APPENDIX E

Bloomington Airport South Drainage & Water Quality Modeling Update (Dec 2008)

Bloomington Airport South Drainage and Water Quality Modeling Update

Prepared for

City of Bloomington

December 2008

Prepared by

Barr Engineering Co.

BARR

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Bloomington Airport South

Drainage and Water Quality Modeling Update

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1.0 Executive Summary

This report describes the results of the hydrologic, hydraulic, and surface water quality modeling analyses completed by Barr Engineering Co. for the Airport South Drainage District (ASDD) within the City of Bloomington. Previous modeling analyses completed for the ASDD include an XP-SWMM hydrologic and hydrologic model (originally developed and calibrated in 1998, and updated in 2002 and 2005) and a P8 water quality model (completed in 2003). As part of this project, the previously-developed models have been verified and/or updated to reflect the most current land use conditions and available data.

The ASDD is located in the northeastern corner of the City of Bloomington, bounded by Interstate-494 (I-494) to the north, Trunk Highway (TH) 77 (Cedar Avenue) to the west and the Minnesota River to the south and east (see Figure 2-1). The approximately 1,000-acre drainage district is a composite of commercial, industrial, residential, recreational, and conservation land uses, including portions of the Minnesota Valley National Wildlife Refuge and adjoining bluff land area. The ASDD receives stormwater inflows from the adjacent Smith Pond Drainage District, located west of TH 77 and south of I-494

The hydrology and hydraulics of the ASDD was modeled using XP-SWMM, Version 6.0, which uses rainfall and watershed information to generate runoff that is routed simultaneously through complicated pipe and overland flow networks. The water quantity modeling was used to help assess the flow patterns within the complex storm sewer network in the ASDD. The model was also used to evaluate the capacity of the existing storm sewer systems and assess whether the systems meet the desired level of service and protection. Surcharged conditions resulting from the 2-, 10-, and/or 100-year frequency events were identified for portions of the ASDD storm sewer network. The XP-SWMM model was also used to predict flood elevations of ponding basins and identify areas of inundation, such as streets and parking lots, from the 100-year frequency event. As future redevelopment occurs within the ASDD, the hydrologic and hydraulic modeling results can be used to identify and further refine stormwater management improvements.

In 2002, the City of Bloomington initiated the *Bloomington Airport South District Storm Water Treatment Feasibility Study* (SRF, 2002), which 1) evaluated the effectiveness of storm water treatment systems existing at the time, and 2) assessed future treatment strategies to maintain or improve the quality of water being discharged to Long Meadow Lake, part of the U.S. Fish and

Wildlife Service National Wildlife Refuge. The P8 model originally developed for the feasibility study was updated as part of this project to reflect the current conditions within the ASDD, including additional on-site stormwater treatment systems and recent upgrades to Pond C. The model was converted to the most recent P8 version (Version 3.4) and used to 1) estimate the quantity and quality of the surface runoff in the ASDD, and 2) evaluate the removal efficiencies of the existing water quality treatment systems for an ‘average’ climatic year and the 2-year, 24-hour Soil Conservation Service (SCS) Type II storm event.

Since much of the ASDD developed well before the era of water quality treatment requirements, stormwater runoff from portions of the area has historically received little or no water quality treatment prior to discharge into downstream Long Meadow Lake. However, the City’s implementation of onsite water quality treatment requirements for more recent development and redevelopment projects and construction of regional water quality treatment basins have resulted in a reduction in ‘untreated areas’ and improvements in the quality of water discharged to Long Meadow Lake. The P8 modeling results indicate that through on-site and regional Best Management Practices (BMPs) and naturally occurring wetlands, approximately 52% of the annual total phosphorus and 80% of the total suspended solids loads generated from the Smith Pond and Airport South Drainage Districts are removed prior to discharge to downstream Long Meadow Lake.

2.0 Background and Purpose

This report describes the results of the hydrologic, hydraulic, and surface water quality modeling analyses completed by Barr Engineering Co. for the Airport South Drainage District (ASDD) within the City of Bloomington. Previous modeling analyses completed for the ASDD include an XP-SWMM hydrologic and hydrologic model (originally developed and calibrated in 1998, and updated in 2002 and 2005) and a P8 water quality model (completed in 2003). As part of this project, the previously-developed models have been verified and/or updated to reflect the most current land use conditions and available data.

The ASDD is located in the northeastern corner of the City of Bloomington, bounded by I-494 to the north, Trunk Highway (TH) 77 (Cedar Avenue) to the west and the Minnesota River to the south and east (see Figure 2-1). The approximately 1,000-acre drainage district is a composite of commercial, industrial, residential, recreational, and conservation land uses. The developed portions of the drainage district include high-density retail development such as the Mall of America, a large number of hotels due to the proximity to the Minneapolis-St. Paul International Airport (MSP), high-density office and mixed-use space, and a low density residential land use area in the southern portion of the district. The ASDD also encompasses portions of the Minnesota Valley National Wildlife Refuge and adjoining bluff land area.

2.1 Drainage Patterns

Stormwater runoff from the Airport South Drainage District is conveyed from the area through a complex storm sewer network that has been repeatedly modified and improved in the past fifty years as development and redevelopment has occurred in the area. There are several locations throughout the Airport South storm sewer network where flow is redirected to alternate trunk storm sewer systems during large, high-flow runoff events. The main flow redirection junctions are identified in Figure 2-1. Stormwater runoff from the ASDD ultimately drains to Long Meadow Lake, located within the Minnesota River floodplain, through four outfalls that are operated and maintained by the City of Bloomington (see Figure 2-1).

The northeastern corner of the ASDD drains to the 80th Street trunk storm sewer system, which discharges to the north side of Long Meadow Lake via a 48-inch reinforced concrete pipe (RCP). The drainage area to the 80th Street system generally consists of commercial land use (hotels, office space, parking facilities). Runoff from this area receives limited water quality treatment prior to being discharged to Long Meadow Lake. The City recently installed an in-line underground stormwater treatment structure just east of 80th Street to remove pollutants from the runoff during low flow runoff events.

The Ceridian storm sewer outfall is located to the southwest of the 80th Street outfall and also discharges to Long Meadow Lake (Figure 2-1). The 36-inch corrugated metal pipe (CMP) system conveys runoff from the Ceridian property, the Health Partners redevelopment site, and an adjacent residential area. Runoff from a large portion of the drainage area receives onsite stormwater treatment, but no regional treatment is provided prior to discharge into Long Meadow Lake.

The Hogback trunk storm sewer is a large system that drains stormwater runoff from primarily high-density commercial and industrial areas within the ASDD, including a significant portion of the runoff from the Mall of America site. The storm sewer system consists of a 72-inch RCP system that conveys stormwater to the steep, unnamed ravine, with a 54-inch RCP located within the ravine, then a 72-inch outlet to Hogback Ridge Pond. The Hogback Ridge Pond, which is operated and maintained by the U.S. Fish and Wildlife Service (USFWS), ultimately discharges to Long Meadow Lake through a control structure located on the southwest side of the pond.

Pond C is a large regional stormwater detention pond located at the base of the bluff, just east of TH 77 (Cedar Avenue). Pond C receives stormwater from a significant portion of the ASDD, including portions of the commercial area surrounding the Mall of America, the residential area south of the Mall of America and generally west of Old Shakopee Road, drainage from TH 77 (Cedar Avenue) and inflows from the neighboring Smith Pond Drainage District located west of TH 77. Discharge from Pond C is conveyed to Long Meadow Lake through a 54-inch RCP.

2.2 Water Quality Treatment

In 2002, the City of Bloomington initiated the *Bloomington Airport South District Storm Water Treatment Feasibility Study* (SRF, 2002), which 1) evaluated the effectiveness of storm water treatment systems existing at the time, and 2) assessed future treatment strategies to maintain or improve the quality of water being discharged to Long Meadow Lake, part of the U.S. Fish and Wildlife Service National Wildlife Refuge. The feasibility study included the ASDD and the adjacent Smith Pond and Wright's Lake drainage areas to the west. The study evaluated several regional water quality treatment scenarios to assess future treatment opportunities, and made recommendations for stormwater treatment improvements. Among the recommendations were to upgrade Pond C, a regional stormwater treatment pond located just northeast of the TH 77 Bridge crossing the Minnesota River, within the Minnesota Department of Transportation (Mn/DOT) right-of-way. The City of Bloomington completed the recommended upgrades to Pond C in 2007.

The previously developed P8 model was updated to reflect the current conditions within the ASDD, including additional on-site stormwater treatment systems and recent upgrades to Pond C. No changes were made to the portions of the model representing the Smith Pond Drainage District. The model was converted to the most recent P8 version (Version 3.4), to simulate the quantity and quality of the surface runoff in the ASDD and evaluate the average annual removal efficiency of the existing water quality treatment systems. The runoff quality and system treatment efficiencies were also evaluated for the 2-year, 24-hour Soil Conservation Service (SCS) Type II storm event.

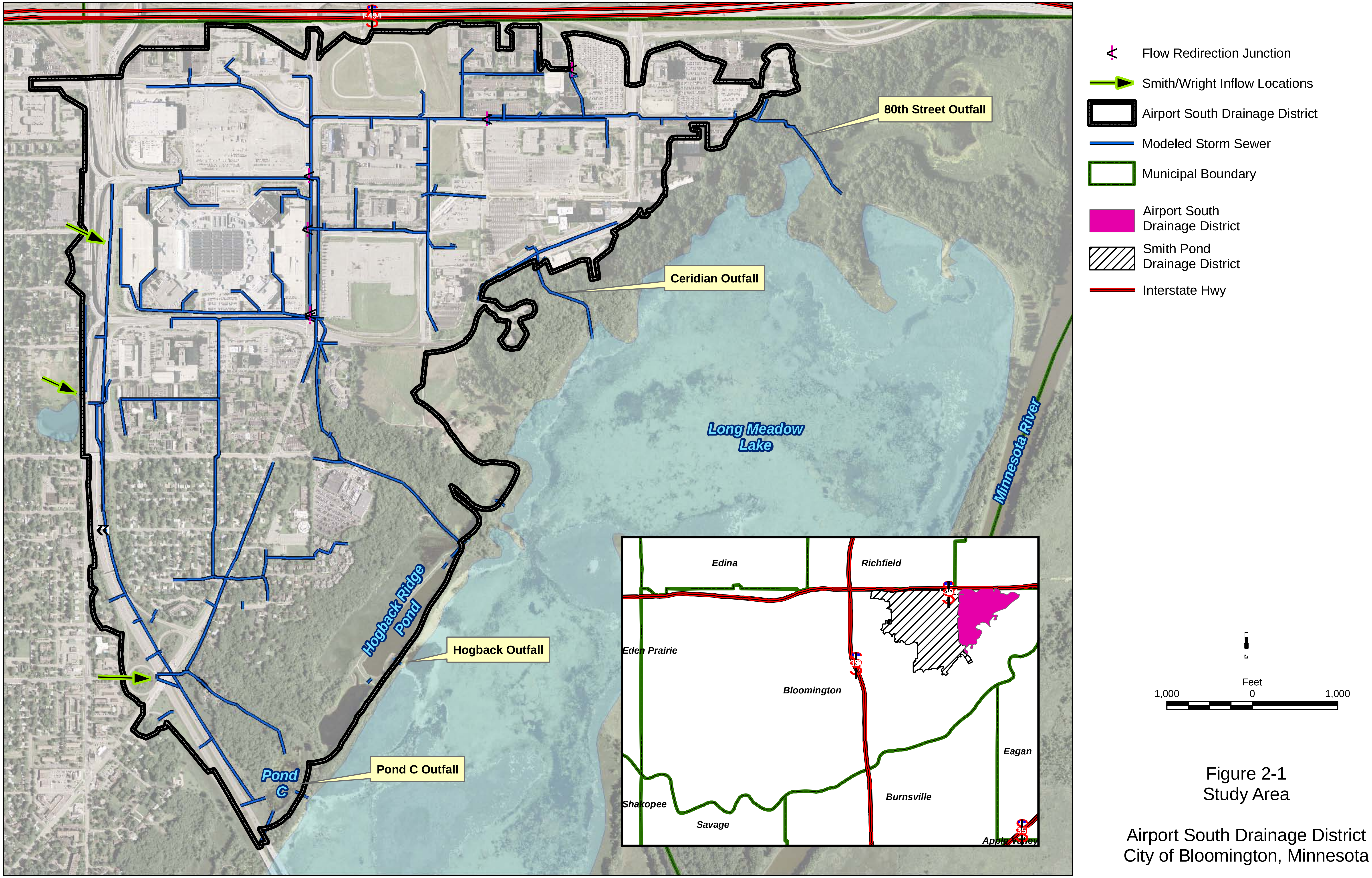


Figure 2-1
Study Area

Airport South Drainage District
City of Bloomington, Minnesota

3.0 Methodology for Hydrologic/Hydraulic Modeling

3.1 XP-SWMM Computer Model

The US E.P.A.'s Storm Water Management Model (SWMM), with a computerized graphical interface provided by XP Software (XP-SWMM), was used to model the hydrology and hydraulics of the ASDD. XP-SWMM uses rainfall and watershed information to generate runoff that is routed simultaneously through complicated pipe and overland flow networks. The model can account for detention in ponding areas, backwater conditions, surcharging of manholes, and backflow through pipes, all of which do occur within the study area. XP-SWMM Version 10.6 was used to simultaneously model the storm sewer and overland flow systems within the ASDD.

3.2 Hydrologic Modeling

Generation of storm water runoff was simulated using the SWMM Runoff Non-linear Reservoir Method in the XP-SWMM software. This method simulates hydrologic processes to determine the amount of rainfall that will infiltrate, evaporate, or remain on the ground surface and the amount of rainfall that will leave the watershed as runoff throughout the duration of a precipitation event. To predict the rate and volume of stormwater runoff from a watershed, it is necessary to develop input parameters to describe the physical characteristics of the watershed that impact the hydrologic processes. These input parameters are developed for each sub-drainage basin and are used to generate inflow hydrographs at various points in the stormwater system. Three major types of information are required by XP-SWMM for hydrologic modeling: (1) watershed data, (2) inputs regarding hydrologic processes, and (3) rainfall data. The methodologies used to develop the main hydrologic input parameters for these categories used are described in the following sections.

3.2.1 Watershed Data

Examination of the watershed characteristics for the study area involved assessments of topography and drainage patterns, soil types, land use and residential density, and the impervious fraction of the land in the watershed. ArcView geographic information system (GIS) software was used extensively in assessing the watershed characteristics.

3.2.1.1 Verify Existing and Delineate Additional Sub-Drainage Basins

The accuracy of hydrologic and hydraulic modeling is highly dependent upon the quality of the data input to the model. As such, it is important to delineate the contributing areas (sub-drainage basins) using the best information available, including topographic data and storm sewer information. The sub-drainage basins from previous modeling efforts were evaluated and revised based on the digital two-foot contour interval topographic data (1995, updated in 2005), 2006 orthophotography, and digital storm sewer system data. Significant revisions were made to the sub-drainage basins throughout much of the study area, based on the availability of digital storm sewer mapping for both the public and private storm sewer systems and updated topographic information. Sub-drainage basins were delineated at a scale that represents the direct drainage area to low points in the streets and other ponding areas (such as wetlands, ponds, parking lots), and at key connections to the storm sewer system. In areas where the direction of flow was not clear based on the digital topographic data, sub-drainage basin delineations were field verified. The delineated sub-drainage basins are shown in Figure 3-1.

Other revisions made to the delineation of sub-drainage basins reflected recent changes in land use based on 2006 orthophotography, as-built construction plans, or other development documents provided by the City of Bloomington. Examples of these areas include the Metropolitan Airports Commission (MAC) Runway Protection Zone (RPZ) area (north of American Boulevard and west of 24th Avenue), the IKEA retail development, the Minnesota Department of Transportation (Mn/DOT) Hiawatha Light Rail Transit (LRT) corridor, and the Bloomington Central Station redevelopment project. The Bloomington Central Station redevelopment project encompasses approximately 54 acres and includes the area known as the “Health Partners Campus”. The Bloomington Central Station redevelopment is being completed in a phased approach. The XP-SWMM model and related sub-drainage basin delineation have been updated to reflect completion of Phase 1 of the redevelopment, which represented ‘existing conditions’ at the time of the model development.

3.2.1.2 Land Use / Imperviousness

The imperviousness of a watershed is a key parameter in predicting the amount of runoff generated. The quantity of runoff generated from different land uses varies based on the imperviousness of the land. Land use characterized by high imperviousness (e.g., commercial areas) will generate higher runoff rates and volumes than land uses with lower imperviousness (e.g., residential areas).

The percentage of impervious area was estimated for each individual sub-drainage basin as an input parameter for the hydrologic model. The imperviousness of each sub-drainage basin was estimated using land use data for existing (2007) conditions. The 2007 land use used for the modeling update was originally developed for the City of Bloomington *Nondegradation Loading Assessment Report* (Barr Engineering Co., 2007). This land use layer was created based on 2007 parcel-based land use provided by the City of Bloomington. This land use layer was verified against the 2006 aerial photo, and for areas in question, the land use was field-verified and adjusted as necessary. The 2007 land use data includes the following categories: agriculture, commercial, developed park, forest, grassland, high-density residential, highway, industrial, institutional, low-density residential, medium-density residential, and water/wetland.

The existing (2007) land use information was used to estimate the total amount of impervious area within each sub-drainage basin as well as the amount of directly-connected impervious area. The directly-connected impervious fraction consists of the impervious surfaces that are “connected” directly to stormwater conveyance systems, meaning that flows do not cross over pervious areas. The total impervious and directly-connected impervious percentages used for the ASDD are consistent with the values applied for the *Nine Mile Creek/Bloomington Use Attainability Analysis (UAA)* (Barr Engineering Co., 2001).

