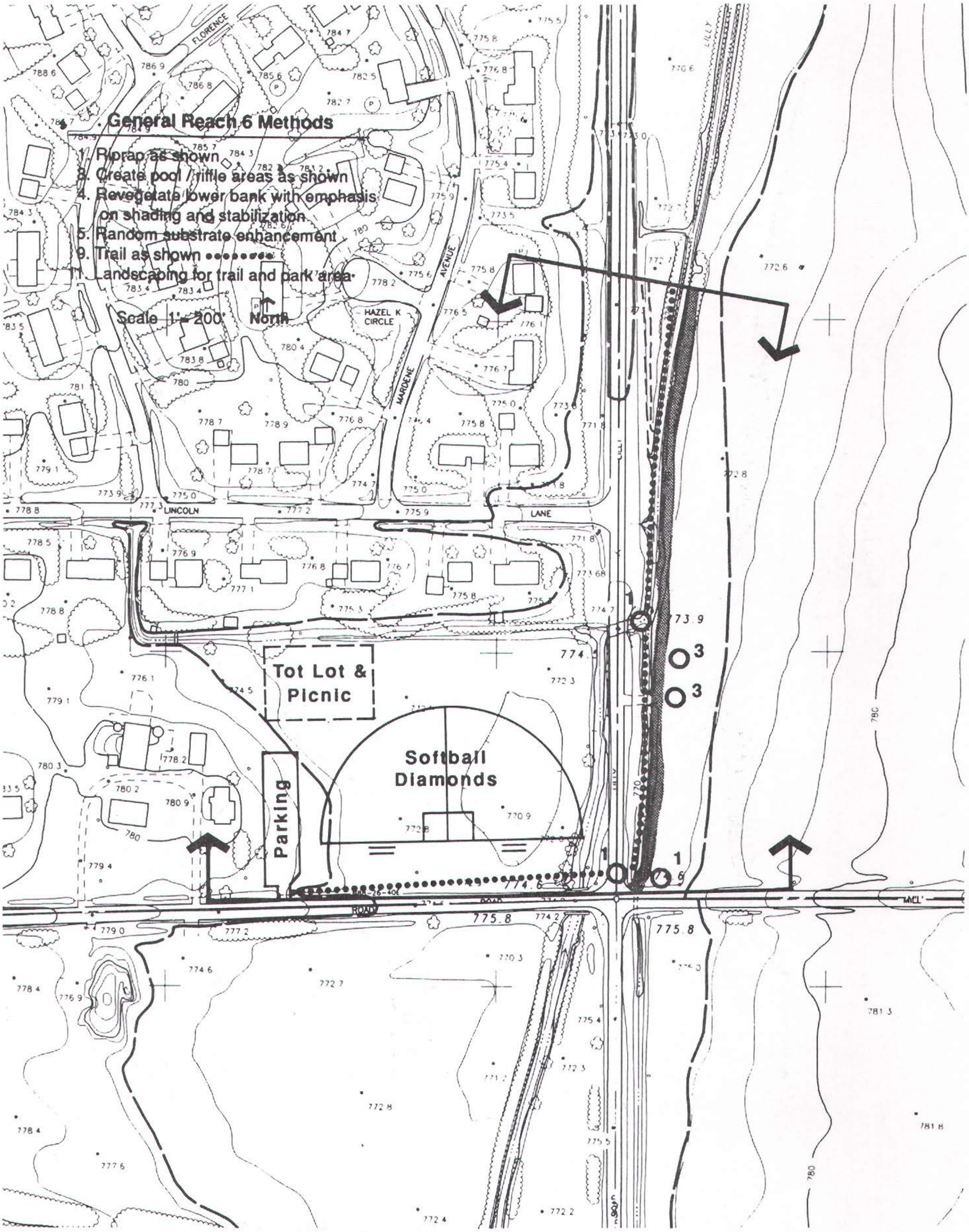


Figure 14
Reach 5
Cross Section



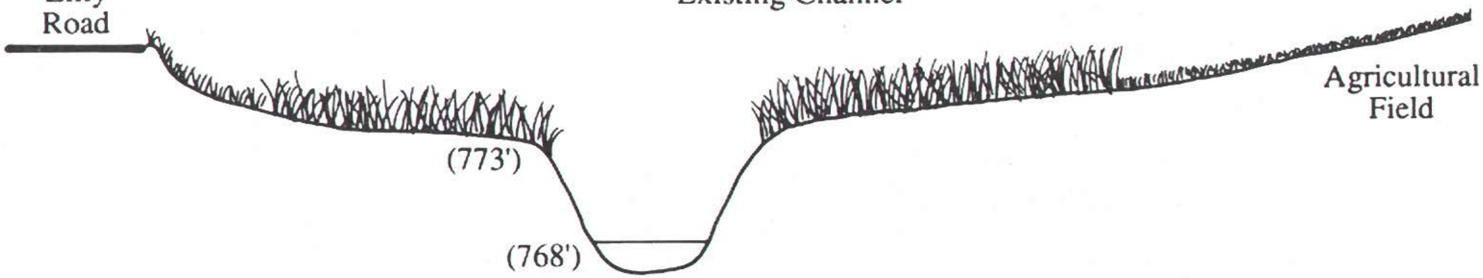
Lilly Creek Restoration & Enhancement Measures