The impervious percentages by land use as developed for the *Nondegradation Loading Assessment Report* (Barr Engineering Co., 2007) were not used for this study, with exception of the impervious percentage for agricultural land use. This was because the percentages by land use were typically lower than the values assumed in the *Nine Mile Creek/Bloomington UAA.* The values used in the *Nondegradation Loading Assessment Report* were based on a city-wide analysis comparing the City’s land use coverage with the 2002 Metropolitan Council imperviousness coverage for the Twin Cities metro area. However, land use within the ASDD is generally very dense with large amounts of imperviousness. Therefore, the higher impervious percentages for each land use were applied and verified through the calibration process (see Section 3.4).

Table 3-1 summarizes the Existing (2007) Bloomington land use categories within the ASDD and the associated total impervious and directly-connected impervious percentages.

Table 3-1 Percent Imperviousness by Land Use

Land Use	Total Impervious Percentage	Directly Connected Impervious Percentage
Agriculture	9	0
Commercial	90	80
Commercial- Mall of America	100	100
Developed Park	2	0
Forest	2	0
Grassland	2	0
High Density Residential	70	40
Highway	50	50
Industrial	90	80
Institutional	40	20
Low Density Residential	40	20
Medium Density Residential	55	30
Open Water	100	0

3.2.1.3 Watershed Width and Slope

The SWMM Runoff Non-linear Reservoir Method was used as the hydrograph generation method. This method computes outflow as the product of velocity, depth and a watershed width factor. The watershed "width" in XP-SWMM is defined as twice the length of the main drainage channel, with adjustments made for watersheds that are skewed (i.e., the areas on both sides of the main drainage channel are not equal). This factor is a key parameter in determining the shape of the hydrograph for each watershed and is often used as a calibration parameter. To determine the width parameter, the main drainage channel for each watershed was digitized in ArcView and a customized ArcView script was used to calculate the width based on the skew of the drainage path within the subwatershed. This methodology for calculation of the 'width' parameter is consistent with the approach used in the other areas modeled by Barr Engineering Co. within the City of Bloomington.

The subcatchment slope should reflect the average slope of the individual sub-drainage basin. The average slope (ft/ft) for each sub-drainage basin was calculated in ArcView Spatial Analyst using a digital elevation model (DEM) developed from the City's 2005 digital two-foot contour interval topographic data. The DEM data is in a grid format and the area-weighted average slope was calculated by measuring the differences in elevation between each grid cell within each individual sub-drainage basin.

Calculating average subcatchment slope using a DEM in ArcView greatly minimizes the amount of effort required to generate this watershed input. Generally, this methodology results in estimated average slopes for sub-drainage basins that are consistent with slopes calculated manually based on the measured elevation difference divided by the length of flow. However, in some cases where significant elevation changes occur within portions of a sub-drainage basin (e.g., retaining walls around parking lot perimeters), the area-weighted average slope can become skewed by the large differences in elevation between grid cells. In these cases, the calculated average slope for a sub-drainage basin can be unrealistically high.

3.2.1.4 Soils

The soil characteristics of a watershed can play a significant role in the amount of stormwater runoff generated. Soils with a high infiltration capacity (well-drained, sandy soils) have a low runoff potential, while soils with a low infiltration capacity (poorly drained, clayey soils) will generate more runoff. Soils data for portions of the ASDD was obtained through the Hennepin County Natural Resources Conservation Service (NRCS) Soils GIS database. This database includes the soil names and the hydrologic soil group (HSG) designation, which classifies soils into groups (A, B, C, and D) based on the infiltration capacity of the soil (well drained, sandy soils are classified as “A” soils; poorly drained, clayey soils are classified as “D” soils). However, much of the study area is classified as ‘urban’ or ‘undefined’ soils in the database and has not been assigned an HSG designation. For this portion of the study area, soil characteristics were assigned based on review of soil boring logs obtained by the City of Bloomington during the development or redevelopment process and verified through model calibration (see Section 3.4). The predominant soil types in the study area are SCS Type A (sandy) and B (sandy loam).

3.2.2 Assumptions for Hydrologic Processes

3.2.2.1 Infiltration

Infiltration was simulated in the XP-SWMM models using the Horton Infiltration equation. This equation is used to represent the exponential decay of infiltration capacity of the soil that occurs during rainfall or snowmelt events. The soil infiltration capacity is a function of the following variables: F_o (maximum or initial value of infiltration capacity), F_c (minimum or ultimate value of infiltration capacity), k (decay coefficient), and time. These infiltration parameters are used for the generation of runoff from the individual sub-drainage basins.

The actual values of F_o, F_c, and k are dependent upon soil, vegetation, and initial moisture conditions prior to a rainfall or snowmelt event. Because it was not feasible to obtain this detailed information for each sub-drainage basin through field samples, infiltration assumptions were made based on the soil types throughout the study area. Composite infiltration parameters (F_o and F_c) were calculated for each sub-drainage basin based on the fraction of each soil type within each individual sub-drainage basin. Global databases containing the infiltration parameters for each sub-drainage basin were developed and imported into the XP-SWMM model.

The values of F_o, F_c, and k applied for each Hydrologic Soil Group are summarized in Table 3-2. These values were selected based on the calibration results presented in the *Nine Mile Creek/Bloomington UAA* and modified for Type A and Type B soils based on the model calibration (see Section 3.4). Note that the ASDD is predominantly Type A and Type B soils.

Table 3-2 Horton Infiltration Parameters

Hydrologic Soil Group	F_o (in/hr)	F_c (in/hr)	k (1/sec)
A	6	0.7	0.00139
B	6	0.7	0.00139
C	2.0	0.1	0.00115
D	1.0	0.03	0.00115

The F_o and F_c values were determined for each sub-drainage basin by calculating a weighted average based on the given soil groups within each basin.

Global databases with the infiltration parameters for each sub-drainage basin were developed and read into the model. Each infiltration global database was assigned the same name as the respective sub-drainage basin.

3.2.2.2 Depression Storage

Depression storage represents the volume (in inches) of storage on the land surface that must be filled with rainfall prior to the occurrence of runoff. This parameter characterizes the loss or "initial abstraction" caused by such phenomena as surface ponding, surface wetting, interception and evaporation. The model handles depression storage differently for pervious and impervious areas. The impervious depression storage is replenished during dry simulation periods by evaporation. The water stored as pervious depression storage is subject to both infiltration and evaporation. Therefore, separate depression storage input values are required in XP-SWMM for pervious and impervious areas. Depression storage inputs were set within the general range of published values. Based on the model calibration discussed in Section 3.4, an impervious depression storage of 0.05 inches and a pervious depression storage of 0.20 inches were used in the XP-SWMM model. XP-SWMM also uses a “Zero Detention Storage” parameter to account for areas that generate immediate runoff (i.e., water surface areas). This parameter was estimated for each sub-drainage basin by dividing the water surface area by the directly connected impervious surface area.

3.2.2.3 Overland Flow Roughness

Overland flow is the surface runoff that occurs as sheet flow over land surfaces prior to concentrating into defined channels. A modified version of Manning’s equation is used to calculate the rate of overland flow in XP-SWMM. A key parameter in the Manning’s equation is the roughness coefficient, which accounts for the surface friction that occurs as water flows across different land surfaces. The shallow flows typically associated with overland flow result in substantial increases in surface friction. As a result, the roughness coefficients typically used in open channel flow calculations are not applicable to overland flow estimates. These differences can be accounted for by using an effective roughness parameter instead of the typical Manning’s roughness parameter.

Typical values for the effective roughness parameter are published in the *HEC-1 User’s Manual*, September 1990 and in *Engineering Hydrology: Principles and Practices* (Ponce, 1989). The overland flow roughness values for pervious and impervious areas were used as adjustment parameters during the calibration process. The overland flow roughness for pervious and impervious areas that resulted in the best fit to the observed calibration data were 0.20 and 0.04, respectively. An area weighted pervious roughness was determined for each sub-drainage basin in the study area by weighting the pervious area and unconnected impervious area.

3.2.3 Rainfall Data

Rainfall data was collected for the model calibration process from a continuous-recording tipping bucket rain gauge located within the ASDD. The gauge measured rainfall in 5-minute intervals for a time period between May and July 1998. The rainfall data for five storm events within this time period were input into XP-SWMM to simulate the runoff from these events and calibrate the predicted flows with observed flows from within the watershed. See Section 3.4 for additional information on the calibration process.

After calibration of the model, rainfall hyetographs were used as inputs to the XP-SWMM models to predict flood elevations for the 2-, 10-, and 100-year frequency, 24-hour precipitation events (2.75. 4.15, and 6.0 inches, respectively). The hypothetical SCS Type II 24-hour rainfall hyetograph was used for each of the frequency events. Rainfall amounts for the modeled events were obtained from the U.S. Department of Commerce's Technical Paper No. 40 – Rainfall Frequency Atlas of the United States (TP40).

3.3 Hydraulic Modeling

The stormwater runoff hydrographs generated by the XP-SWMM 'Runoff Mode' are routed through the storm sewer, ponding, and overland flow network in the 'Hydraulics Mode' of the model. XP-SWMM has advanced hydraulic capabilities and can handle complex hydraulic situations such as large drainage networks, detailed hydraulic structures, natural channel stream flow, detention in ponding areas, backflow in pipes, surcharging of manholes, and impacts of tailwater conditions on upstream storage or flows. The XP-SWMM hydraulics model allows manhole surcharging (water flowing out of a storm sewer manhole as opposed to flowing into a manhole). In addition, XP-SWMM assumes that this water disappears, or is 'lost' from the system, when it exceeds the respective spill crest elevation unless accounted for by the user.

To prevent stormwater from being 'lost' from the system, the ASDD was modeled using a two-tiered hydraulics network, one network of storm sewers and the other a network of overland flow paths (generally representing street flow and/or natural drainage ways). These two networks were generally connected by manholes, except when the overland flow paths represented natural drainage ways such as ditches or ravines. Upon surcharging of a manhole, stormwater is conveyed through the overland flow network to a downstream storm sewer inlet or ponding area.

The data required and assumptions made for the hydraulic modeling in the ASDD are summarized in the following subsections.

3.3.1 Storm Sewer Network

The storm sewer network modeled in XP-SWMM was generally limited to the City's trunk storm sewer system, and excluded most smaller storm leads and private systems. The City of Bloomington provided detailed information for the City's trunk storm sewer system in GIS format, including pipe size, type, length, invert elevations and top of casting elevations. Although the top of casting elevations for the catch basins and manholes of the trunk storm sewer were provided in the GIS database, many of the reported top of casting elevations conflicted with the available topographic information, varying by as much as several feet in some cases. Consequently, the top of casting elevations used in the XP-SWMM model were based on the corresponding elevation from the GIS digital elevation model. All nodes and links representative of manholes, catch basin manholes, and pipes were labeled according to the City's labeling convention in their GIS database. All elevations entered into the model are in feet above Mean Sea Level (MSL), based on the NGVD 29 datum. A roughness coefficient (Manning's "n") for all concrete pipes was assumed to be 0.013 (the typical design value). Roughness coefficients for other pipe materials were based on the guidance in *Open Channel Hydraulics* (Chow, 1959).

In portions of the ASDD, the privately-owned parking lots contain a notable amount of flood storage. In such cases, the sub-drainage basins were delineated to these low areas and the private storm sewer systems and leads that connect these areas to the City's trunk system were also included in the XP-SWMM modeling. The GIS information available for the small storm sewer leads and privately-owned systems was generally limited to spatial information, and detailed attribute information was not provided. If pipe size information for these systems was available, the associated invert elevations were estimated based on the invert elevations of the downstream trunk system and a reasonable pipe slope (0.1%). If pipe size was not available in the GIS database, the information was obtained from as-built plans provided by the City.

Although the private storm sewer systems were generally not modeled in detail, the City requested that the Mall of America system be modeled in detail. This request was in response to recent flooding that has occurred at several locations within the Mall of America property, most notable the Hiawatha LRT Station in the southeast corner of the site. Information for this storm sewer system was obtained from as-built plans provided by the City from the Mall of America and the Mn/DOT Hiawatha LRT Project.

The outlets from ponding areas were modeled based on as-built information provided from the City. Outlets from ponding areas that may be inlet-controlled were modeled in XP-SWMM assuming a groove end projecting pipe inlet condition. This model condition allows XP-SWMM to determine the controlling flow condition in the outlet pipe (i.e., is the flow in the pipe controlled by the inlet size, barrel capacity, or tailwater conditions) and accurately estimate the water surface elevation of the pond. The normal water level (NWL) for each ponding basin was set at the outlet control elevation or at a downstream control elevation, with exception of the large infiltration basin in sub-drainage basin APS-42, which functions as an infiltration basin. Appropriate manhole junction losses were entered into the model based on the storm sewer configuration for each node included in XP-SWMM. Manhole junction losses were estimated based on the methodology presented in *Modern Sewer Design* (American Iron and Steel Institute, 1980) or other suitable references.

3.3.2 Overland Flow Network

As previously mentioned, the XP-SWMM model incorporated a two-tiered routing system: a network of storm sewers and another network of overland flow paths (generally representing street flow and/or natural drainage ways). The following stepwise procedure was used as a guide for the overland flow network data entry at selected locations until water that was otherwise lost from the system is "captured." Varying levels of these steps will be iteratively implemented to "capture" the water at any one given location.

1) Adding storage to modeling nodes (manholes) based on the two-foot topographic information to account for surface ponding in streets, parking lots, etc.
2) Addition of overland flow paths with the following characteristics
    a) Overland flow along streets
        i) Trapezoidal channels with
            (1) Bottom width = approximated based on street width
            (2) Side slopes = 1H:1V
            (3) Manning's "n" for the surface flow channels set equal to 0.014 for flow down paved streets
            (4) Channel depth = 1 foot
    b) Natural overland flow paths
        i) Trapezoidal channels with
            (1) Bottom width = variable based on topographic information
            (2) Side slopes = variable based on topographic information
            (3) Manning's "n" where overland flow is clearly over vegetated areas or onto boulevards a Manning's "n" of 0.03 will be used
            (4) Channel depth = 1 foot
3) Increasing overland flow depth, if consistent with the topographic information
4) Raising the spill crest elevation if the nearby pond's water surface exceeds the node spill crest elevation and the storage is accounted for at the storage node (pond)

5) Route the water out of the system if indicated on storm sewer maps or as-built drawings (i.e., a possible out of district overflow location)
6) Activate “Ponding Allowed” in the model for a given node if indicated by the street and manhole rim elevations but not reflected on the two-foot topographic information

3.3.3 Inflows from the Smith Pond Drainage District

The Smith Pond Drainage District is located south of Interstate-494 and west of TH 77, adjacent to the ASDD. The Smith Pond Drainage District covers approximately 1,700 acres and includes Smith Pond and Wright’s Lake. There are several locations where flows from the Smith Pond district flow into the Airport South storm sewer system. These locations are shown in Figure 3-1 and discussed briefly below.

3.3.3.1 82nd Street Inflow

There is a 42-inch RCP trunk storm sewer system along 82nd Street in the Smith Pond Drainage District that connects with the TH 77 system. Flow through this storm sewer connection is controlled by a special regulator structure that restricts flows from the Smith Pond drainage area during times of high flow in the TH 77 storm sewer system. For the Airport South modeling update, inflow hydrographs generated from the previously developed Smith Pond XP-SWMM model were imported into the updated Airport South XP-SWMM model to represent predicted flows through the regulator for the 2-, 10-, and 100-year frequency, 24-hour events.

3.3.3.2 Wright’s Lake Outlet

Wright’s Lake is located directly west of TH 77 and north of 86th Street, within the Smith Pond Drainage District. The lake receives discharge from the upstream Smith Pond as well as local drainage. The Wright’s Lake outlet is located on the east side of the lake and connects to the TH 77 trunk storm sewer system, which eventually drains to Pond C. The original outlet from Wright’s Lake was a 36-inch CMP. A 1978 hydrologic and hydraulic analysis completed by Barr Engineering Co. for the Smith Pond-Wright's Lake storm sewer system recommended installing a larger-capacity outlet (30-inch weir into a 60-inch RCP) from Wright's Lake, which would result in a 100-year peak discharge of approximately 260 cfs and a reduced 100-year flood elevation to 810.8 ft MSL (Barr Engineering Co., 1978). The report indicates that Mn/DOT agreed to provide a 60-inch outlet pipe from Wright's Lake and as-built plans dated October 29, 1980 from the City of Bloomington indicate that this recommendation was implemented.

As-built construction plans from a subsequent project on TH 77 indicate that changes were made to the TH 77 storm sewer system, in that a portion of the 60-inch system that served as the outlet from Wright's Pond was removed and replaced with a 48" RCP. As such, it appears that the current outlet capacity from Wright's Lake is limited by the 48" RCP, which differs from the recommended outlet capacity in Barr's 1978 study.

For the Airport South modeling update, inflow hydrographs generated from the previously developed Smith Pond – Wright's Lake XP-SWMM model were imported into the updated Airport South XP-SWMM model to represent predicted discharge for the 2-, 10-, and 100-year frequency, 24-hour events. Based on the existing Smith Pond – Wright's Lake XP-SWMM model, the peak discharge from Wright's Lake from the 100-year frequency, 24-hour event is approximately 140 cfs and the high water elevation in Wright's Lake exceeds 812 ft MSL. Review of the digital two-foot topography for this area indicates that Wright's Lake would likely overflow to TH 77 at an approximate elevation of 811 ft MSL, which is not accounted for in the existing Smith Pond – Wright's Lake XP-SWMM model. The existing model also does not 'capture' stormwater that surcharges from the modeled storm sewer system, thus a significant volume of runoff from the Smith Pond drainage area (319 acre-feet for the 100-year event) is not accounted for (i.e., 'lost' from the system).