Menomonee Falls

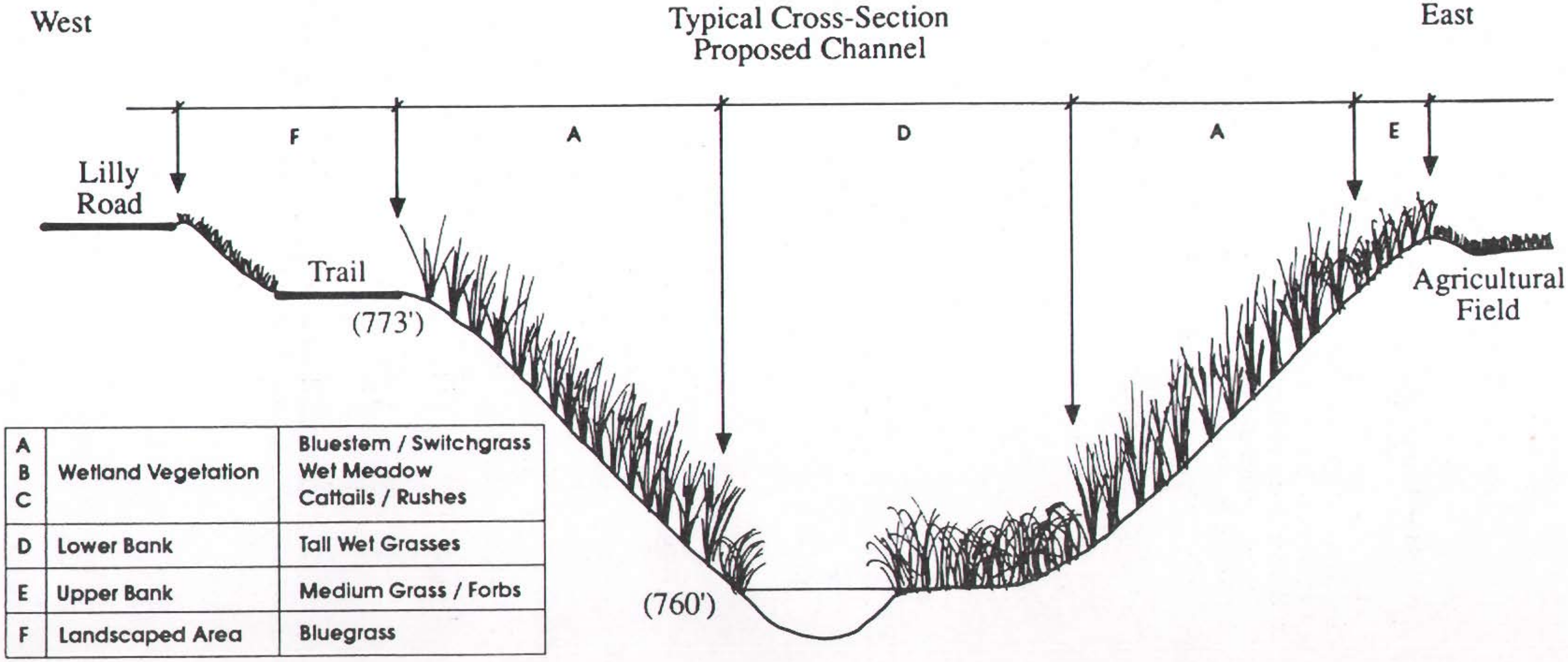
Figure 15
Reach 6
Plan View

Reach #6

Typical Cross-Section
Existing Channel

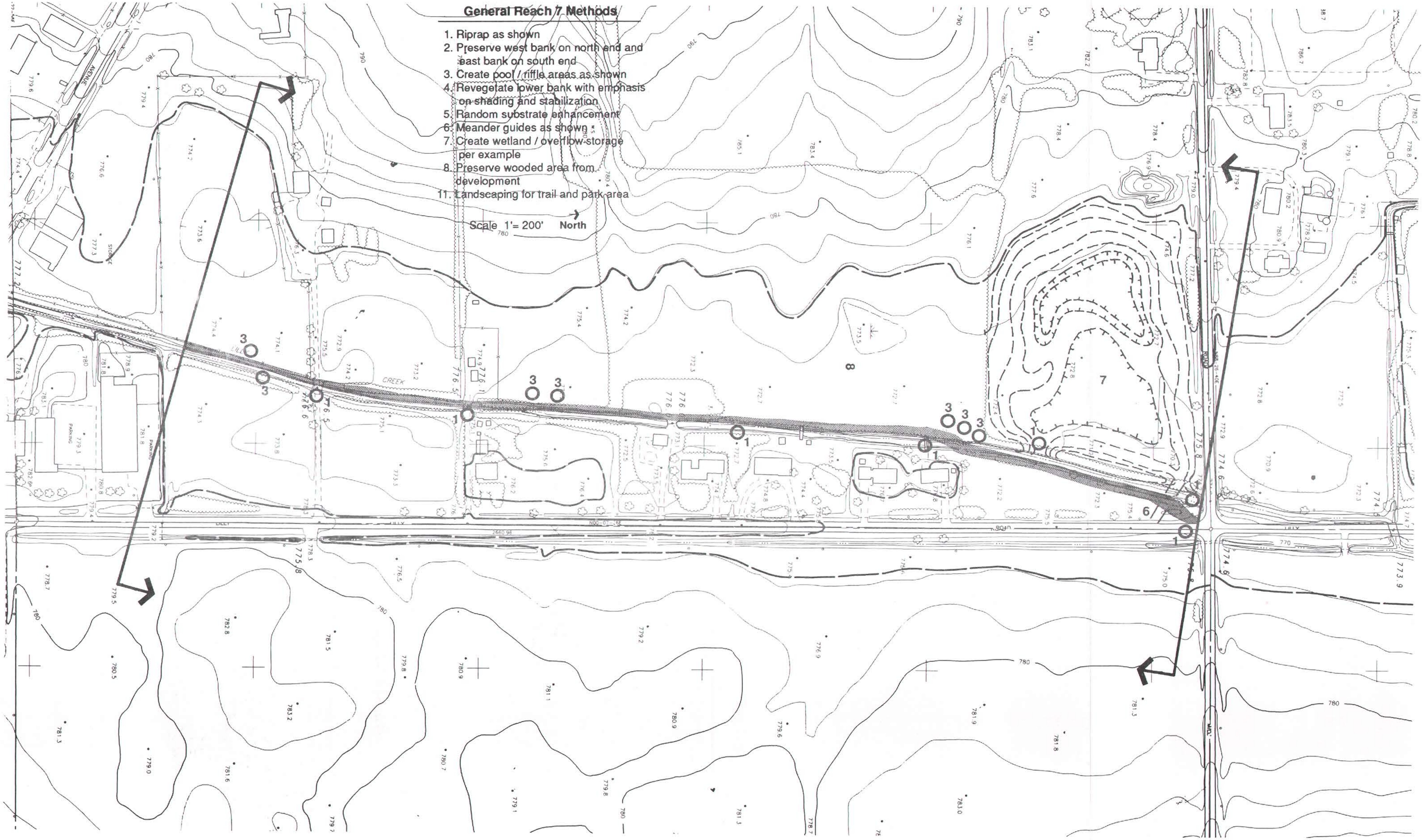


Typical Cross-Section
Proposed Channel



A B C	Wetland Vegetation	Bluestem / Switchgrass Wet Meadow Cattails / Rushes
D	Lower Bank	Tall Wet Grasses
E	Upper Bank	Medium Grass / Forbs
F	Landscaped Area	Bluegrass

Figure 16
Reach 6
Cross Section



General Reach 7 Methods

1. Riprap as shown
2. Preserve west bank on north end and east bank on south end
3. Create pool / riffle areas as shown
4. Revegetate lower bank with emphasis on shading and stabilization
5. Random substrate enhancement
6. Meander guides as shown
7. Create wetland / overflow storage per example
8. Preserve wooded area from development
11. Landscaping for trail and park area

Scale 1'= 200' North

B R W

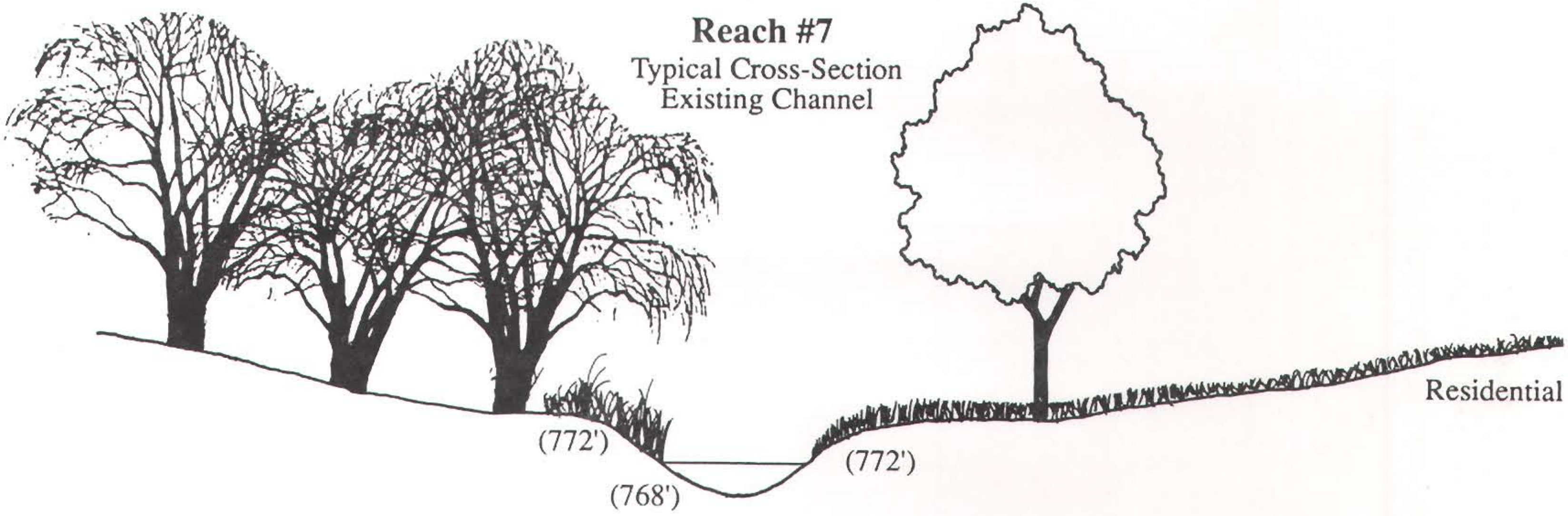
Lilly Creek Restoration & Enhancement Measures