As a result of the modeling deficiencies mentioned above, there is significant uncertainty in the outflows from Wright's Lake predicted in the Smith Pond – Wright's Lake XP-SWMM model. The absence of an overflow conveyance from Wright's Lake to TH 77 and the loss of significant runoff volumes from the system make it difficult to confidently predict the high water elevation in Wright's Lake and associated outflows to the ASDD. Since revisions to the Smith Pond – Wright's Lake XP-SWMM model to correct the aforementioned deficiencies would require effort well beyond the scope of this project, City staff has indicated that the Airport South model updates should proceed based on the predicted outflow hydrographs from the existing Smith Pond-Wright's Lake model. As such, the modeling results for the TH 77 storm sewer network and the downstream Pond C should be considered with caution.

3.3.3.3 90th Street Inflow

The 90th Street storm sewer in the Smith Pond Drainage District consists of parallel 42-inch RCP and 66-inch RCP trunk storm sewer systems that combine into one 66-inch RCP system near the west side of the TH 77 and Old Shakopee Road intersection. For the Airport South modeling update, inflow hydrographs generated from the previously developed Smith Pond XP-SWMM model were imported into the updated Airport South XP-SWMM model to represent predicted flows through the parallel storm sewer system for the 2-, 10-, and 100-year frequency, 24-hour events.

3.3.4 Tailwater Effects

Stormwater runoff from the ASDD ultimately drains to Long Meadow Lake, which is located within the Minnesota River floodplain. The outfalls to Long Meadow Lake were modeled in XP-SWMM as free outfalls (i.e., no tailwater effects). The Minnesota Department of Natural Resources (DNR) Ordinary High Water Level (OHWL) for Long Meadow Lake is 695.5 ft MSL, which is below the outfall from the Hogback Ridge ponds. The downstream invert elevations of Pond C, 80th Street and Ceridian outfalls are slightly below the Long Meadow Lake OHWL, so the outfall pipes may be partially submerged at times. However, the submerged conditions are unlikely to have a significant impact on the discharge capacity.

The Lower Minnesota Flood Plain Study, completed by the U.S. Geologic Survey, Army Corps of Engineers and the Lower Minnesota River Watershed District in 2004, indicates that the 100-year flood elevation for the Minnesota River at the Long Meadow Lake area is approximately 714 ft MSL. At this flood elevation the entire Long Meadow Lake and adjacent Hogback Ridge ponds and Pond C would be inundated by approximately 18 feet of water. However, the hydrologic conditions that would result in the Minnesota River reaching the predicted flood levels are not synonymous with the ‘critical’ conditions that would result in peak flood elevations in the ASDD. As such, the flood levels reported in this study reflect flood conditions based on locally ‘critical’ events.

3.4 Model Calibration

An XP-SWMM model of the ASDD was originally developed in 1998 by Montgomery Watson. At the time, the model was calibrated based on stormwater monitoring data collected from two gauges located in small isolated watersheds in the ASDD between June 20 and July 14, 1998. As part of this project, Barr re-evaluated the calibration to verify that the results from the updated XP-SWMM model closely represent runoff conditions observed during the 1998 monitoring period. The XP-SWMM model was re-calibrated to ensure that the parameters revised since development of the previous Airport South XP-SWMM model result in a good fit with the observed data, including imperviousness assumptions and sub-drainage basin delineations. The calibration process included modifications to numerous hydrologic parameters to accurately represent 1998 observed runoff volumes, peak runoff rates, and runoff timing. The results of the calibration are described in further detail below.

3.4.1 Calibration Sites

Monitoring gauges were installed at two locations within the ASDD to collect continuous flow data. The two locations were selected to represent areas of varying development conditions. A flow meter was installed within a 36-inch RCP storm sewer located beneath Metro Drive, just north of 80th Street (Figure 3-2). The tributary area to this flow monitoring gauge is approximately 22 acres and is predominantly commercial land use with high imperviousness. A second flow meter was installed in a 27-inch RCP storm sewer located in a residential area near the intersection of 88th Street and Old Shakopee Road (Figure 3-2). The tributary area to this flow monitoring gauge is approximately 29 acres and is comprised of low- and medium-density residential and institutional land uses.

3.4.2 Calibration Data

Based on information provided in a memorandum to the City of Bloomington regarding the XP-SWMM Model Calibration, Montgomery Watson indicated that the monitoring gauges were installed in early May 1998 and removed in late July 1998 (Montgomery Watson, 1998). The gauges recorded flow depth and velocity at 5-minute intervals. Flow rates were then calculated based on the recorded velocity and flow area.

During the flow monitoring period, a continuous recording rainfall gauge was used to collect precipitation depths at 5-minute intervals. The rainfall gauge was located in the right turn lane island at northbound Old Shakopee Road and Killebrew Drive (southeast of the Mall of America).

Based on the Montgomery Watson memorandum, the gauges captured five runoff events for which there was good flow data at both flow gauge locations and good rainfall data. The five rainfall events used for calibration represent a wide range of rainfall magnitudes, durations and intensities, as summarized in Table 3-3. Figures 3-3 through 3-7 show the distribution of rainfall for each event. The mix of land use types for each of the calibration sites, combined with the variety of storm events monitored, provides a means for calibrating the model based on the variability of the observed stormwater monitoring data. Additional information on the data used for calibration can be found in the Montgomery Watson December 2, 1998 memorandum (Appendix A).

Table 3-3 Summary of Calibration Rainfall Events

Event Date	Rainfall Depth (in.)	Rainfall Duration (hrs:min)	Peak Intensity (in./hr)
June 20, 1998	0.54	1:45	2.2
June 24, 1998	0.99	6:55	1.2
June 25, 1998	0.45	0:50	2.5
June 26, 1998	3.01	7:40	4.1
July 14, 1998	1.38	4:40	1.3

3.4.3 Calibration Method

Multiple calibration model runs were conducted using variations of numerous hydrologic parameters with the goal of closely matching the observed and modeled runoff hydrographs for each storm event at each calibration site. Modeling results indicated that with exception of the June 26, 1998 storm event, the intensities of the rainfall events used for calibration were not high enough to generate runoff from the pervious areas of the calibration watersheds (all rainfall upon pervious areas was intercepted, infiltrated and/or stored onsite). As such, the calibration process focused heavily on adjustment of hydrologic parameters affecting runoff from impervious areas, such as the percentage of impervious area, impervious depression storage, and the impervious roughness coefficient. The runoff hydrographs were insensitive to adjustments in the width parameter. Modeling results indicated that the intensity of the June 26, 2008 storm was high enough to generate runoff from the pervious areas. This storm event was used to calibrate several hydrologic parameters affecting runoff from pervious areas, such as infiltration parameters, pervious depression storage, pervious roughness coefficient, and the percentage of impervious area.

The calibration process focused on comparison and calibration of the observed and modeled runoff hydrographs for each representative storm at each calibration site. The comparison of observed and modeled runoff was focused on verifying/calibrating peak runoff rates, timing, and the general shape of the hydrographs. The results of the hydrograph calibration are discussed in further detail below.

The calibration process also included evaluation of the runoff volume. Observed and modeled runoff coefficients were calculated for each storm event at each calibration site. The runoff coefficients were calculated from the fraction of rainfall measured/modeled as runoff using the equation:

Runoff Coefficient = Measured/Modeled Runoff Volume/(Total Rainfall * Drainage Area)

3.4.4 Calibration Results

Comparisons of the observed and modeled hydrographs from the two calibration sites are shown for each rainfall event in Figures 3-8 through 3-12. Overall, the modeled hydrographs reflect a good fit with the monitored runoff. In some cases, the modeled hydrographs do not reflect the same shape as the observed hydrograph or are 'missing' runoff peaks or troughs throughout the storm events, in comparison with observed conditions. This is likely a result of variation in the amount and timing of precipitation that occurred at each calibration site in comparison to the recorded precipitation at Killebrew Drive and Old Shakopee Road, as the spatial variability in rainfall patterns can be significant at times.

The Nash-Sutcliffe efficiency index is a widely used statistic for assessing the goodness of fit of hydrologic models. The Nash-Sutcliffe coefficient of efficiency (E_f) was calculated for each storm event at the two calibration stations to evaluate the goodness of fit of the modeled hydrographs with observed conditions. The Nash-Sutcliffe coefficient of efficiency equation is shown below:

$$E_f = 1 - \frac{\sum^{n}\left(\hat{Y}_i - Y_i\right)^2}{\sum\left(Y_i - \overline{Y}\right)^2}$$

Where,

$\hat{Y}_i$ and Y_i = predicted and measured flow values, respectively;

$\overline{Y}$ = mean of the measured flow values; and

n = sample size.

The results are summarized in Table 3-4. In general, the calculated efficiency indexes indicated a good fit between observed and modeled conditions. Low values of E_f can be the result of model bias in the calibration, with bias resulting from either differences in the magnitude of flow or time offset for time-dependent models (McCuen et al, 2006). Low values of E_f can also be sensitive to parameters such as sample size, time period, or outliers in observed flow.

Table 3-4 Nash-Sutcliffe Efficiency Indexes for Model Calibration

Storm Event	88th Street Calibration Site	Metro Drive Calibration Site
	Efficiency Index (E_f)	Efficiency Index (E_f)
June 20, 1998	-0.85	0.82
June 24, 1998	-1.11	0.64
June 25, 1998	0.78	0.93
June 26, 1998	0.54	0.57
July 14, 1998	0.54	0.75

As mentioned above, the calibration process also included evaluation of the runoff volume. Monitored and modeled runoff coefficients were calculated for each storm event at each calibration site. Table 3-5 summarizes the runoff coefficients for observed conditions and the calibrated model results. In general, the overall modeled runoff coefficients closely matched the runoff coefficients from observed conditions.

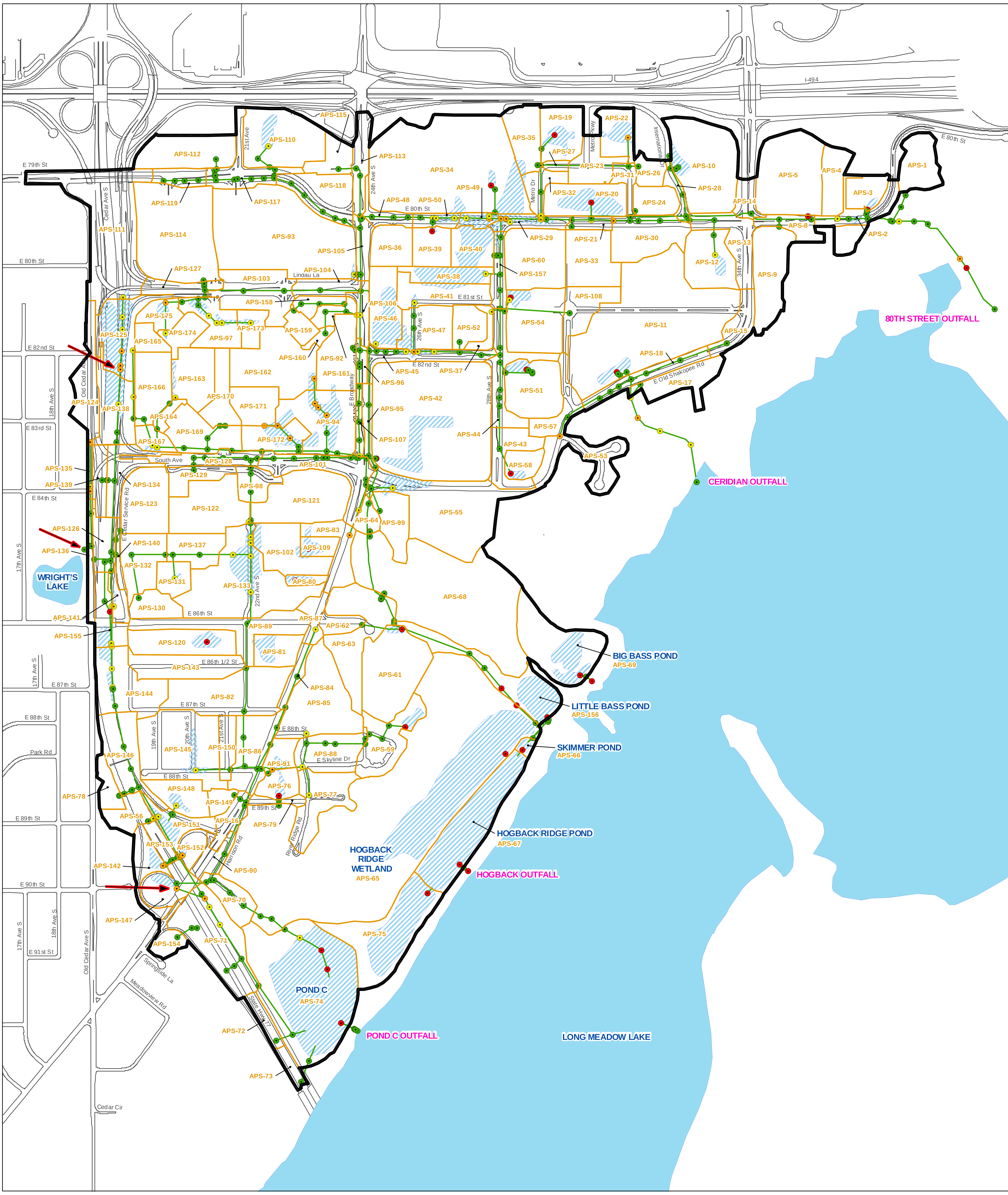
Table 3-5 Runoff Coefficients based on Observed and Modeled Conditions

Storm Event	88th Street Calibration Site			Metro Drive Calibration Site		
	RO Coeff. [Monitored]	RO Coeff. [XP-SWMM]	% Difference [XP-SWMM – Monitored]	RO Coeff. [Monitored]	RO Coeff. [XP-SWMM]	% Difference [XP-SWMM – Monitored]
June 20, 1998	0.13	0.21	38	0.63	0.70	10
June 24, 1998	0.18	0.22	19	0.61	0.73	16
June 25, 1998	0.18	0.21	15	0.62	0.69	10
June 26, 1998	0.20	0.29	30	0.71	0.81	12
July 14, 1998	0.14	0.23	36	0.68	0.75	10
Average =	0.17	0.23	28	0.65	0.74	12

The final calibrated hydrologic parameters were selected through comparison of the observed and modeled hydrographs from multiple calibration model runs using variations of numerous hydrologic parameters. The five storm events used for calibration represented a wide range of rainfall magnitude, intensity, and duration. As such, the hydrologic parameters associated with the 'best fit' often varied for each storm event and/or calibration site. The hydrologic parameters that were selected resulted in the best fit to observed conditions for the most storm events and calibration sites. The final calibrated hydrologic parameters and the ranges of values evaluated for each parameter during the calibration process are summarized in Table 3-6.