Menomonee Falls

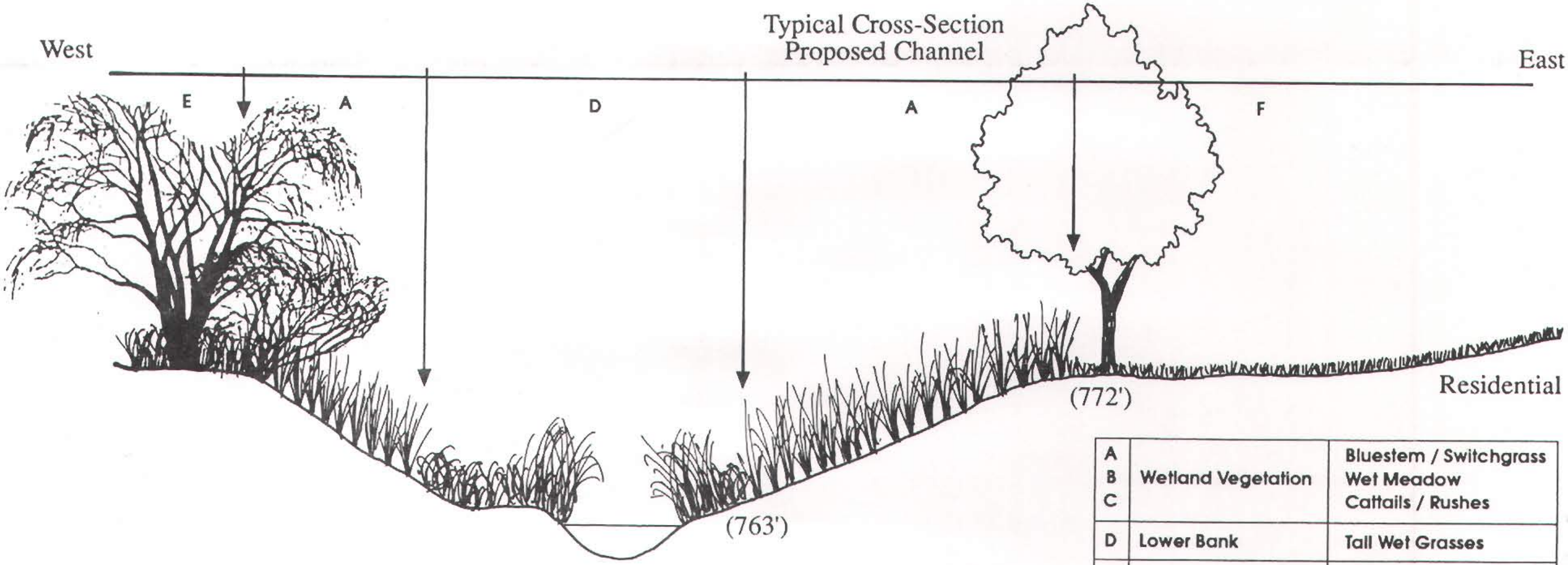
Figure 17
Reach 7
Plan View

Reach #7

Typical Cross-Section
Existing Channel

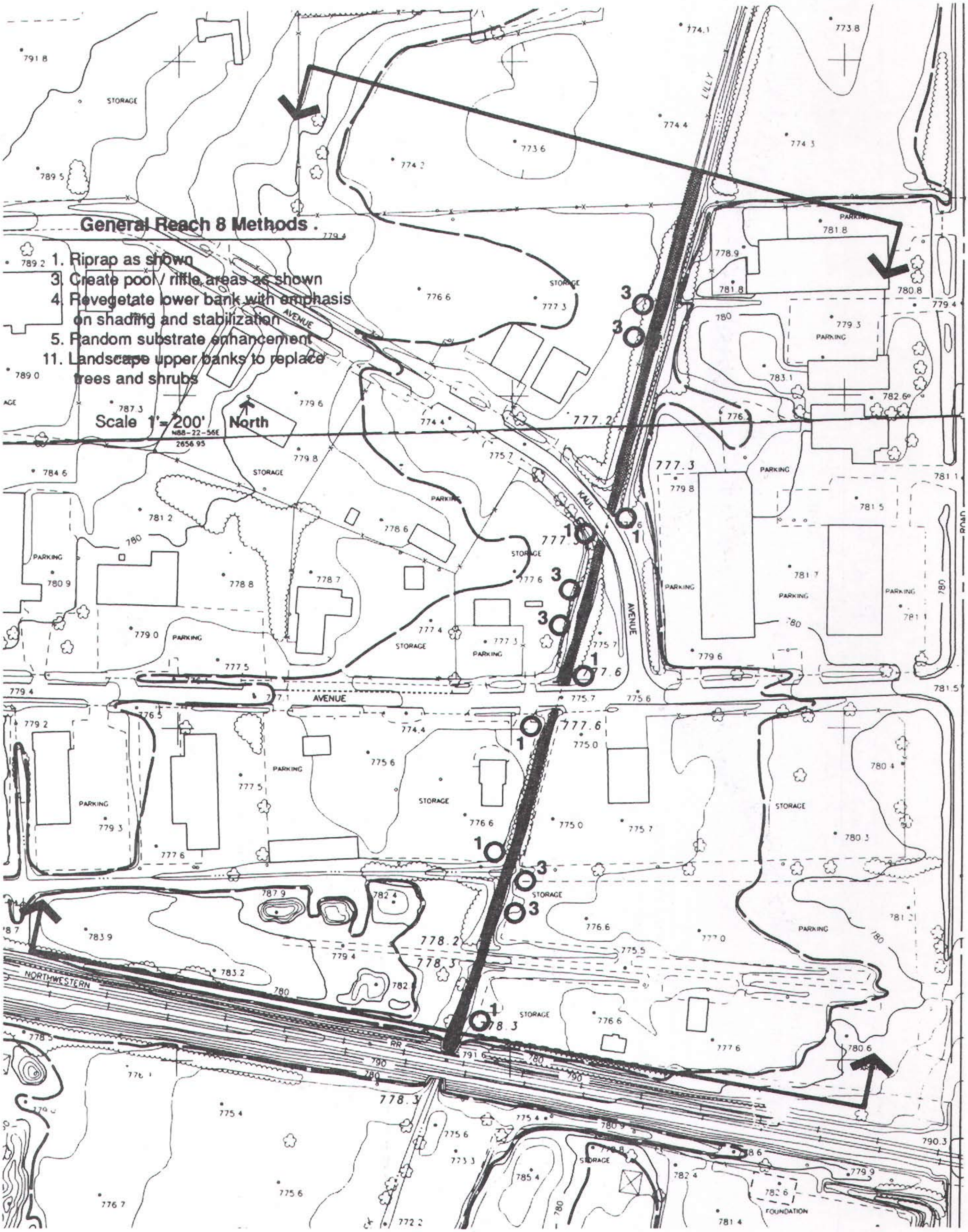


Typical Cross-Section
Proposed Channel



A B C	Wetland Vegetation	Bluestem / Switchgrass Wet Meadow Cattails / Rushes
D	Lower Bank	Tall Wet Grasses
E	Upper Bank	Medium Grass / Forbs
F	Landscaped Area	Bluegrass

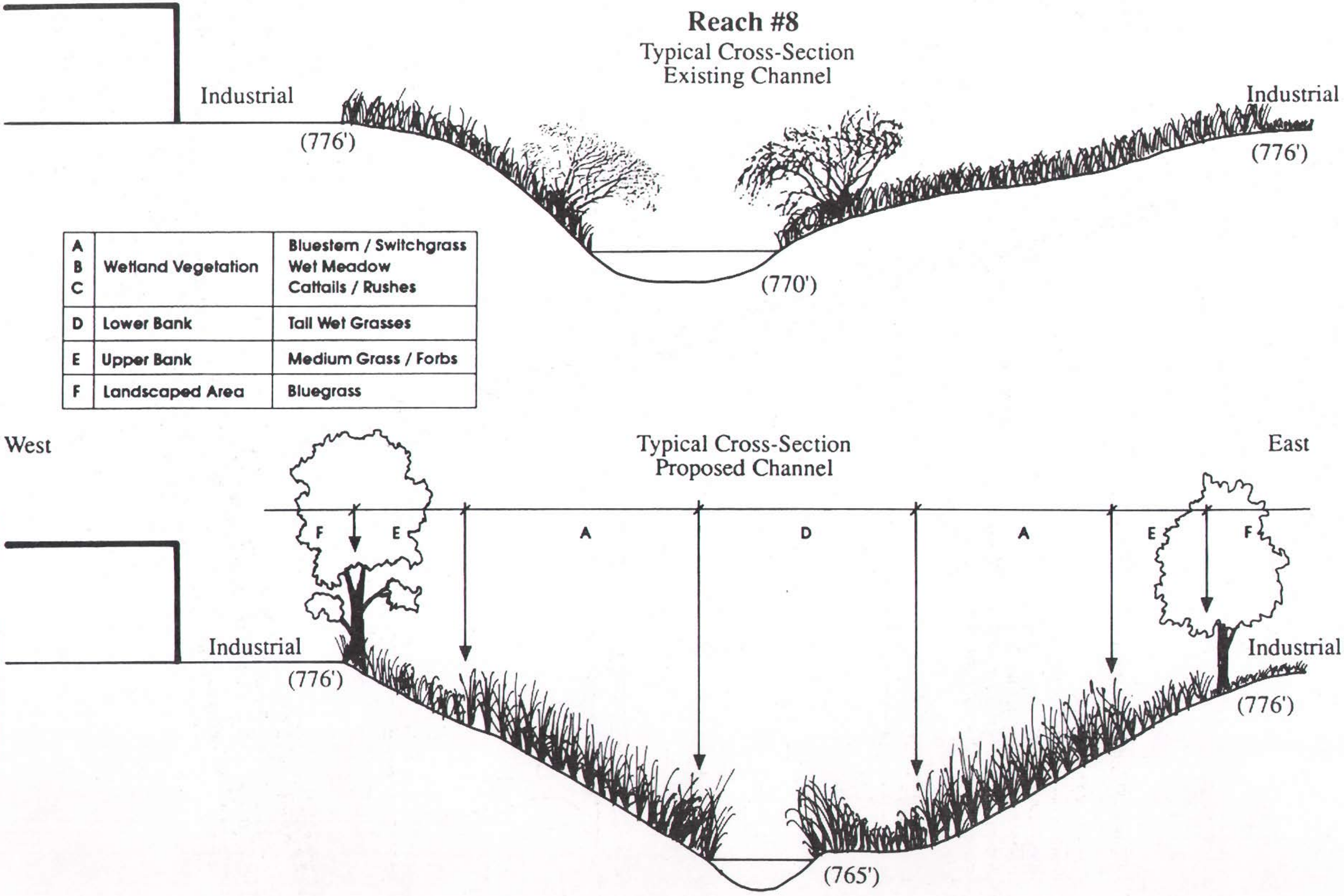
Figure 18
Reach 7
Cross Section



BRW

Lilly Creek
Restoration &
Enhancement
Measures

Figure 19
Reach 8
Plan View



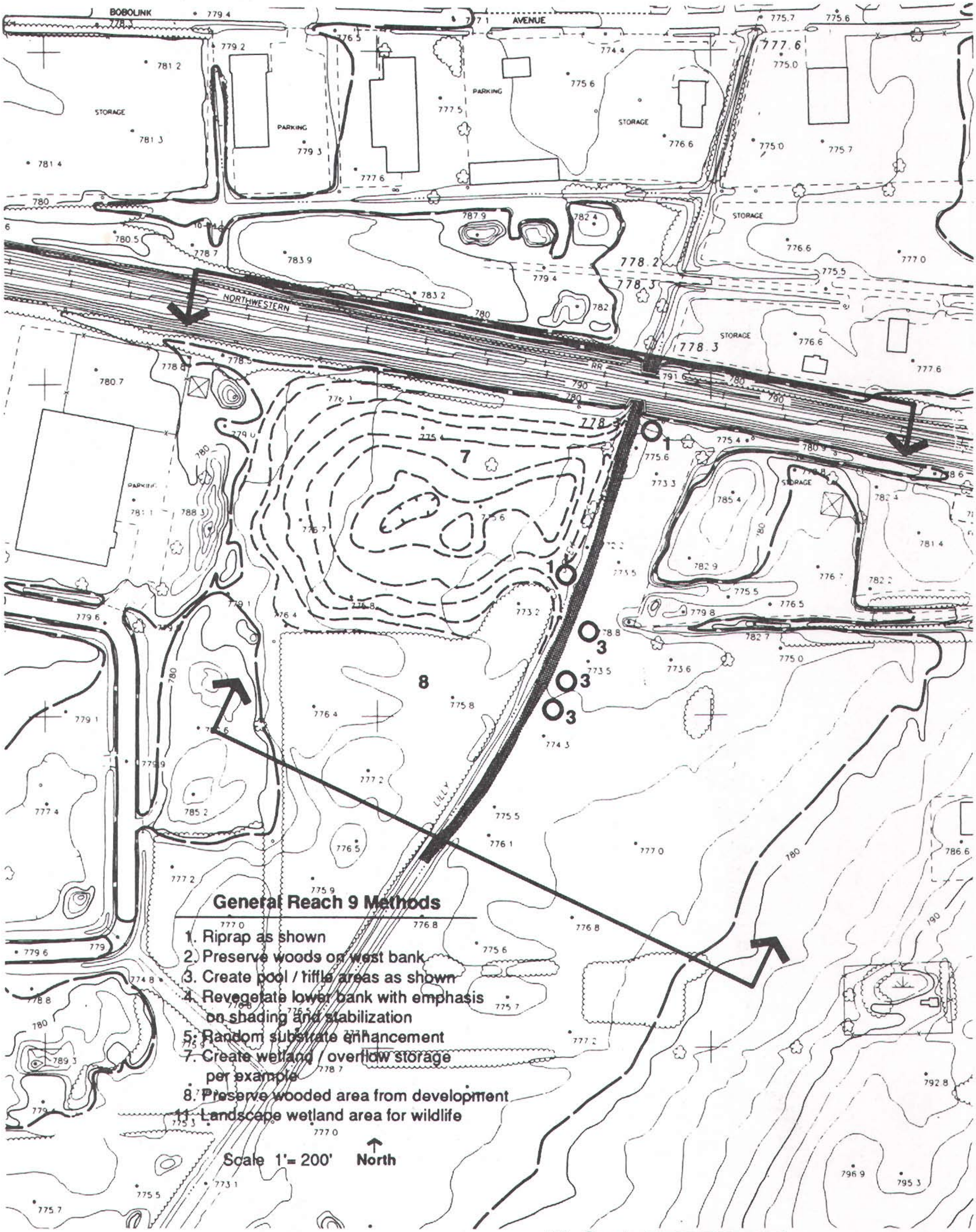
Reach #8

Typical Cross-Section Existing Channel

Typical Cross-Section Proposed Channel

A	Wetland Vegetation	Bluestem / Switchgrass
B		Wet Meadow
C		Cattails / Rushes
D	Lower Bank	Tall Wet Grasses
E	Upper Bank	Medium Grass / Forbs
F	Landscaped Area	Bluegrass

Figure 20
Reach 8
Cross Section



General Reach 9 Methods

1. Riprap as shown
2. Preserve woods on west bank
3. Create pool / riffle areas as shown
4. Revegetate lower bank with emphasis on shading and stabilization
5. Random substrate enhancement
7. Create wetland / overflow storage per example
8. Preserve wooded area from development
11. Landscape wetland area for wildlife

Scale 1'= 200' North

B R W

Lilly Creek
Restoration &
Enhancement
Measures

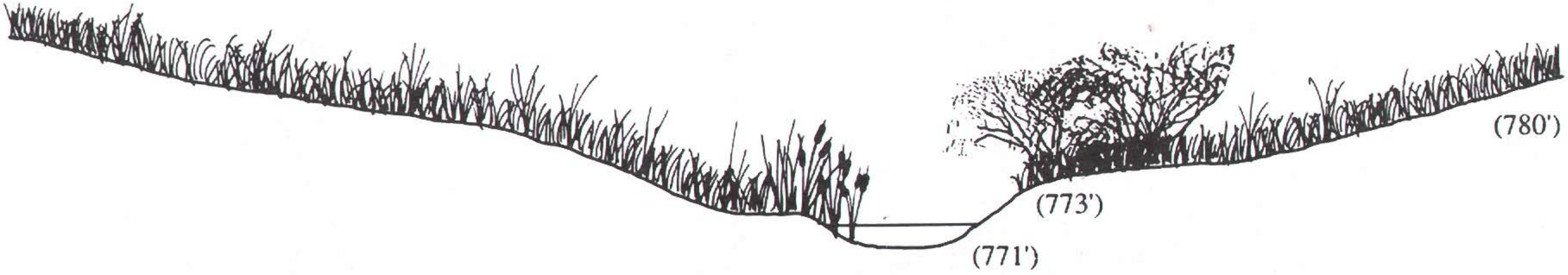
Menomonee Falls

Figure 21
Reach 9
Plan View

Source: BRW, Inc.

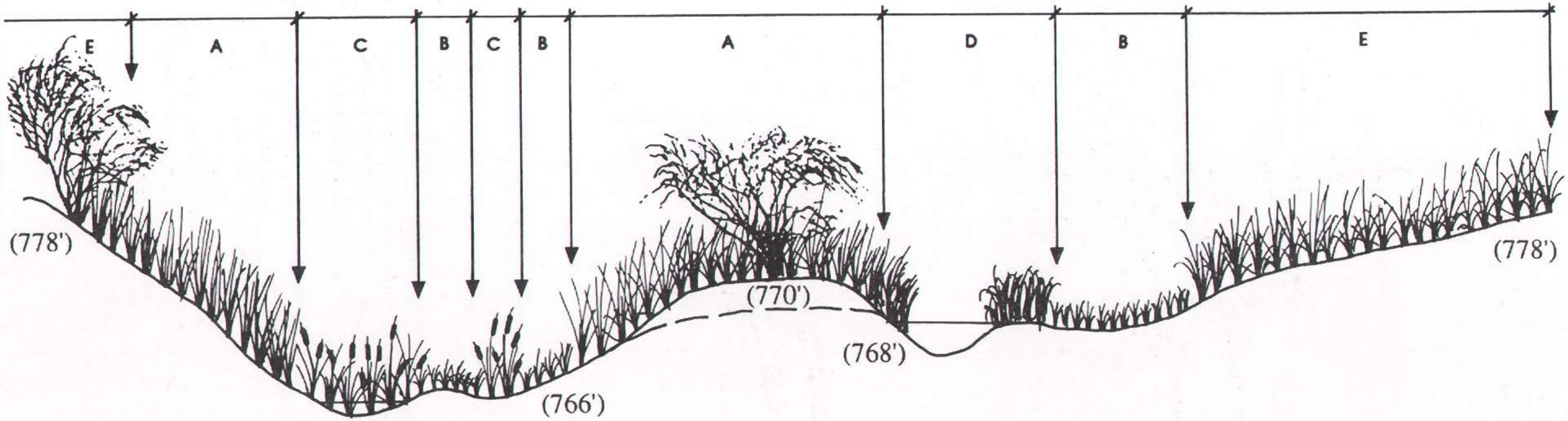
Reach #9

Typical Cross-Section
Existing Channel



A	Wetland Vegetation	Bluestem / Switchgrass
B		Wet Meadow
C		Cattails / Rushes
D	Lower Bank	Tall Wet Grasses
E	Upper Bank	Medium Grass / Forbs

Typical Cross-Section
Proposed Channel



Lilly Creek Restoration & Enhancement Measures

Figure 22
Reach 9
Cross Section

Appendix D

WETLAND ALTERNATIVES ANALYSIS FOR COMPLIANCE WITH CHAPTER NR 103 OF THE WISCONSIN ADMINISTRATIVE CODE

INTRODUCTION

This appendix documents the practicable alternatives analysis performed for those stormwater management and flood control features which were called for under the preliminary recommended plan and which involved disturbance of wetlands. Where a practicable alternative to location of a given facility in a wetland was identified, the final recommended plan presented in Chapter VI of this report was revised to include that alternative. The final recommended stormwater management and flood control plan is shown on Map 20 and wetlands in the study area are shown on Map 22 in Chapter VI. The alternatives analysis was prepared in the context of the system plan presented in this report and is intended to be adequate to obtain conceptual approval of the stormwater management and flood control plan from the Department of Natural Resources and to expedite the permitting process at such time as specific features of the recommended plan are implemented. It may be necessary for the applicant for a state permit required to implement a facility recommended under this plan to provide additional data in support of the proposed project.