Table 3-6. Hydrologic Calibration Parameters

Calibration Parameter	Calibration Range		Final Calibration Values
	Min	Max	
Depression Storage, Pervious	0.1	0.20	0.20
Depression Storage, Impervious	0.0	0.20	0.05
Overland Flow Roughness, Pervious	0.10	0.35	0.20
Overland Flow Roughness, Impervious	0.01	0.10	0.04
Initial Infiltration Capacity (F_o), Type A Soils	5.0	6.0	6.0
Ultimate Infiltration Capacity (F_c), Type A Soils	0.38	0.70	0.7
Decay Coefficient, Type A Soils	0.0008	0.00139	0.00139
Initial Infiltration Capacity (F_o), Type B Soils	3.0	6.0	6.0
Ultimate Infiltration Capacity (F_c), Type B Soils	0.23	0.7	0.7
Decay Coefficient, Type B Soils	0.0008	0.00139	0.00139



Legend

- Airport South Drainage District Boundary
- Sub-drainage Basins

Node/Manhole Surcharge Condition

- 2, 10, & 100 year
- 10 & 100 year
- 100 year
- No Surcharge
- Modeled Storm Sewer
- SmithWright Inflow Locations
- 100-Year Inundation Areas
- Ponds Outside of Study Area
- Streets

N

Feet

0 500 1,000 1,500 2,000 2,500 3,000 3,500 4,000

Meters

0 200 400 600 800 1,000

Figure 3-1

XP-SWMM MODEL RESULTS
Airport South Drainage District
City of Bloomingtion, MN

Barr Footer: Date: 12/1/2008 9:23:20 AM File: I:\Projects\23\27\I40\Maps\Reports\Figure_CalibrationAreas_11x17.mxd User: jak2

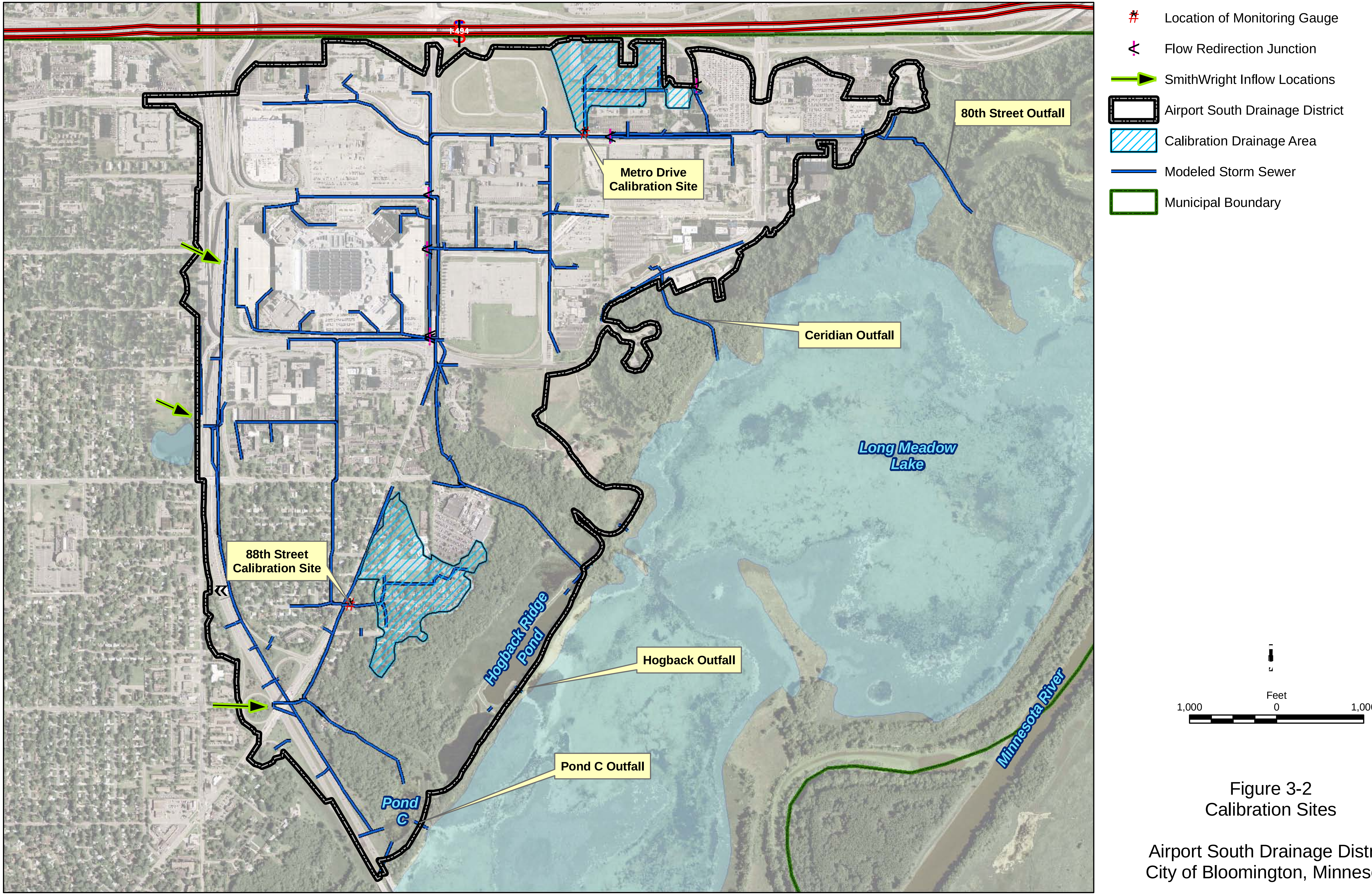


Figure 3-2
Calibration Sites

Airport South Drainage District
City of Bloomington, Minnesota

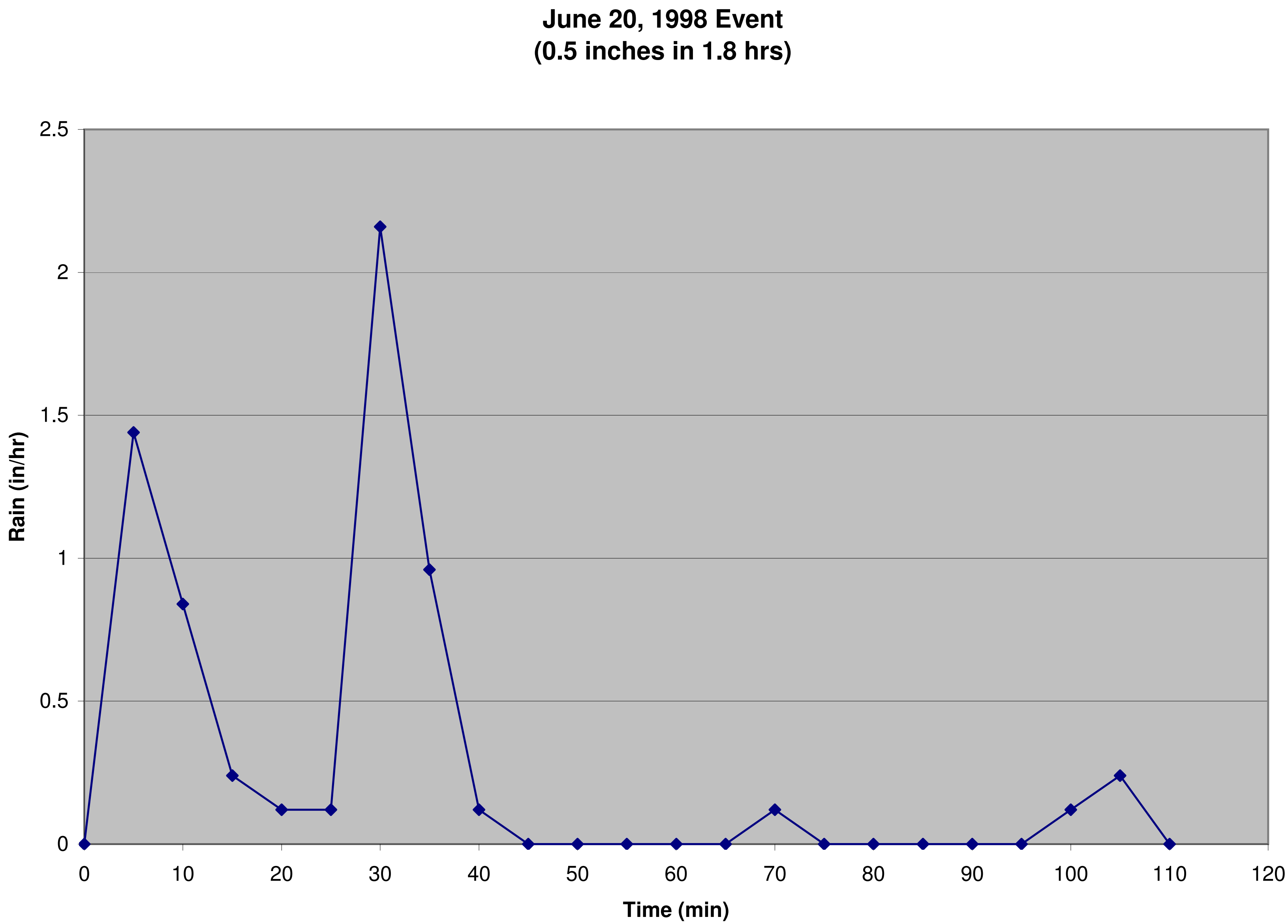

June 20, 1998 Event
(0.5 inches in 1.8 hrs)
Rain (in/hr)
2.5
2
1.5
1
0.5
0
0
10
20
30
40
50
60
70
80
90
100
110
120
Time (min)

Figure 3-3

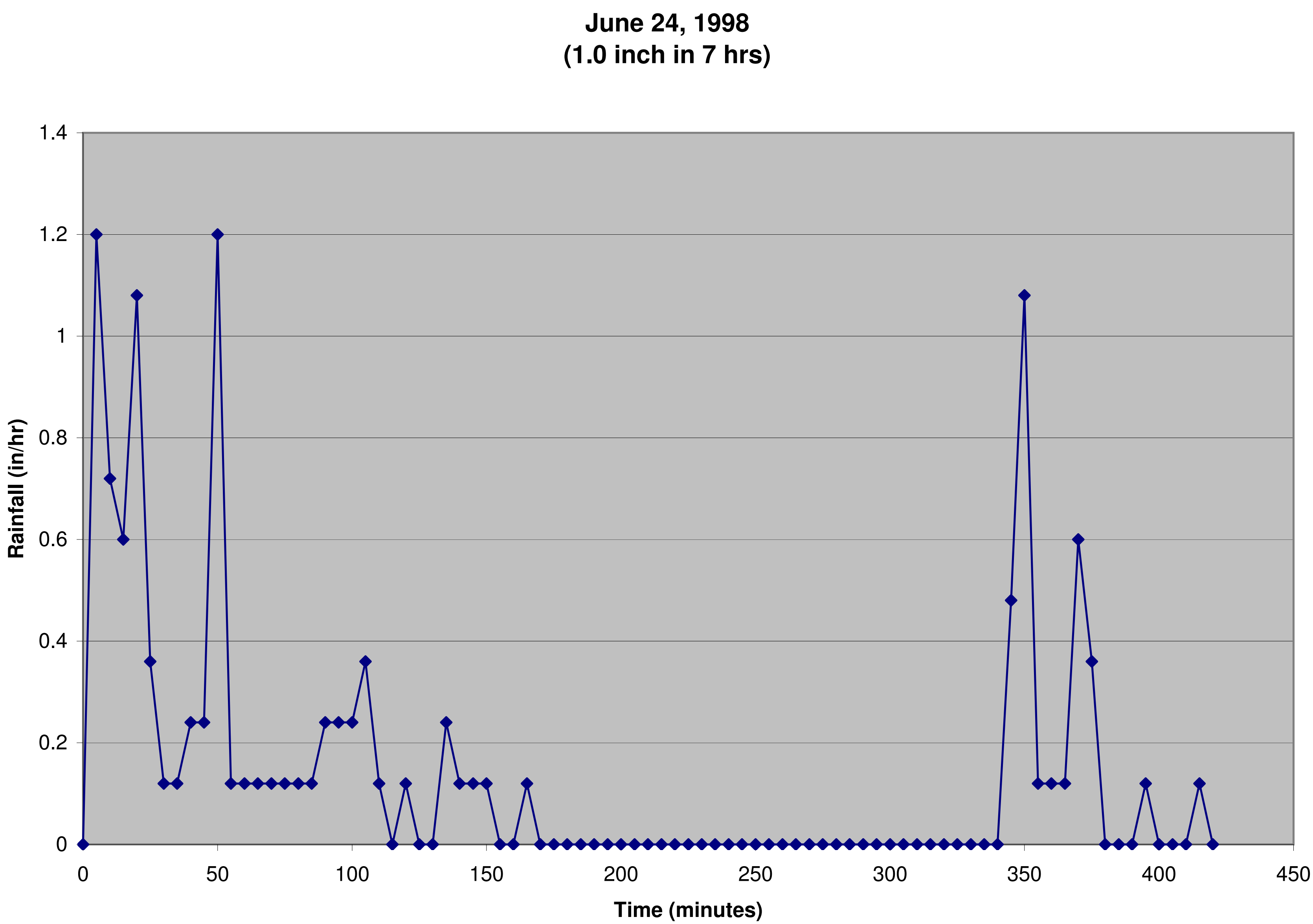

June 24, 1998
(1.0 inch in 7 hrs)
Rainfall (in/hr)
Time (minutes)
0
0.2
0.4
0.6
0.8
1
1.2
1.4
0
50
100
150
200
250
300
350
400
450

Figure 3-4

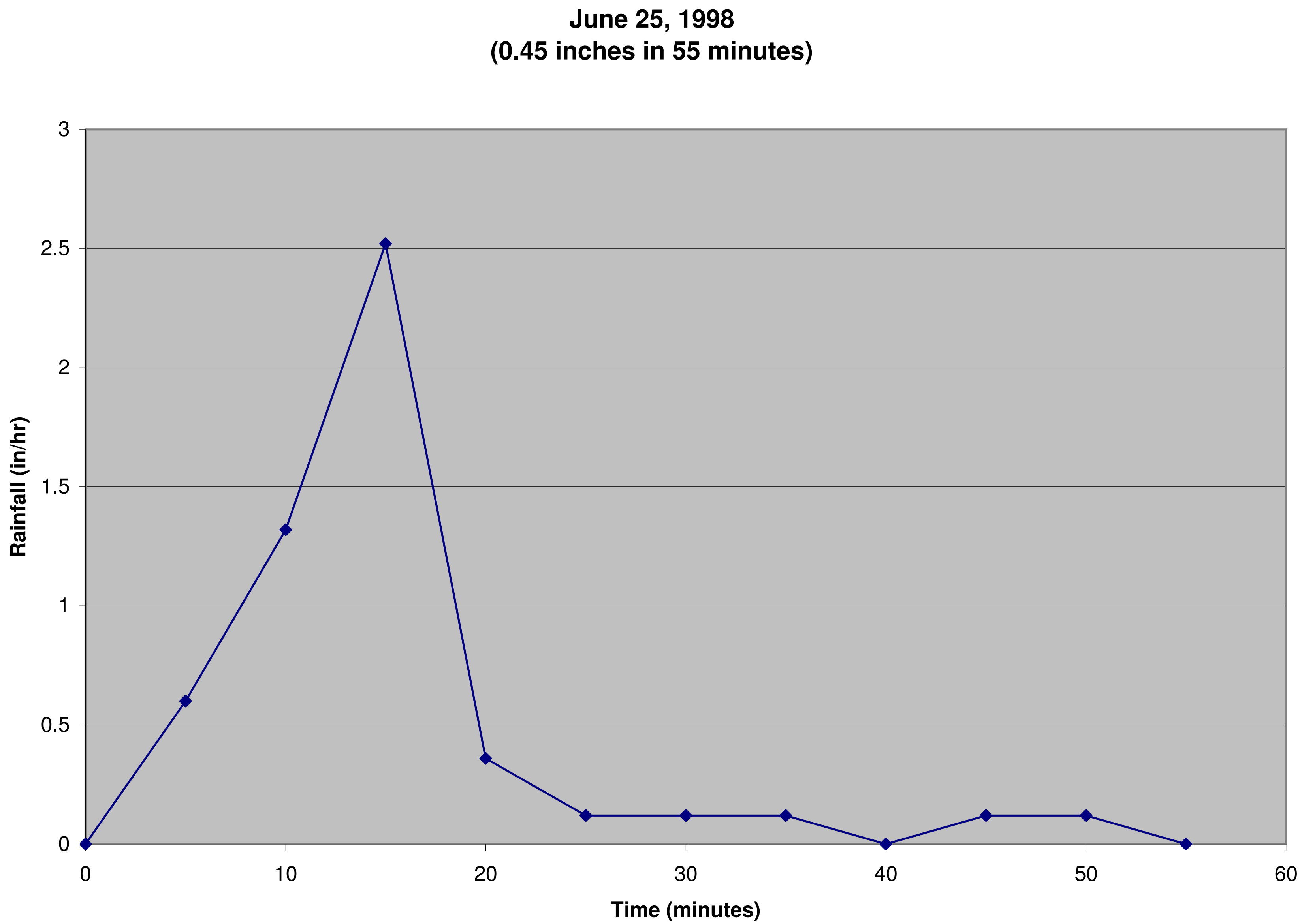
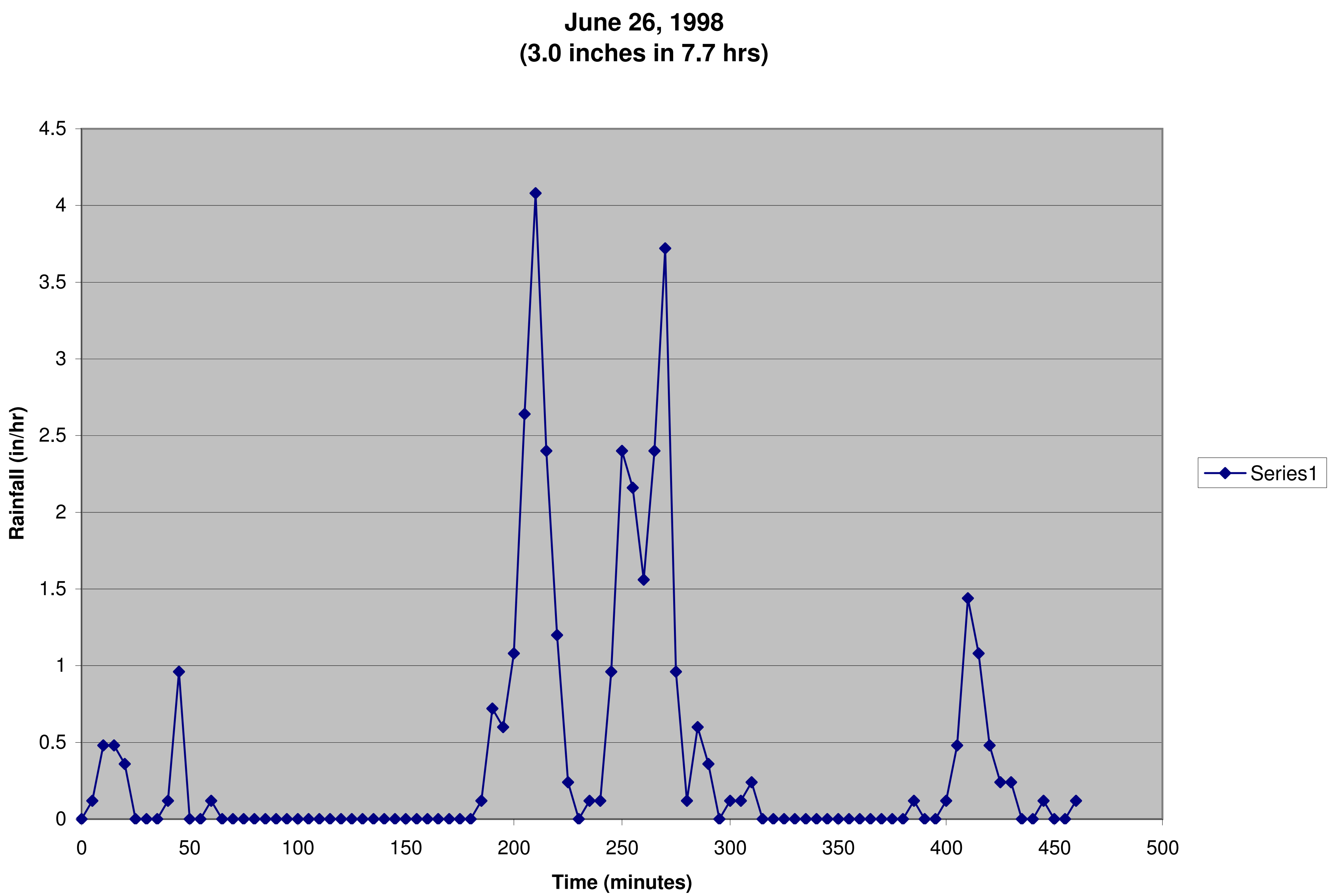