WETLAND CONSIDERATIONS IN HYDROLOGIC UNIT B, PHILLIPS TRIBUTARY

Detention Basin WD2

Under the preliminary recommended stormwater drainage plan presented in Chapter V of this report, dual-purpose detention basin WD2 was proposed to be located in a portion of a 7.5-acre wetland at the headwaters of the Phillips Tributary in the southeast one-quarter of U. S. Public Land Survey Section 27, Township 8 North, Range 20 East. Basin WD2 is intended to control a range of floods, including the more frequent floods which contribute to streambank erosion and streambed scour, and also to provide significant reductions in loadings of nonpoint source pollutants delivered to the Phillips Tributary and to Lilly Creek under planned land use conditions.

The wetland is classified as a nonshoreland wetland on the Village's wetlands inventory map. The total area draining to the portion of the wetland in which basin WD2 was to be located is about 84 acres. As set forth in Table 51 in Chapter VI of this report, the existing land uses tributary to that portion of the wetland are almost entirely rural, while under planned conditions the land uses would be almost completely urban, with about 76 percent of the area developed in industrial uses. The wetland type is classified as an emergent marsh wetland which has wet soils and has been abandoned from agriculture (E1Ka). The wetland was field inspected by Commission staff on October 11, 1990, and the plant community was described as consisting of shallow marsh, fresh (wet) meadow, shrub-carr, and southern wet to wet-mesic lowland hardwoods. Disturbances to the plant community noted at that time included filling and past ditching with side casting of dredge spoil materials. No federal- or state-designated rare, threatened, or endangered species were observed during the field inspection.

The soils in the portion of the wetland which was to include WD2 are classified as Sebewa silt loam. Sebewa silt loam is poorly drained and may occur in areas with a high water table.

The wetland is not in, or adjacent to, an area of special natural resource interest. Wildlife habitat at the site is classified as Class II, or of medium quality.

The portion of the wetland in which basin WD2 was proposed to be located does not have significant flood or sediment storage capacity because of relatively steep land slopes and the absence of a downstream hydraulic control. That portion of the wetland also would not be a significant spawning area in comparison to the larger northern portion of the wetland, where no stormwater management measures are to be located. The wetland does not front on open water and, therefore, does not function to provide shoreline erosion protection.

Because detention basin WD2 could be constructed on open land outside the wetland without significantly compromising the basin's intended water quantity and quality control functions, it was reconfigured to be located outside the wetland under the final recommended stormwater management and flood control plan set forth in Chapter VI of this report.

Storm Sewers to Drain El Camino Drive, Mesa Drive and Vista Lane

The staff of the Village's Department of Public Works specifically requested that an adequate outlet be provided for surface water runoff from the existing subdivision located along El Camino Drive, Mesa Drive, and Vista Lane north of the Phillips Tributary in the southwest one-quarter of U. S. Public Land Survey Section 26, Township 8 North, Range 20 East. Thus, the recommended plan calls for the construction of 1,020 lineal feet of 12-inch-diameter storm sewer and 500 lineal feet of 15-inch-diameter storm sewer to convey runoff from the subdivision through a wetland and to Phillips Tributary.

The wetland is classified as a shoreland wetland on the Village's wetlands inventory map. The total area draining to the wetland is about 370 acres. Agricultural and open space uses and low-density residential uses are the predominant existing land uses in the area tributary to the wetland. Under planned conditions the tributary land uses would be almost completely urban, with industrial and low-density residential uses being predominant.

The wetlands affected by the proposed storm sewers are classified as an emergent marsh wetland with narrow-leaved vegetation on wet soils (E2K) and a broadleaf forested wetland with wet soils (T3K).

The soils in the portion of the wetland where the storm sewers would be constructed are classified as Pella silt loam. Pella silt loam is poorly drained and may occur in areas with a high water table. The soil survey indicates that this soil type generally consists, below the surface layers, of silty clay loams and silt loams.

The wetland is not in, or adjacent to, an area of special natural resource interest. Wildlife habitat at the site is classified as Type II, or of medium quality.

The wetland has significant flood and sediment storage capacity and could provide significant spawning habitat. Construction of the proposed storm sewers would not reduce the flood and sediment storage capacities or significantly degrade spawning habitat.

The recommended storm sewers are required to alleviate existing problems with standing water and saturated soils adjacent to the foundations of the residences in the subdivision. It is possible to construct open channels in place of the storm sewers, but such an installation is not a permitted use under Chapter NR 117 of the Wisconsin Administrative Code, dealing with shoreland-wetland regulations. The only other alternative would be to do nothing and allow the existing drainage problems to continue. That alternative is unacceptable to the Village. It was, therefore, concluded that there is no practicable alternative to the recommended storm sewers channel which would provide the required drainage benefits. Thus, it is recommended that the storm sewers be constructed, with special provisions made to avoid adverse impacts on the functional values of the wetland. Such provisions could include providing gasket joints on the sewer pipe and constructing regularly spaced impervious cutoffs in the sewer trench.

WETLAND CONSIDERATIONS IN HYDROLOGIC UNIT C, BOWLING GREEN TRIBUTARY

Under the preliminary recommended stormwater drainage plan, an open channel to divert flow in the Bowling Green Tributary around the Bowling Green Industrial Park to Lilly Creek was proposed to be located in a portion of a four-acre wetland adjacent to Lilly Creek in the northeast one-quarter of Section 26. The diversion channel is intended to reduce the size of the drainage facilities recommended for the Industrial Park and, through such size reduction, to limit the deepening of the Lilly Creek channel to what is necessary to achieve the flood control objectives of this plan.

Almost all of the wetland is classified as a shoreland wetland on the Village's wetlands inventory map. The total area draining to the wetland under existing drainage conditions is about five acres, including the four-acre wetland itself and about one acre of industrial storage yard. Under preliminary proposed conditions, a total of about 111 acres of predominantly low- and medium-density residential land would drain to the diversion channel which would pass through the wetland.

The wetland is classified as an emergent marsh wetland with narrow-leaved vegetation on wet soils (E2K). The soils in the wetland are classified as Ashkum silty clay loam and Drummer silt loam. Both soil types are poorly drained and both may occur in areas with a high water table. The soil survey indicates that Ashkum soils generally consist of silty clays and silty clay loams to depths of at least five feet, but Drummer soils may have sand and gravel strata occurring at depths of about four feet or greater.

The wetland is not in, or adjacent to, an area of special natural resource interest. Wildlife habitat at the site is classified as Class II, or of medium quality. The wetland provides marginal spawning habitat.

The wetland is located primarily in the floodplain of Lilly Creek and has significant flood and sediment storage capacity and may provide some minor shoreline erosion protection.

In the upstream portion of the wetland, the diversion channel would convey all flows up to, and including, the 100-year recurrence interval flood flow within the excavated channel. The downstream portion of the diversion channel would be located in the 100-year floodplain of Lilly Creek. Water levels in that reach would be governed by Lilly Creek, rather than the diversion channel. In the western portion of the wetland, construction of the diversion channel would involve excavation to depths of from three to six feet below the existing grade. In the eastern portion of the wetland the depth of excavation would increase from about three feet to 10 feet in order to match the lowered streambed in the restored Lilly Creek channel. Such a channel could drain the wetland because of increased soil permeability in the portions where the excavated channel intercepts possible sand and gravel layers characteristic of the Drummer soil series.

Direct disturbance of the wetland could be avoided either by realigning the diversion channel so that it would pass either north or south of the preliminarily recommended alignment or by constructing a diversion pipe with the hydraulic capacity to convey the runoff from a 100-year recurrence interval storm along an alignment to the north of the wetland. Rerouting the proposed diversion channel along an alignment to the south of the wetland would involve the acquisition of five properties in the Bowling Green Industrial Park at an estimated cost of $470,000. Realigning the diversion channel along a route north of the wetland would involve acquisition of a portion of one property and of buildings on that property at an estimated cost of $70,000. That property includes the subject wetland and the estimated cost of land acquisition includes the cost of purchasing the wetland. The southerly alignment, in addition to having a significantly higher land acquisition cost than the northerly alignment, would also have a higher construction cost because of the need for more excavation. It was found that a piped diversion would have lower total capital and annual operation and maintenance costs than would an open channel diversion along a northern alignment and also that a piped diversion would place fewer restrictions on future development along the route of the diversion. Thus, the realignment of the proposed Bowling Green Tributary diversion to the north of the wetland was selected as the most practicable alternative and the recommended plan reflects a diversion pipe along that alignment.

WETLAND CONSIDERATIONS IN HYDROLOGIC UNIT E, AREA EAST OF LILLY CREEK AND NORTH AND SOUTH OF W. MILL ROAD

Under the preliminary recommended stormwater drainage plan, a storm sewer was recommended to be constructed to convey the runoff from planned medium-density residential development to recommended dry detention basin DD5. The preliminarily recommended storm sewer alignment essentially followed the existing drainage patterns which run through a 5.5-acre wetland in the northwest one-quarter of Section 25. The recommended detention basin is located outside the wetland.

The wetland is classified as a nonshoreland wetland on the Village's wetlands inventory map. The total area draining to the wetland under existing drainage conditions is about 33 acres, consisting of wetlands, woodlands, other open lands, and low-density residential development. Under planned land use conditions, a total of about 44 acres of predominantly medium-density residential land would drain to the storm sewer which would pass through the wetland.

The wetland is classified as a broadleaf shrub-dominated wetland on wet soils (S3K). The soils in the wetland are classified as Ashkum silty clay loam, Aztalan loam, Martinton silt loam, and Mequon silt loam. Ashkum soils are poorly drained, the other soil types are somewhat poorly drained. The soils generally occur in areas with a seasonally high water table. The soil survey indicates that the soils present in the wetland generally consist of silty clays and silty clay loams below the surface layers.

The wetland is not in, or adjacent to, an area of special natural resource interest. Wildlife habitat at the site is classified as Class II, or of medium quality. The wetland is located about 1,600 feet east of Lilly Creek and since it is not on a stream, it provides no spawning habitat.

The wetland has no hydraulic control at its outlet which would impound runoff; thus, the flood and sediment storage capacity of the wetland is considered insignificant. The wetland is not located adjacent to an open water body; therefore, it does not provide shoreline erosion protection.

Direct disturbance of the wetland could be avoided by realigning the preliminarily recommended storm sewers so that they would pass to the north and south of the wetland prior to discharging to recommended detention basin DD5. The realigned sewers could be constructed for about the same cost as those originally recommended. Therefore, in order to be in compliance with Chapter NR 103, the realignment of the proposed storm sewers to avoid the wetland was considered to be a practicable alternative and the recommended plan presented in Chapter VI of this report includes the realigned storm sewers.

WETLAND CONSIDERATIONS IN HYDROLOGIC UNIT F, LINCOLN LANE TRIBUTARY

Under the preliminary recommended stormwater drainage plan, a storm sewer was recommended to be constructed to convey the runoff from planned medium-density residential development in Subbasin LCH01. The preliminarily recommended storm sewer alignment follows the existing drainage patterns which run through a 4.5-acre wetland in the southwest one-quarter of Section 23.

The wetland has been classified as a nonshoreland wetland by Village's and the Commission's staff on the basis that the portion of the Lincoln Lane Tributary on which the wetland is located is a nonnavigable stream. The total area draining to the wetland under existing drainage conditions is about 54 acres, consisting of wetlands, other open lands, and suburban- and low-density residential development. Under planned land use conditions, approximately 90 percent of the area draining to the wetland would be in medium-density residential uses.

The wetland is classified as a broadleaf forested wetland with wet soils (T3K). The soils in the wetland are classified as Ashkum silty clay loam, Mequon silt loam, and Ozaukee silt loam. Ashkum soils are poorly drained, Mequon soils are somewhat poorly drained, and Ozaukee soils are well-drained and moderately well-drained. Ashkum and Mequon soils generally occur in areas with a seasonally high water table. The soil survey indicates that Ashkum and Mequon soils, which are those occurring along the proposed storm sewer alignment, generally consist of silty clays and silty clay loams below the surface layers.

The wetland is not in, or adjacent to, an area of special natural resource interest. Wildlife habitat at the site is classified as Class III, or of good quality. The wetland is located about 0.75 mile west of Lilly Creek and the downstream 0.2 mile of the Lincoln Lane Tributary have been enclosed. Therefore, it is unlikely that the wetland provides significant spawning habitat.

The wetland has no hydraulic control at its outlet which would impound runoff and the land slopes are relatively steep; thus, the flood and sediment storage capacity of the wetland is considered insignificant. The wetland is not located adjacent to an open water body; therefore, it does not provide shoreline erosion protection.

There are two feasible options for providing adequate stormwater drainage for new development in the areas upstream of the wetland. The first would be to construct the preliminarily recommended storm sewer through the wetland, using gasket joints and periodically spaced impervious cutoff walls in the trench to avoid the possibility of the wetland being drained by inflow to the sewer or along the sewer trench. Disturbance of the wetland would occur only during the construction period; such disturbance could be minimized through the provision of stringent construction erosion controls and site restoration procedures. The second option would be to avoid disturbance of the wetland by realigning the storm sewer so that it would pass to the south of the wetland, since a southern alignment is the only one which would avoid the wetland. That option would require installing 280 lineal feet more storm sewer than under the preliminary recommendation, but, because steeper slopes would permit the use of smaller diameter sewer for part of the total length, the additional cost of rerouting the storm sewer would only be $20,000. If the additional cost to add gasket joints and impervious cutoffs to the preliminarily recommended storm sewer were considered, the cost differential between the two options would be even less. Therefore, in order to be in compliance with Chapter NR 103, the realignment of the proposed storm sewers to avoid the wetland was considered to be a practicable alternative and the recommended plan presented in Chapter VI of this report includes the realigned storm sewers.

WETLAND CONSIDERATIONS IN HYDROLOGIC UNIT G, JERRY LANE TRIBUTARY

Under the preliminary recommended stormwater drainage plan, a modified open channel was recommended to be constructed partially in a wetland to provide an outlet for the discharge pipe from recommended dual-purpose detention basin WD22. That detention basin, which is located outside the wetland, would control the quality and quantity of the stormwater runoff to the Jerry Lane Tributary and to the 13.2-acre wetland in the southwest and southeast one-quarters of Section 23 through which the Tributary flows. In addition, dual-purpose detention basin WD15 is recommended to be constructed in the easternmost portion of the wetland, just upstream of the existing enclosure of the Tributary in a 66- to 72-inch-diameter reinforced concrete pipes.

The wetland has been classified as a shoreland wetland on the Village's wetland inventory map. The total area draining to the wetland under existing drainage conditions is about 188 acres, consisting of wetlands, woodlands, other open lands, and suburban- and low-density residential development. Under planned land use conditions, approximately 60 percent of the area draining to the wetland would be in medium-density residential uses, resulting from the development of existing open land other than wetlands and woodlands.

The wetland is classified as a broadleaf forested wetland on wet soils (T3K). The wetland was field inspected by Commission staff on January 16, 1992, and the plant community was described as consisting of fresh (wet) meadow, shrub carr, and second-growth southern wet-mesic lowland hardwoods. Disturbances to the plant community which were noted at that time included grading and filling along the wetland edge and side casting of dredge spoils for pond construction. No federal- or state-designated rare, threatened, or endangered species were observed during the field inspection.

The soils in the wetland are classified as Navan silt loam. Navan soils are poorly drained and generally occur in areas with a high water table. The soil survey indicates that Navan soils generally consist of silty clays and sandy clay loams below the surface layers.

The wetland includes an isolated, disturbed southern mesic woodland which is not considered to be an area of special natural resource interest, but which was inventoried under the natural area management plan being prepared by the Regional Planning Commission.[1] The woodland consists of open-grown trees and does not appear to be a forest remnant.

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[1] *SEWRPC Planning Report No. 41, Natural Area and Critical Species Habitat Protection and Management Plan for Southeastern Wisconsin, in preparation.*

Wildlife habitat in the wetland and environs site is classified as Class II, or of medium quality. The wetland is upstream of an approximately 1,400-foot-long enclosure of the Jerry Lane Tributary in 66- to 72-inch-diameter storm sewer. The wetland could provide spawning habitat, although the existing stream enclosure would be expected to significantly impair fish migration into the wetland, reducing the wetland's value as a spawning site.

Although the wetland may provide some minimal sediment storage capacity, its flood storage capacity is considered insignificant. The wetland may provide some shoreline erosion protection along the Jerry Lane Tributary because of the root systems of vegetation and the velocity reductions in the overbanks and main channel.

It was found that the modified channel called for under the preliminary recommended stormwater drainage plan could be reduced in length and slope to avoid disturbance of the channel within the wetland. Therefore, in order to be in compliance with Chapter NR 103, the length of the modified channel was reduced to avoid the wetland and the recommended plan presented in Chapter VI of this report includes that reduction in length.