June 25, 1998
(0.45 inches in 55 minutes)
Rainfall (in/hr)
Time (minutes)
0
0.5
1
1.5
2
2.5
3
0
10
20
30
40
50
60

Figure 3-5



June 26, 1998
(3.0 inches in 7.7 hrs)
Rainfall (in/hr)
Time (minutes)
Series1
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
0
50
100
150
200
250
300
350
400
450
500

Figure 3-6

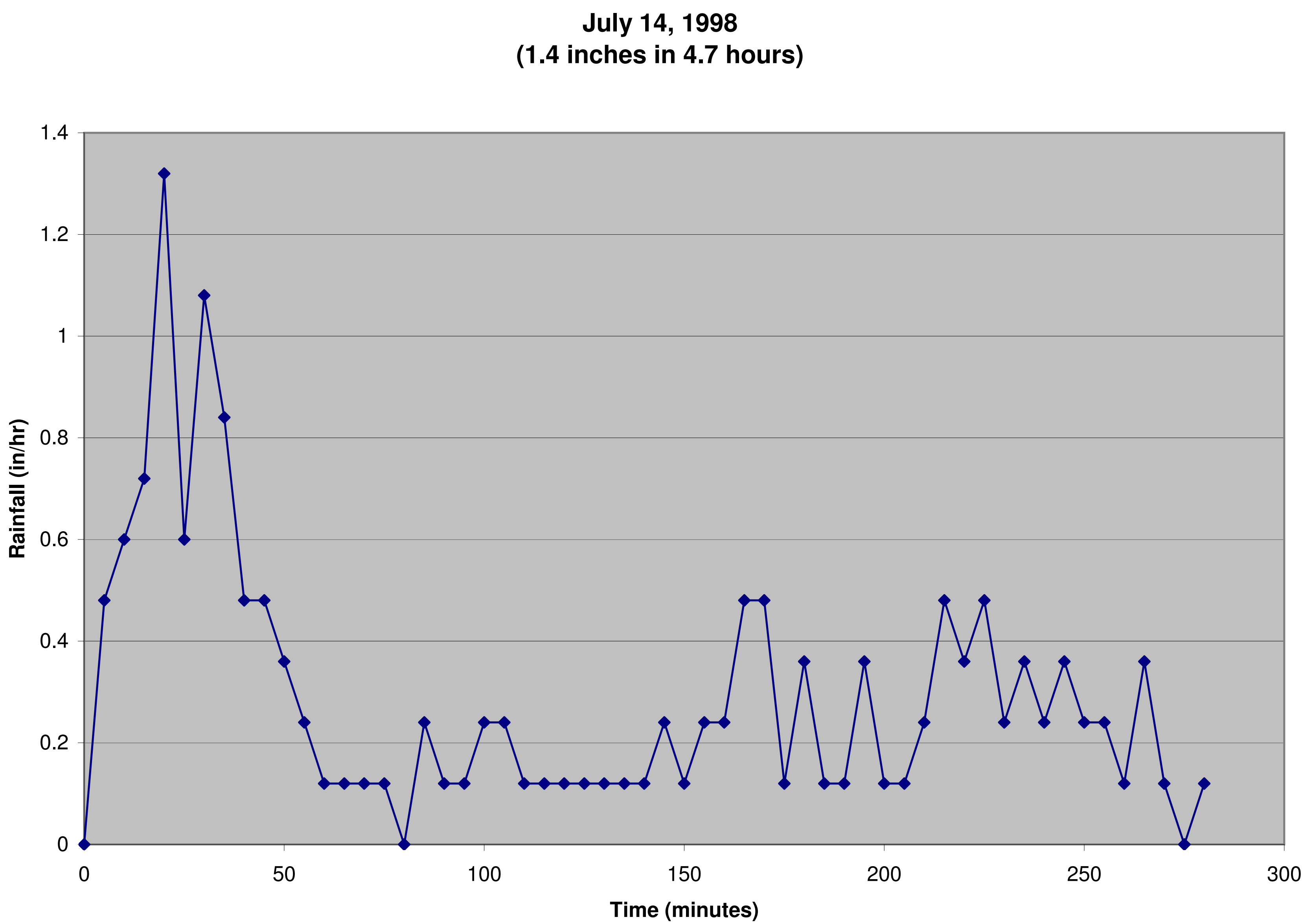
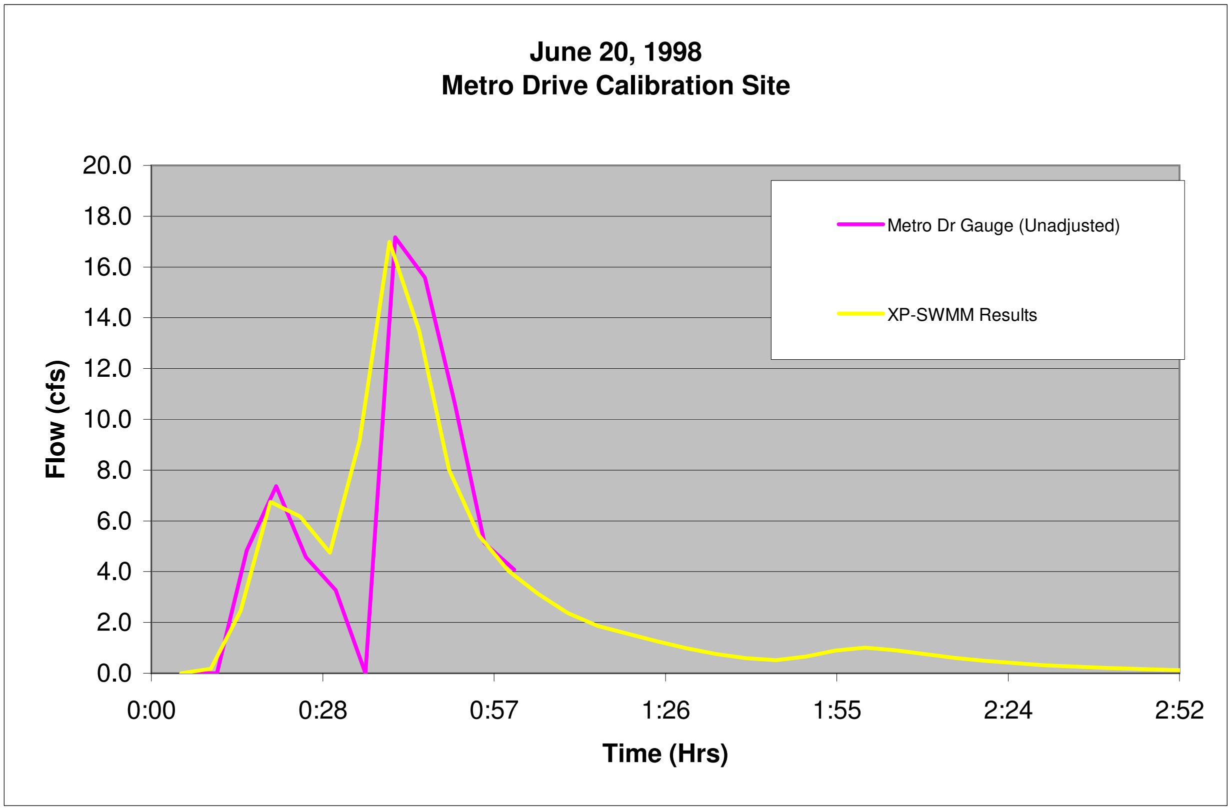

July 14, 1998
(1.4 inches in 4.7 hours)
Rainfall (in/hr)
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0
50
100
150
200
250
300
Time (minutes)

Figure 3-7


June 20, 1998
Metro Drive Calibration Site
Metro Dr Gauge (Unadjusted)
XP-SWMM Results
Flow (cfs)
20.0
18.0
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
0:00
0:28
0:57
1:26
1:55
2:24
2:52
Time (Hrs)

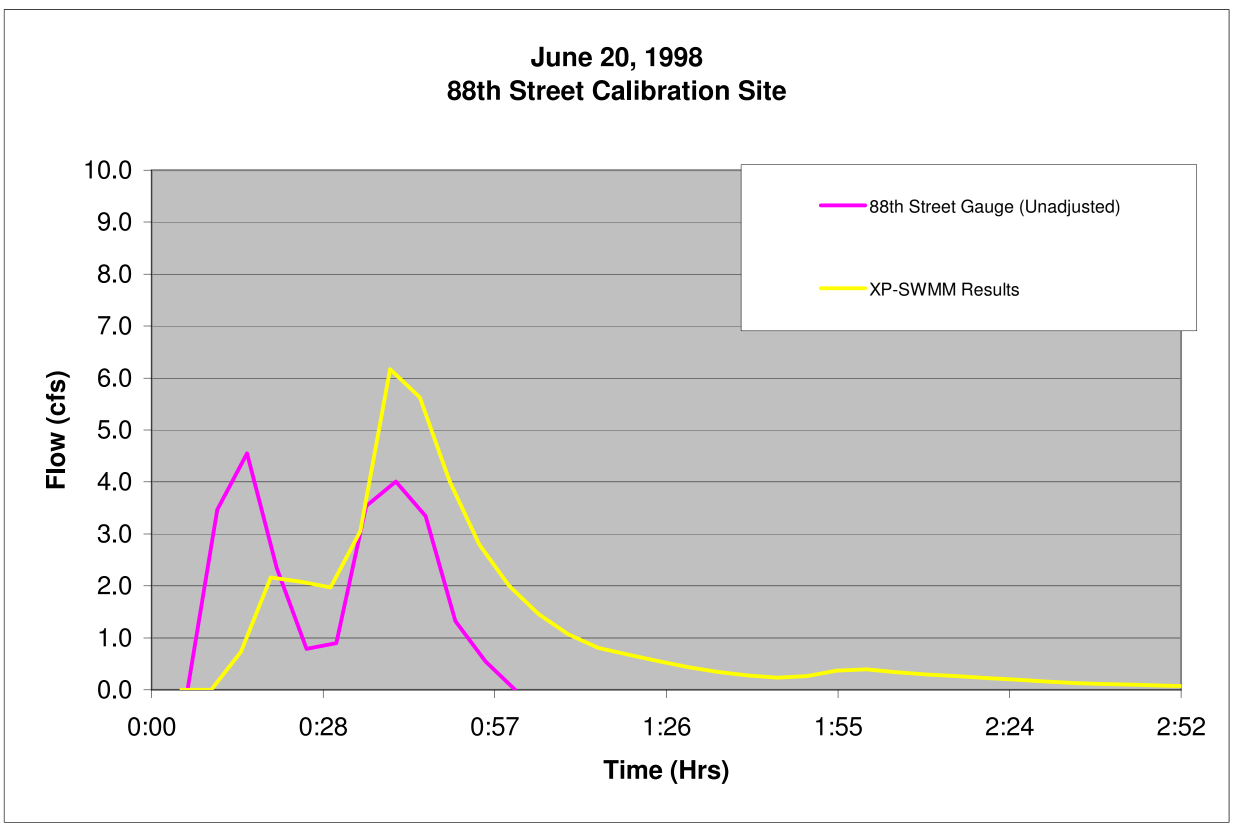
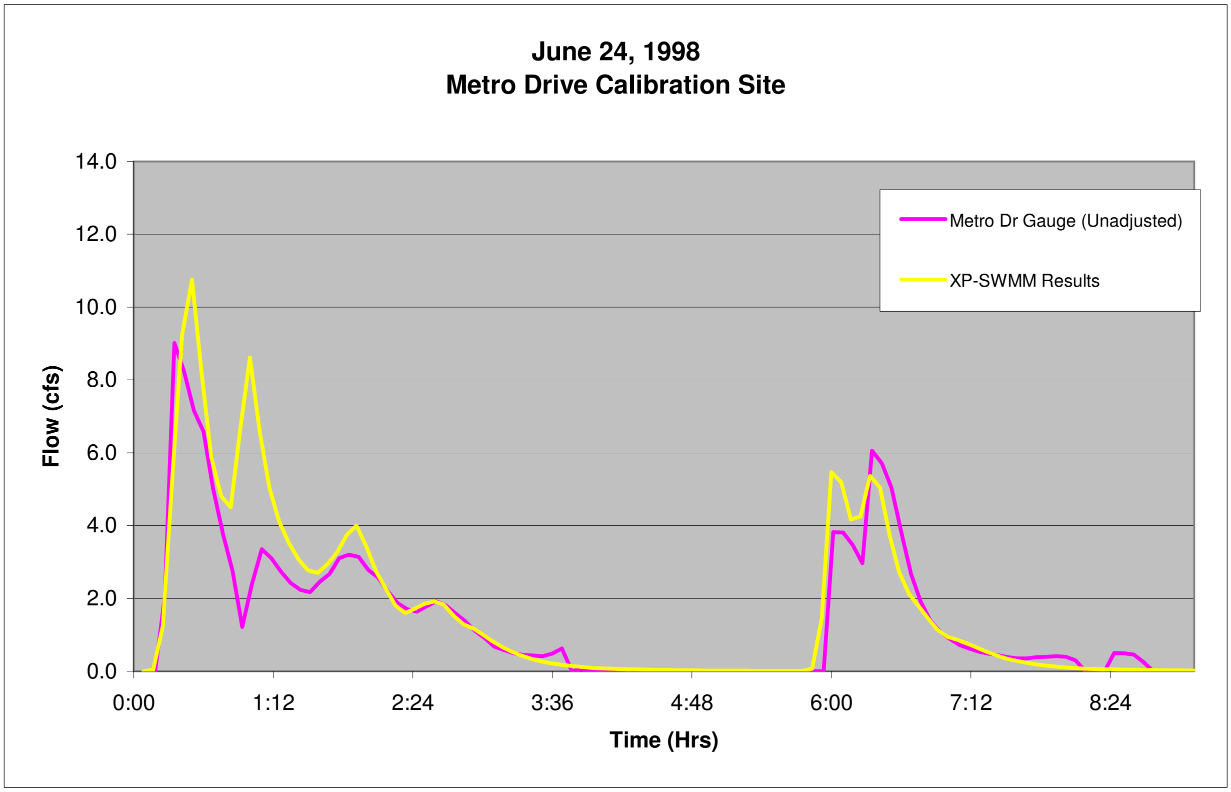

June 20, 1998
88th Street Calibration Site
88th Street Gauge (Unadjusted)
XP-SWMM Results
Flow (cfs)
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0:00
0:28
0:57
1:26
1:55
2:24
2:52
Time (Hrs)

Figure 3-8


June 24, 1998
Metro Drive Calibration Site
Metro Dr Gauge (Unadjusted)
XP-SWMM Results
Flow (cfs)
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
0:00
1:12
2:24
3:36
4:48
6:00
7:12
8:24
Time (Hrs)