The site for dual-purpose detention basin WD15 was reviewed by Commission staff biologists prior to its selection as a basin site. Because of the overall high functional values of most of the wetland as set forth above, the basin was located to take advantage of the lower quality area in the extreme eastern end of the wetland, reducing permanent inundation or excavation in those areas with the highest values. However, the basin still extends into a higher quality area of the wetland.

The basin is intended to control nonpoint source pollution, thereby improving the water quality of Lilly Creek, and to reduce peak flood flows, thereby avoiding the costly construction of additional stormwater conveyance facilities between the basin site and Lilly Creek. Those objectives might also be partially achieved through decentralized detention at scattered sites throughout the area tributary to the recommended basin. However, the provision of such decentralized detention would be likely to entail substantially higher capital and annual operation and maintenance costs than the recommended centralized detention basin. Decentralized detention would also present significant difficulties in designing basins to control runoff from the same amount of land as controlled under the centralized approach and to insure that those basins achieved the same degree of runoff quantity and quality control as afforded by the recommended centralized basin. It was, therefore, concluded that there is no practicable alternative to the recommended centralized detention basin which would be expected to meet the multiple objectives of water quantity and quality control in a cost-effective manner. Thus, it is recommended that centralized dual-purpose basin WD15 be constructed in the eastern portion of the wetland.

WETLAND CONSIDERATIONS IN HYDROLOGIC UNIT H, OAKWOOD TRIBUTARY

Under the preliminary recommended stormwater drainage plan, a 66-inch-diameter storm sewer was recommended to be constructed along the South Branch of the Oakwood Tributary through about 2.3 acres of a 14.4-acre wetland located in the northeast and northwest one-quarters of Section 23. In addition to the storm sewer, dual-purpose detention basin WD16 was recommended to be constructed in a portion of the eastern part of the wetland along both the North and South Branches of the Oakwood Tributary.

The wetland has been classified as a shoreland wetland on the Village's wetland inventory map. The total area draining to the wetland under existing drainage conditions is about 446 acres, of which 37 percent is low-density residential development and 59 percent is open lands, with the remainder in government and institutional and wetland uses. Under planned land use conditions, the area currently in open lands, other than wetlands, would be converted to medium-density residential land uses.

The 12.1-acre eastern portion of the wetland, in which basin WD16 is recommended to be located, is classified as an emergent marsh wetland on wet soils and which is grazed (ElKg). The wetland was field inspected by Commission staff on November 19, 1991, and the plant community was described as consisting of disturbed fresh (wet) meadow with small areas of shallow marsh. Disturbances to the plant community which were noted at that time included past heavy grazing and water level changes due to ditching, draining, and channel realignment. It was noted that the wetland appeared to be a groundwater discharge area. No federal- or state-designated rare, threatened, or endangered species were observed during the field inspection.

The 2.3 acre portion at the extreme western end of the wetland, through which the storm sewer is proposed to be constructed, is classified as a broadleaf forested wetland on wet soils (T3K).

The soils in the wetland are classified as Ashkum silty clay loam, Matherton silt loam, and Mequon silt loam. Ashkum soils are poorly drained and generally occur in areas with a high water table. Matherton and Mequon soils are somewhat poorly drained and generally occur in areas with a seasonally high water table. The recommended storm sewer would be constructed in Ashkum soils. The soil survey indicates that Ashkum soils generally consist of silty clays and silty clay loams below the surface layers.

The wetland is not in, or adjacent to, an area of special natural resource interest. Wildlife habitat in the portion of the wetland along the recommended storm sewer alignment is classified as Class III, or of good quality. Wildlife habitat in the portion of the wetland in which the recommended detention basin would be located is classified as Class II, or of medium value, with Class II a higher habitat classification than Class III. The wetland could provide limited spawning habitat.

Because of its severely degraded condition, the wetland provides insignificant sediment and flood storage capacity and shoreline erosion protection along the North and South Branches of the Oakwood Tributary.

Alternative Evaluation of Recommended 66-Inch-Diameter Storm Sewer in the Western Portion of the Wetland

The recommended storm sewer would be part of an overall drainage system which is recommended to provide protection from 100-year recurrence interval flooding of three existing houses along Terrace Drive, under planned land use conditions and to accommodate runoff from planned medium-density residential development. In the area of existing development along Terrace Drive, it is necessary to install storm sewers which would function in conjunction with the existing roadside swales to convey the 100-year recurrence interval flood without flooding existing houses. The flow in those storm sewers, plus that in the existing swales along Terrace Drive, would be conveyed through downstream areas of new medium-density residential development in the recommended 66-inch storm sewer. If an open channel were substituted for the recommended storm sewer in the area of new development downstream of the existing east end of Terrace Drive, it would have to be about eight feet deep in order to accommodate the storm sewer installed to alleviate flooding of existing development along Terrace Drive. That channel depth would not be safe and could result in the wetland being drained. It was, therefore, concluded that there is no practicable alternative to the recommended storm sewer, which would be expected to provide flood control benefits and would not present a safety hazard. Thus, it is recommended that the 66-inch-diameter storm sewer be constructed in the western portion of the wetland, with special provisions made to avoid lowering groundwater levels in the wetland. Such provisions could include providing gasket joints on the sewer pipe and constructing regularly spaced impervious cutoffs in the sewer trench.

Alternative Evaluation of Recommended Dual-Purpose Detention Basin WD16 in the Eastern Portion of the Wetland

Basin WD16 would provide significant water quantity and quality benefits along the Oakwood Tributary as well as along Lilly Creek. The basin is an integral part of the overall plan to reduce nonpoint source pollutant loadings to Lilly Creek under existing and planned land use conditions and to reduce streambank erosion and streambed scour caused by frequently occurring floods. It also provides large reductions in peak flood flows and stages along the Oakwood Tributary and, in conjunction with the other detention basins recommended under this system plan, along one of the reaches of Lilly Creek with the highest flood damage potential. Construction of basin WD16 provides protection for four houses along the Oakwood Tributary which could be flooded during a 100-year recurrence interval flood occurring under planned land use and existing drainage and channel conditions. As shown on Map 20 in Chapter VI of this report, a large portion of the recommended detention basin would be inundated only during floods. The Village plans to utilize those lands outside the permanent pond for park and recreation purposes.

The site for detention basin WD16 was reviewed by Commission staff biologists prior to its selection as a basin site and was identified as a highly disturbed wetland. Because of the degraded nature of the existing wetland at the detention basin site, it is proposed that the water quality control portion of the basin be designed as a wetland enhancement which would improve the wildlife habitat characteristics and functional values of the existing wetland. Thus, the basin would be the main component of a multi-objective project intended to control nonpoint source pollution, provide wetland enhancement, control floods, and provide recreational and park area.

Those objectives could not be achieved through decentralized detention at scattered sites throughout the area tributary to the recommended basin. Even if the wetland enhancement and park and recreation objectives were eliminated, decentralized detention would present significant difficulties in designing basins to control runoff from the same amount of land as controlled under the centralized approach and to insure that those basins achieved the same degree of runoff quantity and quality control as afforded by the recommended centralized basin. In addition, because of the relatively large detention basin required to meet the multiple objectives, there is no single suitable upland site for the basin. It was, therefore, concluded that there is no practicable alternative to the recommended centralized detention basin which would be expected to meet the multiple objectives set forth above. Thus, it is recommended that centralized dual-purpose basin WD16 be constructed in the eastern portion of the wetland.

WETLAND CONSIDERATIONS IN HYDROLOGIC UNIT L, MENOMONEE MANOR TRIBUTARY

Under the preliminary recommended stormwater drainage plan, dual-purpose detention basin WD24 was recommended to be constructed on the Menomonee Manor Tributary. The basin would be located in about 2.2 acres of wetland at a storm sewer outfall. The wetland area affected by the basin is part of a large, linear wetland which extends downstream of the site along the Menomonee Manor Tributary and includes areas along Lilly Creek and the Menomonee River.

The wetland has been classified as a shoreland wetland on the Village's wetland inventory map. The total area draining to the wetland under 1985 drainage conditions is about 215 acres, of which 36 percent is low-density residential development, 12 percent is commercial development, 2 percent is in government and institutional uses, 1 percent is high-density residential development, and the remaining 49 percent is in open lands, including wetlands, woodlands, and other open space. Planned land use conditions call for 32 percent in low-density residential development, 39 percent in medium-density residential uses, 16 percent in commercial uses, 2 percent in government and institutional uses, and 11 percent as wetlands.

The wetland is classified as a broadleaf forested wetland on wet soils (T3K). The wetland was field inspected by Commission staff on January 23, 1992, and the plant community was described as consisting of southern wet to wet-mesic lowland hardwoods. Disturbances to the plant community that were noted at that time included past selective tree cutting and past wetland fill along the edge. No federal- or state-designated rare, threatened, or endangered species were observed during the field inspection.