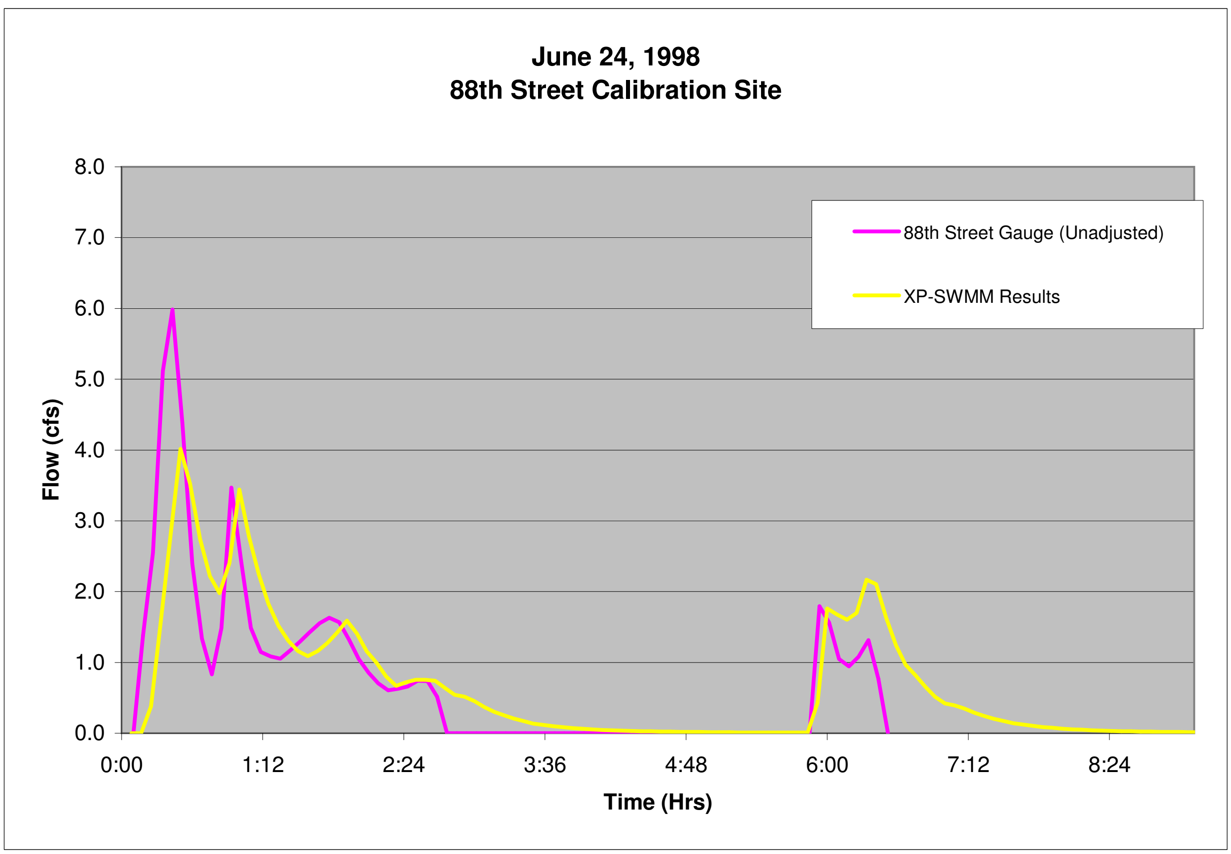
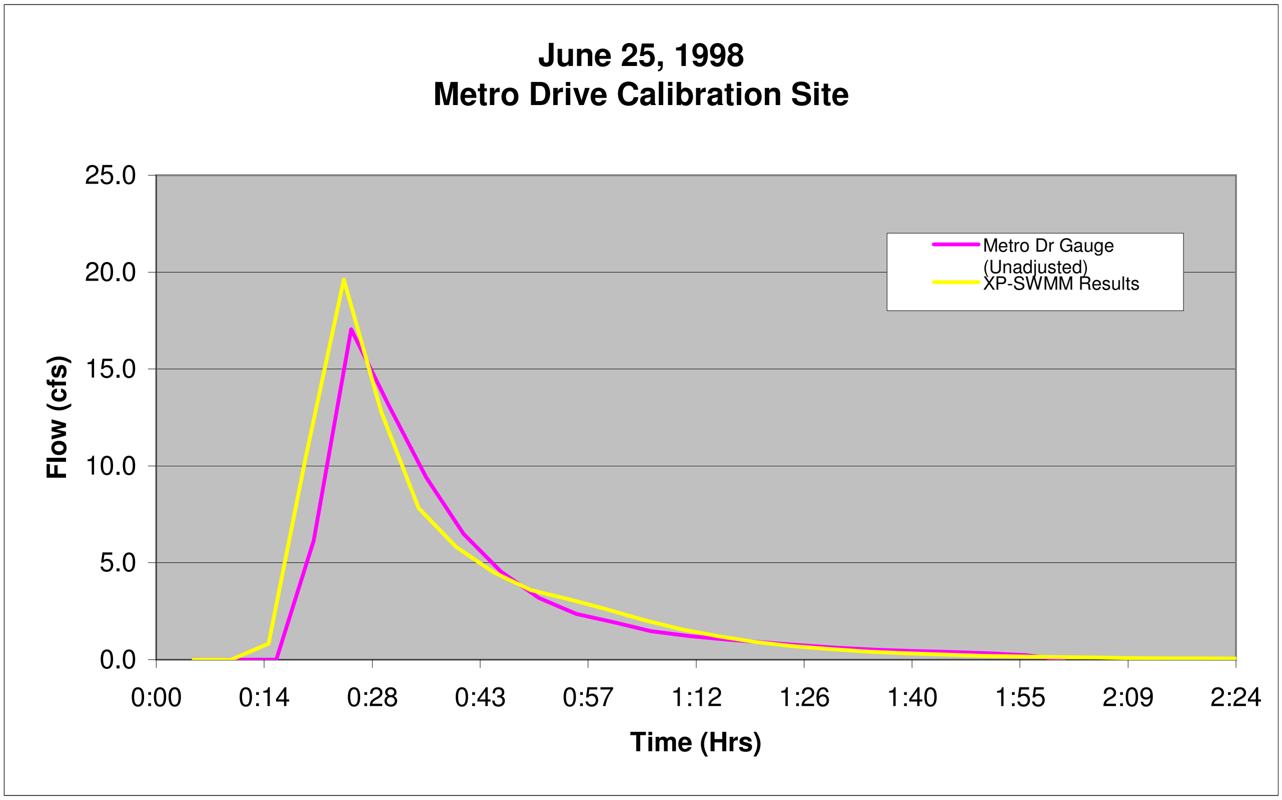

June 24, 1998
88th Street Calibration Site
88th Street Gauge (Unadjusted)
XP-SWMM Results
Flow (cfs)
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0:00
1:12
2:24
3:36
4:48
6:00
7:12
8:24
Time (Hrs)

Figure 3-9


June 25, 1998
Metro Drive Calibration Site
Flow (cfs)
25.0
20.0
15.0
10.0
5.0
0.0
0:00
0:14
0:28
0:43
0:57
1:12
1:26
1:40
1:55
2:09
2:24
Time (Hrs)
Metro Dr Gauge (Unadjusted)
XP-SWMM Results

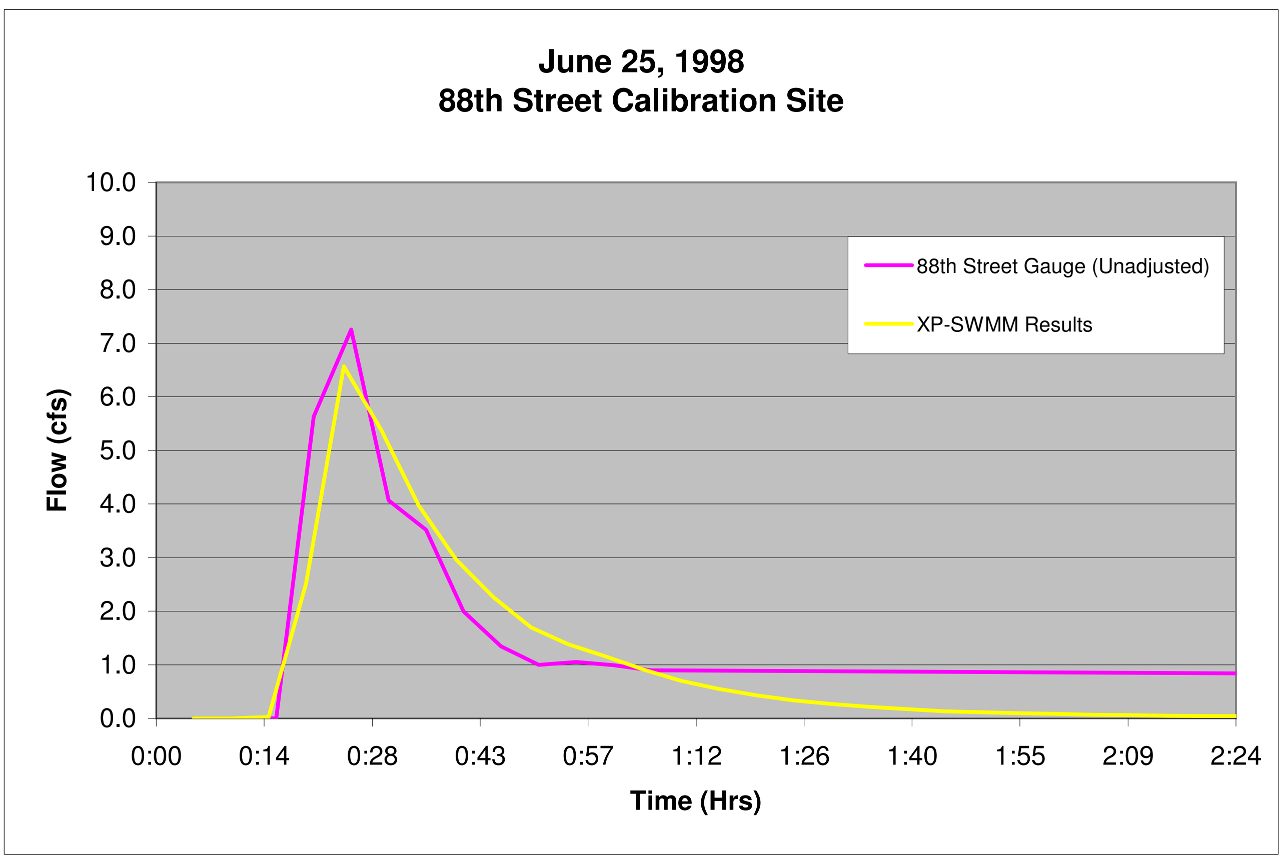
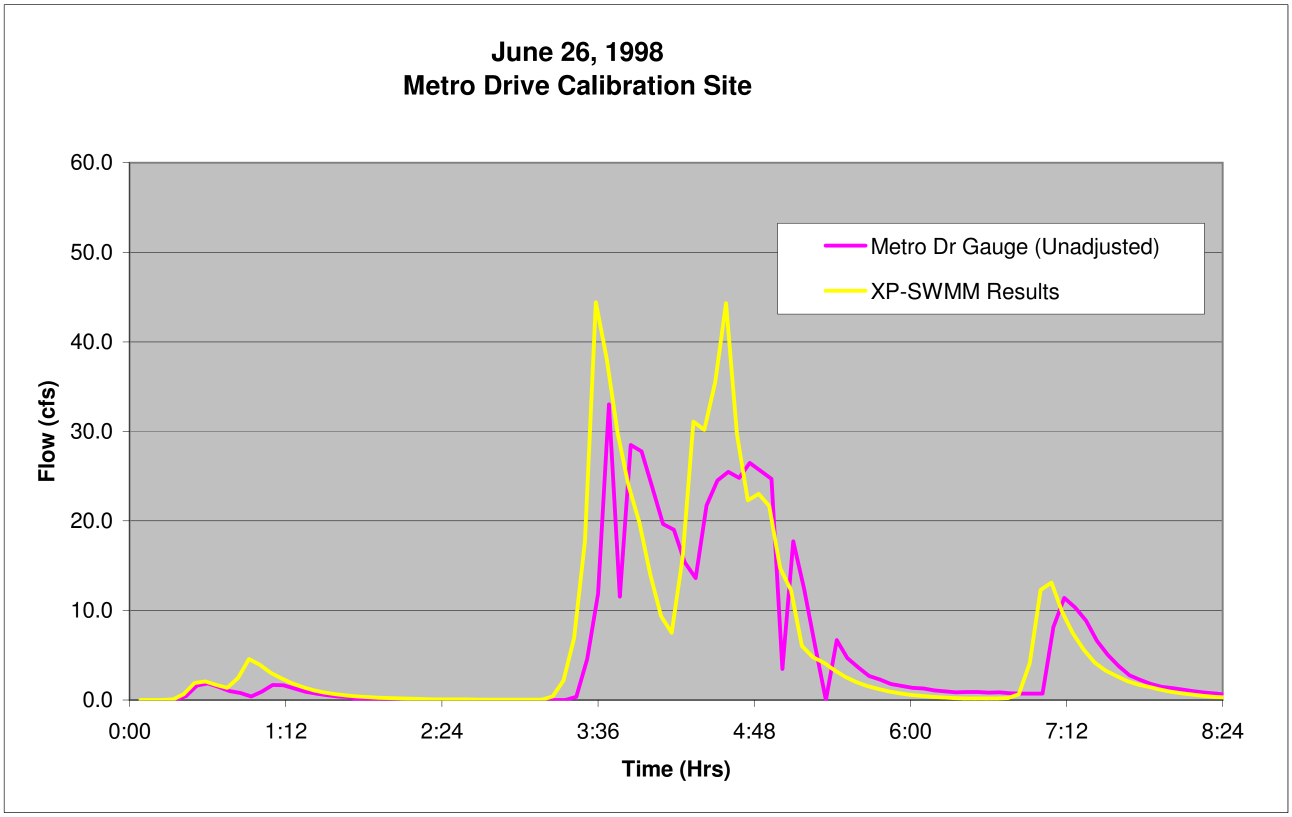

June 25, 1998
88th Street Calibration Site
Flow (cfs)
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0:00
0:14
0:28
0:43
0:57
1:12
1:26
1:40
1:55
2:09
2:24
Time (Hrs)
88th Street Gauge (Unadjusted)
XP-SWMM Results

Figure 3-10


June 26, 1998
Metro Drive Calibration Site
60.0
50.0
40.0
30.0
20.0
10.0
0.0
Flow (cfs)
0:00
1:12
2:24
3:36
4:48
6:00
7:12
8:24
Time (Hrs)
Metro Dr Gauge (Unadjusted)
XP-SWMM Results

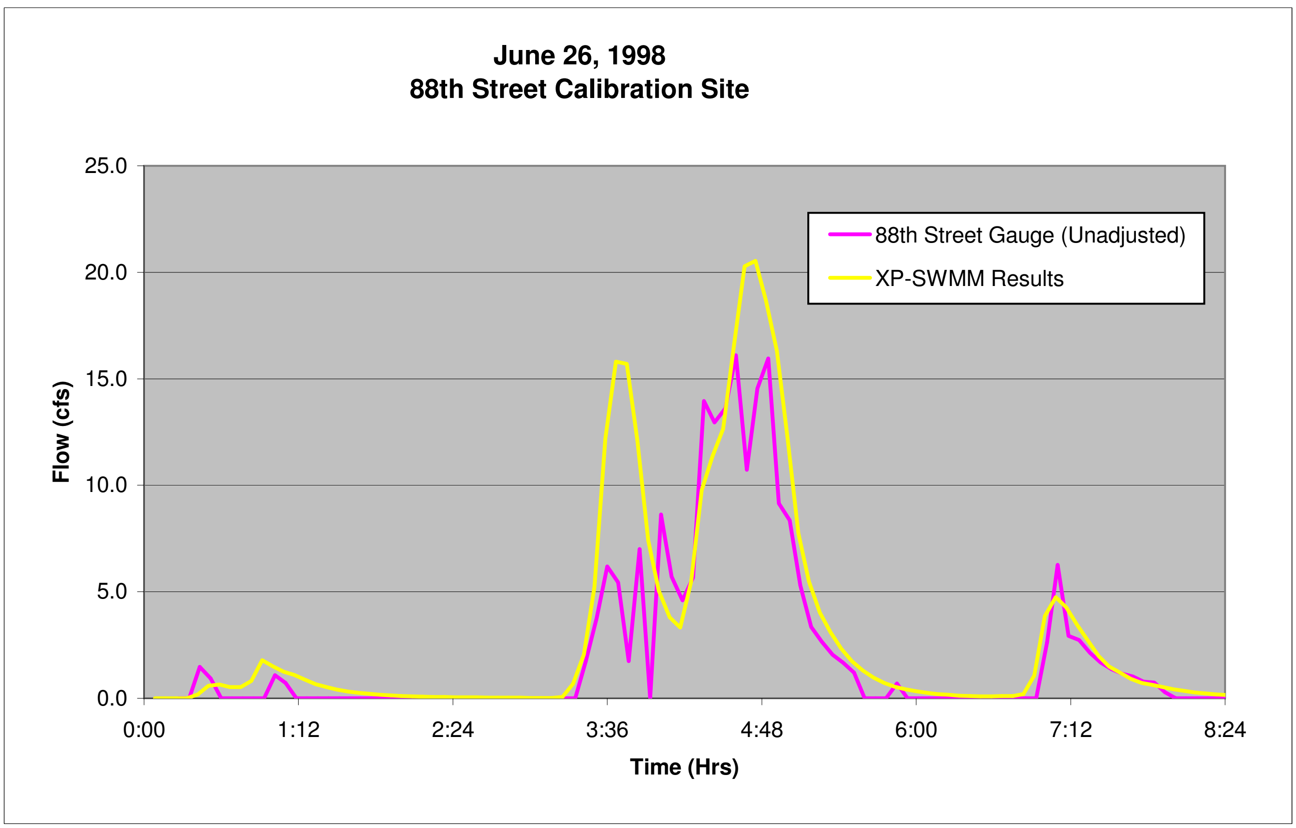
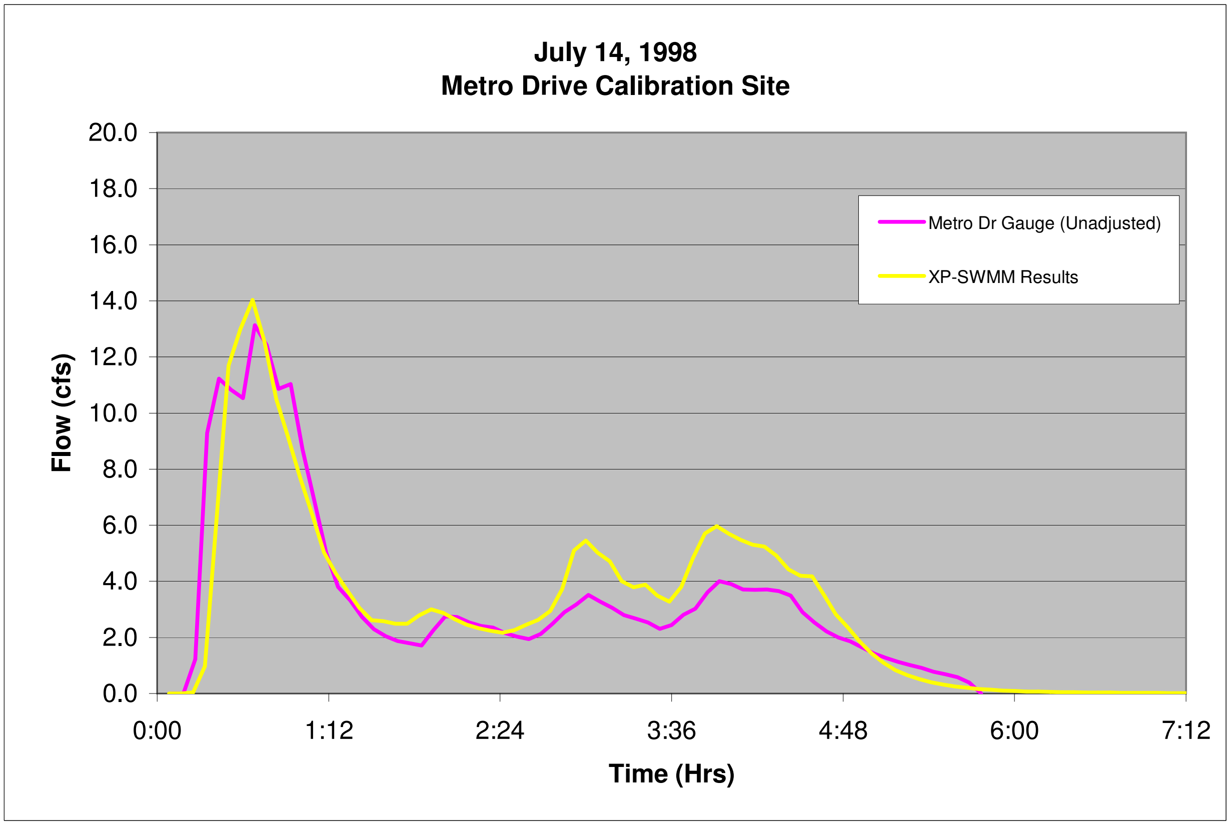