The soils in the wetland are classified as Ashkum silty clay loam and Mequon silt loam. Ashkum soils are poorly drained and generally occur in areas with a high water table. Mequon soils are somewhat poorly drained and generally occur in areas with a seasonally high water table. The soil survey indicates that Ashkum and Mequon soils generally consist of silty clays and silty clay loams below the surface layers.

The wetland is not in, or adjacent to, an area of special natural resource interest. Wildlife habitat in the wetland is classified as Class III, or of good quality. Because of its location near the confluence of the Menomonee River and Lilly Creek, the wetland could provide spawning habitat. That habitat is currently degraded because of sediment discharges from the upstream watershed and bank erosion along the Menomonee Manor Tributary.

The wetland provides insignificant flood storage capacity and shoreline erosion protection along the Tributary. The wetland would also provided only minimal sediment storage capacity, with such storage detrimental to the wetland function as spawning habitat.

Basin WD24 is recommended for the sole purpose of providing significant water quality benefits along the Menomonee Manor Tributary. The basin is an integral part of the overall plan to reduce nonpoint source pollutant loadings to Lilly Creek and the Menomonee River under existing and planned land use conditions and to reduce streambank erosion and streambed scour caused by frequently occurring floods. Such erosion and scour are existing problems which destroys habitat and would be expected to worsen as upstream development continues unless the recommended basin were constructed. If basin WD24 were not constructed, the functional values of the wetland downstream of the basin site would be degraded.

In order to accomplish the water quality control objectives established for the recommended basin, it must be so located that it would treat runoff from the commercial strip areas along W. Appleton Avenue as well as from the upstream residential areas. Because of the existing development patterns in the tributary area, there are not sufficient open-space sites available on which to locate decentralized detention basins which would adequately accomplish the established objectives. Location of the recommended basin on the southwest side of W. Appleton Avenue in the parking lot of the Kmart store in the adjacent commercial strip development would also not be feasible because the basin would take up almost all the parking area for the store. It was, therefore, concluded that there is no practicable alternative to the recommended centralized detention basin which would be expected to meet the water quality objectives set forth above. Thus, it is recommended that centralized basin WD24 be constructed in the western portion of the wetland at the location shown on Map 20 in Chapter VI of this report.

WETLAND CONSIDERATIONS RELATED TO THE PRELIMINARY RECOMMENDED MODIFICATION OF THE LILLY CREEK CHANNEL

Under the final stormwater management and flood control plan set forth on Map 20 in Chapter VI of this report, the modification of 2.08 miles of the Lilly Creek channel is recommended. The modification would begin at River Mile 0.66, between W. Appleton Avenue and W. Good Hope Road, and would extend upstream through the Chicago & North Western Railway to River Mile 2.74. Construction of the recommended modified channel would involve limited disturbance of two wetlands located in a secondary environmental corridor. The northern wetland is located along the west bank between River Miles 1.98 and 2.14; the southern wetland is located along the west bank between River Miles 2.26 and 2.32. The recommended modified channel would extend about 50 feet into the approximately 400-foot-wide northern wetland, disturbing about one acre of the 8.5-acre wetland, and about 40 feet into the 650-foot-wide southern wetland, disturbing about 0.3 acre of the 4.6-acre wetland. The portions of the wetlands which would be disturbed are classified as shoreland wetlands on the Village's wetland inventory map.

Under existing conditions, the total area draining to the northern wetland is about 50 acres, of which 16 percent is low-density residential development, 60 percent is cropland, and 24 percent is wetlands. Planned land use conditions call for 16 percent in low-density residential development, 60 percent in medium-density residential development, and 24 percent in wetlands.

Under existing conditions, the total area draining to the southern wetland is about 18 acres, of which 14 percent is industrial land, 9 percent is low-density residential development, 55 percent is cropland, and 22 percent is wetlands. Planned land use conditions call for 14 percent in industrial uses, 9 percent in low-density residential development, 55 percent in medium-density residential development, and 22 percent in wetlands.

The northern wetland was field inspected by the Commission's staff on January 10, 1985, at the time that the Village was attempting to implement its original channel modification plan, and again on March 28 and May 31, 1991, when a development proposal for the adjacent Mill Ridge subdivision was received by the Village. That wetland is classified as type T3K, corresponding to a broad-leaved, deciduous, forested, wet soil, palustrine wetland. Specifically, the wetland consists of a second-growth southern wet-mesic lowland hardwood forest with ephemeral ponds. Disturbances to the wetland noted during the field inspections included water level changes from ditching, selective tree cutting in the eastern portion of the wetland, past clear-cutting of trees in the western portion of the wetland, clearing of part of the eastern portion of the wetland for residential yards, and past grazing. No federal- or state-designated rare, threatened, or endangered species were observed during the field inspection. The southern wetland has an area of about 4.6 acres and is classified as an emergent marsh wetland with narrow-leaved vegetation on wet soils (E2K).

The soils in the northern wetland are classified as Ashkum silty clay loam, Colwood silt loam, and Drummer silt loam. Each of those soil types is classified as poorly drained and each generally occurs in areas with a high water table. The soil survey indicates that, below the surface layers, Ashkum soils generally consist of silty clays and silty clay loams; Colwood soils consist of silty clay loams, silt, and fine sand; and Drummer soils consist of silty clay loams, sand, and gravel. The excavation for the channel modification would be in Colwood and Drummer soils. Construction of the channel in the highly permeable sand and gravel layer of the Drummer soils could drain the wetland unless special construction measures are taken to seal the channel sides and bottom while still providing adequate slope stability.

The soil survey classifies the soils in the southern wetland as Ashkum silty clay loam and Drummer silt loam. In this case, the channel would also be constructed in Drummer soils, again necessitating special construction measures to avoid draining the wetland.

The wetlands are not in, or adjacent to, areas of special natural resource interest. Wildlife habitat in each wetland is classified as Class II, or of good quality.

Both wetlands provide some flood storage capacity; however, the loss of that capacity due to confinement of flood flows in the modified channel would be offset by the extensive centralized detention storage and overbank flood storage to be created in the subwatershed under the recommended stormwater management and flood control plan. Under existing conditions, the wetlands may afford some streambank erosion protection through a reduction in flow velocities; however, under the recommended channel modification adequate erosion protection would be provided and an additional 2.4-acre wetland area would be created in the west overbank immediately downstream of the existing northern wetland. Spawning habitat lost through elimination of a hydraulic connection between the northern and southern wetlands and the Lilly Creek channel during floods would be restored through the construction of both the downstream wetland and a 2.3 acre wetland in the upstream reach south of the Chicago & North Western Railway embankment.

There are three alternatives to the channel modification recommended along Lilly Creek in the reach adjacent to the two wetlands in question: 1) eliminating the recommended channel modification along all of Lilly Creek, resulting in an estimated $219,000 in flood damages to residential, industrial, and commercial structures during a 100-year recurrence interval flood occurring under planned land use and recommended drainage conditions, creating access problems along Manor Hills Boulevard and at the Bowling Green Industrial Park during floods, and limiting the effectiveness of the recommended stormwater drainage system for the Industrial Park;[2] 2) eliminating the channel modification only in the reach south of W. Mill Road and providing a six-foot-high drop structure in the channel at W. Mill Road, thereby eliminating the possibility of fish migration to the significant wetland spawning areas located south of Mill Road, leaving $84,000 in potential damages during a 100-year flood under planned land use and drainage conditions, creating access problems at the Bowling Green Industrial Park during floods, and limiting the effectiveness of the recommended stormwater drainage system for the Industrial Park; and 3) purchasing residential and industrial properties at an estimated total cost of $1,375,000, including relocation costs, and realigning the modified channel to the east of the proposed alignment.

Alternatives 1 and 2 are clearly unacceptable because they would greatly compromise the plan's flood control and stormwater drainage objectives and because Alternative 2 would eliminate the possibility of fish migration to important spawning areas. Alternative 3 would avoid disturbance of the wetlands, but the channel would be constructed in Drummer soils adjacent to the wetland and special design and construction measures would still be required to avoid draining the wetland. Also, the high cost of Alternative 3 makes it infeasible. Therefore, it was concluded that there is no practicable alternative to the recommended channel modification which would be expected to meet the objectives of this plan adequately. Thus, it is recommended that the channel be modified as proposed in Chapter VI of this report.

Because the two wetlands affected by the recommended channel modification are shoreland wetlands, it would be necessary to rezone the affected portion of the shoreland wetlands prior to construction of the channel modification. That rezoning would have to comply with the requirements of Chapter NR 117 of the Wisconsin Administrative Code. When the channel modification is considered within the overall plan objectives of water quality improvement, stream restoration and habitat improvement, streambank erosion control, stormwater drainage control, and flood control, the rezoning should be acceptable under the requirements of Chapter NR 117.

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[2] *As set forth in Chapter V of this report, structure floodproofing and removal flood control alternatives to channel modification were options previously rejected because of probable difficulties in achieving full implementation and because of their inability adequately to solve possible problems caused by secondary flooding, street flooding, and submergence of stormwater outlets.*