June 26, 1998
88th Street Calibration Site
25.0
20.0
15.0
10.0
5.0
0.0
Flow (cfs)
0:00
1:12
2:24
3:36
4:48
6:00
7:12
8:24
Time (Hrs)
88th Street Gauge (Unadjusted)
XP-SWMM Results

Figure 3-11


July 14, 1998
Metro Drive Calibration Site
Metro Dr Gauge (Unadjusted)
XP-SWMM Results
Flow (cfs)
20.0
18.0
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
0:00
1:12
2:24
3:36
4:48
6:00
7:12
Time (Hrs)

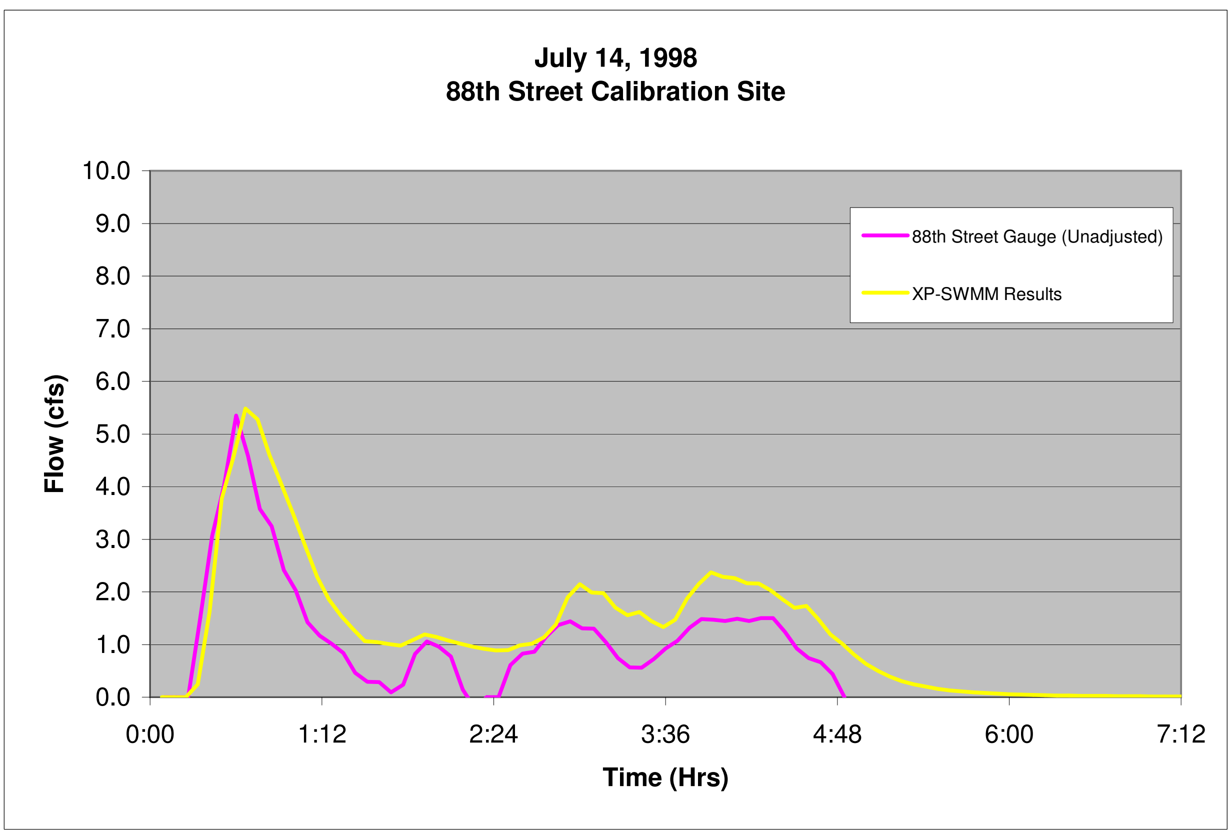

July 14, 1998
88th Street Calibration Site
88th Street Gauge (Unadjusted)
XP-SWMM Results
Flow (cfs)
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0:00
1:12
2:24
3:36
4:48
6:00
7:12
Time (Hrs)

Figure 3-12

4.0 Methodology for Water Quality Modeling

In 2002, the ASDD and the adjacent drainage areas to the west (Smith Pond and Wright's Lake) were modeled as part of the *Bloomington Airport South District Storm Water Treatment Feasibility Study* (SRF, 2002). The drainage districts were modeled using the P8 Urban Catchment Model (Program for Predicting Polluting Particle Passage thru Pits, Puddles, and Ponds), which is a model used for predicting the generation and transport of stormwater runoff and pollutants in urban watersheds. The model tracks the movement of particulate matter (fine sand, dust, soil particles, etc.) as it is carried along by stormwater runoff traveling over land and pavement. Particle deposition in ponds along the way is also tracked, so that the model can estimate the amount of pollutants—carried by the particles—that eventually reach a water body.

The previous modeling effort evaluated the effectiveness of the stormwater treatment systems in place during the year 2000 (existing conditions), as well as 2020 (future) development conditions, assuming construction of NURP ponds in the areas of proposed redevelopment. Additionally, the feasibility of several regional water quality treatment scenarios was evaluated to assess future treatment opportunities, in an effort to maintain and improve the quality of water being discharged to Long Meadow Lake (part of the U.S. Fish and Wildlife National Wildlife Refuge).

The previously developed P8 model was updated to reflect the revised drainage areas and recent development and redevelopment that have occurred within the ASDD. Additional on-site stormwater treatment basins were included in the P8 model, with model input parameters based on the design/as-built drawings and/or two-foot topographic data. The model was converted to the most recent P8 version (Version 3.4), to simulate the quantity and quality of the surface runoff in the ASDD and evaluate the average annual removal efficiency of the existing water quality treatment systems. The runoff quality and system treatment efficiencies were also evaluated for the 2-year, 24-hour SCS Type II storm event.

The previously developed P8 model included the Smith Pond Drainage District located west of TH 77, which is tributary to the ASDD. There were no changes made to the model input parameters or water quality treatment systems in these drainage districts as part of the water quality model update.

4.1 Determination of Watershed Characteristics

Examination of the watershed characteristics for the ASDD involved evaluation of soil types, land use, and the impervious fraction of the land in the drainage district. For the drainage areas to the west of TH 77 (i.e., those outside of the ASDD), the watershed characteristics from the original existing conditions model (SRF, 2002) were used.

4.1.1 P8 Drainage Basins

The sub-drainage basin delineations completed for the XP-SWMM modeling were used as a base for the P8 water quality modeling. Because the P8 water quality modeling does not require the same level of detail as the XP-SWMM analyses, many of the XP-SWMM sub-drainage basins were merged into larger P8 drainage basins. The P8 drainage basins are shown in Figure 4-1, along with the locations of the Best Management Practices (BMPs) and other water bodies that were included in the updated P8 model. The arrows in Figure 4-1 show the general drainage patterns within the ASDD.

4.1.2 Land Use – Existing Conditions

The existing (2007) land use used for the modeling update was originally developed for the City of Bloomington *Nondegradation Loading Assessment Report* (Barr Engineering Co., 2007). This land use layer was created based on 2007 parcel-based land use provided by the City of Bloomington. This land use layer was verified against the 2006 aerial photo, and for areas in question, the land use was field-verified and adjusted as necessary. The 2007 land use data includes the following categories: agriculture, commercial, developed park, forest, grassland, high-density residential, highway, industrial, institutional, low-density residential, medium-density residential, and water/wetland. For P8, the forest and grassland areas were recategorized into a natural land cover classification.

This land use information was used to estimate the total amount of impervious area within each P8 drainage basin as well as the amount of directly-connected impervious area. The directly-connected impervious fraction consists of the impervious surfaces that are "connected" directly to stormwater conveyance systems, meaning that flows do not cross over pervious areas. The total impervious and directly-connected impervious percentages were based on values estimated for the *Nine Mile Creek/Bloomington Use Attainability Analysis UAA* (Barr Engineering Co., 2001), with the exception of the agricultural land use category, which was based on the impervious percentage calculated for the *Nondegradation Loading Assessment Report* (Barr Engineering Co., 2007).

4.1.3 Curve Numbers

The pervious curve number (a measure of how easily water can percolate into the soil) was also determined for each P8 drainage basin within the ASDD. Data from the Hennepin County Soils Survey (NRCS, 2004) was used to determine the hydrologic soil group (HSG), which serves as an indicator of a soil's infiltration capacity. However, since Bloomington is an older, fully-developed city, soils throughout much of the area are typically classified as "undefined" or "urban" soils and have not been assigned a HSG.

To estimate the HSG for areas not classified in the soil survey, the City provided soil boring log information obtained during development and redevelopment projects completed within ASDD. Each of the soil boring logs was reviewed, and in general, soil types throughout the drainage district were classified as silty and sandy soils. Therefore, SCS Type B (moderate infiltration rate) soils were assumed for those soils not classified as part of the county soil survey. This is consistent with HSG assigned in areas that were classified as part of the county soil survey (SCS Types A and B).

A pervious curve number was selected for each P8 drainage basin based upon soil types, land use, and hydrologic conditions (e.g., if soils are Type B and pervious areas are comprised of grassed areas with 50% to 75% cover, then a Curve Number of 69 would be selected). An overall composite pervious curve number was determined by weighting the areas for the given soil groups within each drainage basin. This composite pervious curve number was then weighted with indirect (i.e., unconnected) impervious areas in each P8 drainage basin as follows:

$$WCN = \frac{[(Indirect\ Impervious\ Area] * (98)] + [(Pervious\ Area) * (Pervious\ Curve\ Number)]}{(Indirect\ Impervious\ Area + Pervious\ Area)}$$

4.2 Drainage Patterns

The stormwater management system within the ASDD is a complex network of storm sewer, natural channels, and ponding basins. To assist in identifying the drainage patterns within the district, the P8 drainage basins have been summarized into drainage 'regions' based on outfall and color coded accordingly (see Figure 4-1).

Much of the northern portion of the ASDD is served by a complex storm sewer network, with several locations in which flows are split in multiple directions, depending on the amount of flow through the system (i.e., low flows are conveyed to one trunk system and a portion of the high flows are conveyed to a different trunk system). The locations of the ‘flow splitters’ that were modeled in P8 are shown in Figure 4-1. The P8 drainage basins that are tributary to these flow splitters are identified in Figure 4-1 in a hatched pattern. The flow splitters were modeled using general devices. Stage-discharge relationships for these devices were developed based on the flows predicted in XP-SWMM for the 2-year frequency, 24-hour event. The areas of the general devices were selected such that the percentage of flow volume being discharged in each direction closely matched that simulated in XP-SWMM.

4.3 Pollutant Removal Device Information

The P8 water quality model can predict pollutant removal efficiency for a variety of treatment practices such as detention ponds and infiltration basins. The model can also be used to simulate pollutant removal from alternative BMPs such as underground treatment devices. The modeled treatment practices are referred to in the P8 model as pollutant removal ‘devices’.

4.3.1 Ponds

Water quality ponds (also called detention ponds, stormwater ponds) are the most common BMP within the ASDD. The “dead” storage volume (storage below the normal water level) is an important factor in the pollutant removal efficiency of water quality ponds. As such, it is important to represent this volume as accurately as possible. Digital two-foot topographic data was used in conjunction with as-built development plans to develop the storage volumes for the ponds being modeled in P8. No bathymetric data was readily available for Little Bass Pond (APS-156) and Big Bass Pond (APS-69), located within the Minnesota Valley National Wildlife Refuge. Therefore, field surveys were performed for these ponds to determine the dead storage volume and outlet configuration.

Information on the pond outlet configurations were obtained from the City’s GIS storm sewer database and/or as-built development plans. Because P8 has a limited capacity to model complex outlet structures, outlet rating curves were developed based on the XP-SWMM model results for select outlet structures. Pond storage and outlet information for waterbodies to the west of TH 77 (Smith Pond and Wright’s Lake) were used from the original existing conditions P8 model developed by SRF in 2002.

4.3.2 Infiltration Basins

Several infiltration basins within the ASDD were modeled in P8, including the bio-infiltration basins at the IKEA site and the large infiltration basin located in the northeast quadrant of the intersection of 24th Avenue and Old Shakopee Road (APS-42). The infiltration basins were modeled as detention pond 'devices', with an assumed infiltration rate of 0.5 inches/hour, based on the predominance of Type A soils within the ASDD.

4.3.3 Underground Stormwater Treatment Structures

Underground stormwater treatment structures are proprietary treatment devices often used when site constraints prevent the construction of conventional BMPs such as ponds or infiltration basins. City staff has indicated that numerous underground treatment structures have been installed on private sites throughout the ASDD. However, a complete inventory of the number and locations of these devices has not been developed and the private underground stormwater treatment structures were generally not included in the P8 model.

An underground treatment structure was installed by the City of Bloomington in 2005 to treat a portion of the flow through the 80th Street trunk storm sewer system. This system has been included in the updated P8 model. An underground treatment system installed at the IKEA site was also included in the updated P8 model.

4.4 P8 Model Parameters

The P8 model requires a variety of inputs beyond the watershed characteristics and pollutant removal device (ponds, etc.) characteristics. P8 also requires hourly precipitation and temperature data for either a single storm event or for a long-term climatic period. Additionally, pollutant characteristic information is needed. This pollutant and particle information is typically based on national average information unless more local data is available. The parameters selected for the P8 model are discussed in the following paragraphs. P8 parameters not discussed in the following paragraphs were left at the default setting. As mentioned previously, Version 3.4 of the P8 Model was used for the updated modeling.

4.4.1 Precipitation and Temperature Data

P8 reads hourly precipitation and daily average temperature data from a data file for a continuous simulation of watershed hydrology and the buildup/washoff of water quality constituents. Hourly rainfall data and daily temperature data from the National Weather Service Station at the Minneapolis/St. Paul International Airport was used for the water quality modeling.

- **MSP4907.PCP.** The precipitation file MSP4907.PCP is comprised of hourly precipitation measured at the Minneapolis–St. Paul International Airport were used for the period between 1949 and the end of October 2007.

- **MSP4907.tmp**. The temperature file was comprised of daily average temperature data from the Minneapolis–St. Paul International Airport during the period from 1949 through 2007.

4.4.2 Time Step, Rainfall Breakpoint, Snowmelt, & Runoff Parameters

- **Time Steps Per Hour (Integer)—** 6. Selection was based upon the number of time steps required to eliminate continuity errors greater than two percent.

- **Growing Season AMC—II = 0 and AMC—III = 100**. These parameters were originally selected during the development and calibration of the P8 model for the *NMCWD/Bloomington Use Attainability Analysis* (2001). Selection of these factors was based upon the observation that the model accurately predicted runoff water volumes from monitored watersheds when the Antecedent Moisture Condition II was selected (i.e., curve numbers selected by the model are based upon antecedent moisture conditions). Modeled water volumes from pervious areas were less than observed volumes when Antecedent Moisture Condition I was selected, and modeled water volumes exceeded observed volumes when Antecedent Moisture Condition III was selected. The selected parameters tell the model to only use Antecedent Moisture Condition I when less than 0 inches of rainfall occur during the five days prior to a rainfall event and to only use Antecedent Moisture Condition III if more than 100 inches of rainfall occur within five days prior to a rainfall event.

4.4.3 Particle File Selection

- **NURP50.PAR.** The NURP50 particle file was used for the updated P8 model, which is consistent with the previous P8 model developed for the ASDD. The NURP50 particle file was developed as part of the Nationwide Urban Runoff Program (NURP), a research program conducted by the U.S. Environmental Protection Agency, and provides default parameters for several water quality components, based upon calibration to median, event-mean concentrations reported by NURP (Athayede et al., 1983).

4.4.4 Devices Parameter Selection

- **Detention Pond— Permanent Pool— Area and Volume—** The surface area and dead storage volume of each detention pond were determined. Where available, Barr used outlet stage-discharge relationships or other rating information and pond volume information developed for the hydrologic/hydraulic modeling or from field surveys. If limited information was supplied, Barr assumed an average depth and estimated the surface area (based on digital two-foot topography) to determine the pond permanent pool volume.

- **Detention Pond— Flood Pool— Area and Volume—** The surface area and storage volume under flood conditions (i.e., the storage volume between the normal level and flood elevation) were determined. The areas and volumes were estimated based on information developed for the hydrologic/hydraulic modeling (from digital two-foot topography, as-built development plans, or field survey).

- **Infiltration Rate (in/hr)—** Infiltration rates were only entered for infiltration basins (not for detention ponds). An infiltration rate of 0.5 inches/hour was used based on area soil conditions.

- **Detention Pond—** Orifice Diameter and Weir Length— The orifice diameter or weir length was determined from field surveys or as-built development plans for each detention pond.

- **Detention Pond or Generalized Device—** Particle Removal Scale Factor— Particle Removal Scale Factor— 0.3 for all ponds less than 2 feet deep and 1.0 for ponds 3 feet deep or greater. For ponds with normal water depths between 2 and 3 feet, a particle removal factor of 0.6 was selected. For devices acting as flow splitters, the particle removal factor was 0.0.

- **Pipe/Manhole— Time of Concentration—** The time of concentrations for each pipe/manhole device were originally maintained from the previously developed P8 model. These values were typically less than 0.25 to 0.5 hours. However, these values resulted in computational errors in the new version of the P8 model. To avoid computational errors, the times of concentration for all pipes were revised to 0.5 hours.

4.4.5 Watersheds Parameter Selection

- **Pervious Curve Number**— An overall composite pervious curve number was determined by weighting the areas for the given soil groups within each P8 drainage basin. This composite pervious curve number was then weighted based on the indirect (i.e., unconnected) impervious areas in each drainage basin, based on the assigned land use type, as outlined in Section 5.1.

- **Indirectly Connected Impervious Fraction** – The parameter is a new addition to P8 Version 3.4. This value was set to 0 for all P8 drainage basins to be consistent with the previous P8 modeling completed for the Smith Pond Drainage District and other areas of the city. The areas of indirectly connected imperviousness were accounted for by weighting the pervious curve number as described above.

- **Connected Impervious Fraction—**The direct or connected impervious fraction for each subwatershed was determined. Connectivity estimation of the various impervious surface types was accomplished by associating each surface type with a land use category. See Section 4.1.2 for additional information.

- **Swept/Not Swept—**An "Unswept" assumption was made for the entire impervious watershed area. A Sweeping Frequency of 0 was selected.

- **Depression Storage—** 0.06 inches for all P8 drainage basins, based on the *NMCWD/Bloomington Use Attainability Analysis* (2001).

- **Impervious Runoff Coefficient**— 0.95 for all P8 drainage basins, based on the *NMCWD/Bloomington Use Attainability Analysis* (2001).

4.4.6 Passes through the Storm File

- Passes through Storm File— The number of passes through the storm file was determined after the model had been developed and a preliminary run completed. The selection of the number of passes through the storm file was based upon the number required to achieve model stability. Multiple passes through the storm file were required because the model assumes that dead storage waters contain no pollutants. Consequently, the first pass through the storm file results in lower pollutant loading than occurs with subsequent passes. Stability occurs when subsequent passes do not result in a change in pollutant concentration in the pond waters. Five (5) passes through the storm file resulted in model stability for this model.

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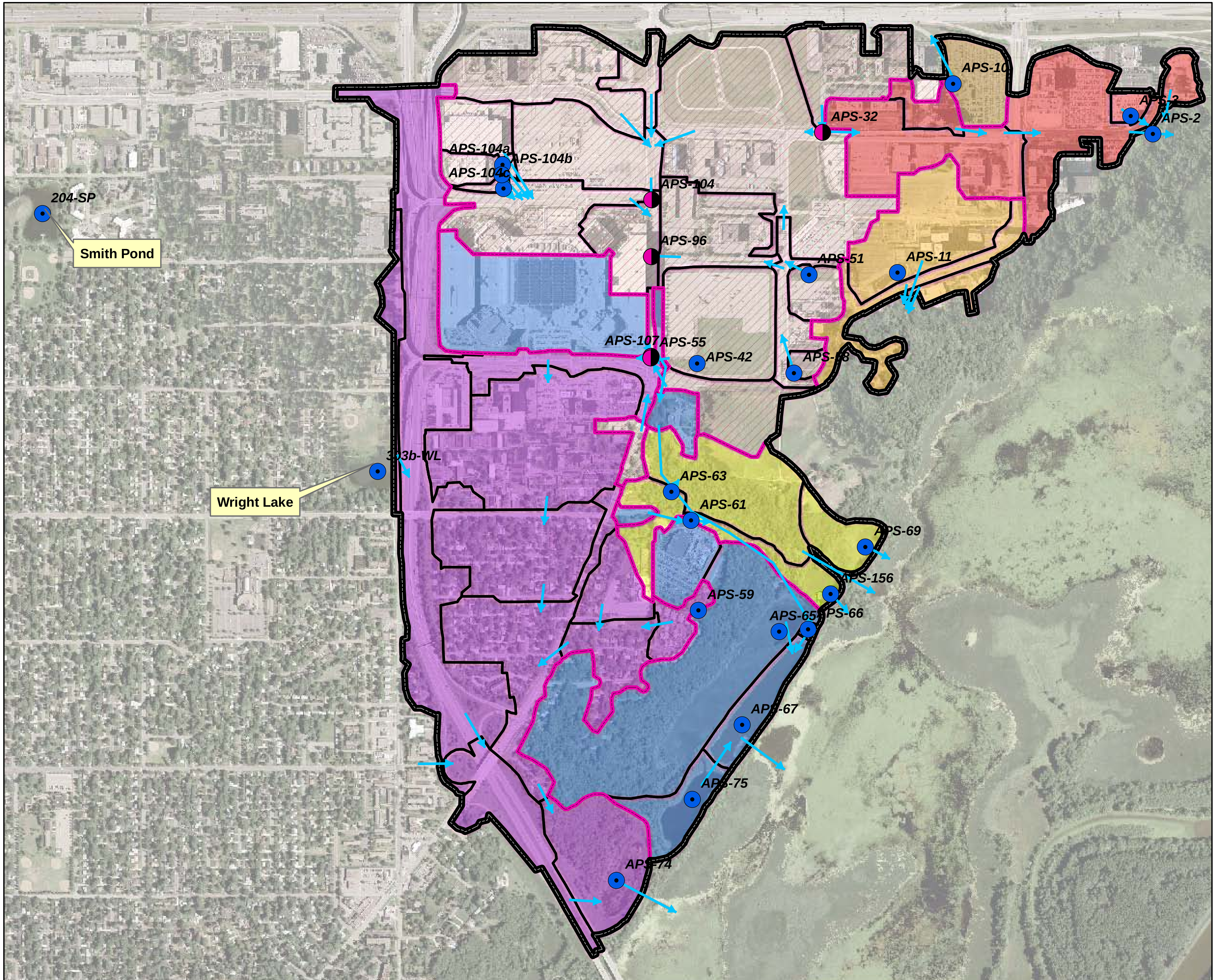

Water Quality Treatment Device (Ponds, Infiltration Basins, Wetlands, & Underground Treatment Structures)

Flow Splitter

Flow Direction

Airport South Drainage District Boundary

P8 Drainage Basins

P8 Drainage Region-1

Split Flows

Pond C Outfall

Hogback Outfall

I-494 Outfall

USFWS Outfalls

Ceridian Outfall

80th Street Outfall

1 - P8 Drainage Regions are organized generally by outfall.

Feet

1,000 0 1,000

Figure 4-1
P8 Drainage Patterns

Airport South Drainage District
City of Bloomington, Minnesota

5.0 Results and Conclusions

5.1 Hydrologic/Hydraulic Modeling Results and Discussion

Table 5-1 lists the modeled results for 2-, 10-, and 100-year frequency, 24-hour storm events for the ASDD based on existing (2008) land use conditions. The column headings are defined as follows:

Sub-drainage Basin Data

- **Subwatershed/Node ID** – Sub-drainage basin and XP-SWMM node identification label.
- **Manhole ID** – The manhole ID used in the City's GIS database that corresponds to the Subwatershed/XP-SWMM Node ID
- **Sub-drainage Basin Area** – The drainage area (in acres) in a given sub-drainage basin.
- **Percent Impervious**– The percentage of the sub-drainage basin area that is considered to be directly connected impervious area, based on an areal weighting of the land use within the given sub-drainage basin.
- **Sub-drainage Basin Peak Runoff Rates –** These three columns list the peak runoff rates in cubic feet per second (cfs) for the respective sub-drainage basins for the SCS Type II 2-, 10-, and 100-year frequency, 24-hour storm events. Note that the reported values reflect the peak runoff rate from the direct drainage area only, and do not include flows from upstream sub-drainage basins.

Ponding Basin /Inundation Area Data

- **Type-** Description of the type of ponding basin or area (i.e., pond, infiltration basin, wetland or inundation area). Note that for the areas identified as 100-year inundation areas but not ponding basins or wetlands, only the 100-year high water level has been reported in Table 5-1.
- **Peak Outflow–** These three columns list the peak discharge rate (cfs) from selected ponding basins for the SCS Type II 2-, 10-, and 100-year frequency, 24-hour storm events. The peak outflow rates reflect the combined discharge from the basin through the outlet structure and any overflow device (e.g., an overflow grate or embankment). In several instances, such as ponds APS-42 and APS-75, the backflow into the detention area exceeds the outflow magnitude (i.e., there is a greater inflow rate than discharge outflow rate).
- **NWL –** Normal water level in the basin (in feet, Mean Sea Level) based on the control elevation of the outlet pipe or structure.

- **100-Year HWL –** The high water level (HWL) in the given basin or inundation area as a result of runoff from the SCS Type II 100-year frequency, 24-hour storm event.
- **100-Year Live Storage –** The maximum volume (in acre-feet) of water that is stored above the normal water level in the ponding basin for the 100-year frequency, 24-hour event. The volumes listed are based on the average end area calculation method.

Figure 3-1 illustrates the XP-SWMM hydrologic/hydraulic modeling results for the ASDD. The information depicted on the figure assumes existing (2008) development conditions, as described in Section 3.2.1. Figure 3-1 illustrates the ASDD boundary, sub-drainage basin boundaries and labels, modeled storm sewer networks, streets, surcharge conditions for the XP-SWMM nodes, and inundation areas.

5.1.1 100-Year Inundation Areas

As mentioned above, Figure 3-1 depicts the '100-year Inundation Areas' throughout the ASDD. The inundation areas, shown in blue hatching, represent the approximate areas of inundation for the low areas and ponding basins based on the City's two-foot topographic data and the predicted water surface elevations for the 100-year frequency, 24-hour SCS Type II storm event. The flooded areas shown are generally ponding basins, but also include low areas such as parking lots and roads. The predicted 100-year high water elevations for the inundation areas are summarized in Table 5-1.

5.1.2 Surcharged Conditions

An XP-SWMM node (generally representing a manhole/catch basin or ponding basin) was considered surcharged if the hydraulic grade line at that node breached the ground surface. Surcharging is likely the result of limited downstream capacity or tailwater impacts. The detention areas were assumed to be surcharged if the modeled hydraulic grade line exceeds the basin's control elevation (NWL). Figure 3-1 shows that several XP-SWMM nodes are predicted to experience surcharged conditions during the 2-year, 10-year and 100-year frequency, 24-hour storm events (shown in red). Many other XP-SWMM nodes are predicted to experience surcharged conditions during the 10-year and 100-year frequency, 24-hour storm events (shown in orange). These manhole and catch basin locations (shown in red and orange) are much more likely to experience inundation during the smaller, more frequent storm events of various durations and should be further evaluated for potential corrective measures.

5.1.3 Mall of America

As discussed in Section 3.3.1, the Mall of America storm sewer system was modeled in detail to assess recent flooding that has occurred at several locations within the Mall of America property, most notable the Hiawatha LRT Station in the southeast corner of the site. Model results indicate that surcharged conditions can be expected to occur during the 10- and 100-year frequency, 24-hour rainfall events for several areas within the Mall of America site, including sub-drainage basins APS-92, APS-172, and APS-175 (see orange nodes in Figure 3-1). Surcharged conditions can be expected for several other sub-drainage basins within the Mall of America site for the 100-year frequency, 24-hour event (see yellow nodes in Figure 3-1). The predicted inundation areas as a result of the 100-year, 24-hour rainfall event for these sub-drainage basins are also shown in Figure 3-1.

5.1.4 Infiltration Basin at Mall of America Recreational Vehicle Parking Lot (APS-42)

The infiltration basin located within the Mall of America Recreational Vehicle Parking Lot was modeled as being dry prior to a storm event. The basin receives stormwater runoff only from its direct drainage area (Sub-drainage District APS-42) under normal circumstances. XP-SWMM modeling predicts zero discharge from the basin during a 2-year frequency, 24-hour event (Table 5-1), indicating that the runoff from the direct drainage area is not sufficient to fill the basin to levels that result in a discharge from the outlet structure. During large storm events, the basin receives backflow from the City's trunk storm sewer system, thus providing regional stormwater detention. During the 10- and 100-year frequency, 24-hour events, the peak flows from the basin are negative (Table 5-1), indicating backflow from the City's trunk storm sewer system.

5.1.5 Inflows from the Smith Pond Drainage District

As mentioned in Section 3.3.3, there is significant uncertainty in the predicted Wright's Lake outflows from the Smith Pond – Wright's Lake XP-SWMM model, due to the absence of an overflow conveyance from Wright's Lake to TH 77 and the loss of significant runoff volumes from the system (surcharged water was not 'captured'). As such, it is important to note that the peak outflows, the 100-year high water levels and live storage results reported in Table 5-1 for the downstream Pond C (APS-74) should not be considered accurate and should be referenced with caution. Similarly, the XP-SWMM modeling results presented in Figure 3-1 for the storm water system downstream of the inflows from the Smith Pond Drainage District, such as inundation areas and surcharged conditions, should be considered approximate and should be referenced with caution.

5.2 Water Quality Modeling Results and Discussion

When evaluating the results of the P8 modeling, it is important to consider that the results provided can be assumed to be more accurate in terms of relative differences than in the absolute values reported. The model will predict the percent difference in phosphorus reduction between various BMP options in the watershed fairly accurately. It also provides a realistic estimate of the relative differences in phosphorus load and runoff volume from the various P8 drainage basins and major outfalls from the ASDD. However, since runoff quality is highly variable with time and location, the phosphorus load estimated by the model for a specific drainage area may not necessarily reflect the actual load, in absolute terms. Various site-specific factors, such as lawn care or street sweeping practices, illicit point discharges and erosion due to construction are not accounted for in the model. The model provides values that are considered to be typical of the region, given the land uses identified for the watershed in question.

The ASDD was divided into P8 drainage basins to facilitate hydrologic and phosphorus modeling (Figure 4-1). The P8 drainage basins for the Smith Pond and Wright's Lake areas are not shown in Figure 4-1, but remain consistent with the P8 modeling completed in 2002. Storm water and phosphorus contributions were estimated from each P8 drainage basin using the P8 model.

5.2.1 Areal Phosphorus Loading

Figure 5-1 illustrates the areal phosphorus loading (lbs./acre/year) simulated by the P8 model for each P8 drainage basin under existing land use conditions. The color of each P8 drainage basin represents the phosphorus load being generated from each P8 drainage basin. As shown in Figure 5-1, the highest areal phosphorus loading occurs from the areas with the highest imperviousness (extent of hard surfaces such as buildings, and parking lots), such as the Mall of America site and other commercial areas within the drainage district. Impervious areas tend to collect dust, debris, lawn clippings and chemicals, automobile fluids, and trash, which are conveyed to the storm sewer system during runoff events. Note that the areal phosphorus loading represents the amount of pollutant generated and does not account for the pollutant removal achieved by stormwater BMPs.

5.2.2 Pollutant Removal Effectiveness

Much of the ASDD developed well before the era of water quality treatment standards and requirements. As a result, stormwater runoff from portions of the drainage district receives little or no water quality treatment prior to discharge into downstream Long Meadow Lake. The City's efforts to implement onsite water quality treatment requirements for more recent development and redevelopment projects and construction of regional water quality treatment basins have resulted in a reduction in 'untreated areas' and improvements in the quality of water discharged to Long Meadow Lake.

The sediment and phosphorus removal efficiencies of the stormwater BMPs within the ASDD vary based on numerous factors, including the size and type of BMP, the volume and flow rate of storm water treated, and design details of the BMP. For water quality treatment ponds, design factors such as the size, shape, and outlet configuration can impact the pollutant removal effectiveness. The phosphorus removal efficiency of a pond or other BMP can also depend upon the sediment particle distribution of the inflowing storm water. Storm water that has been treated in upstream water quality ponds tends to have a higher fraction of soluble phosphorus, as much of the particulate phosphorus has already been settled out. Soluble phosphorus can be extremely difficult to remove through conventional BMPs, and therefore in some cases, the predicted removal efficiency of a pond can be negatively impacted by inflow from upstream BMPs with high fractions of soluble phosphorus. This effect can be especially pronounced with water quality treatment ponds located at the downstream end of a series of treatment ponds.

Table 5-2 summarizes the predicted total suspended solids and total phosphorus removal efficiencies for an 'average' climatic year and for the 2-year frequency, 24-hour event.

The P8 results indicate that the average annual Total Suspended Solids (TSS) removals range from eight percent (underground treatment structure) to nearly 100 percent (ponds/wetlands in the Minnesota River Valley that do not receive stormwater runoff from upstream drainage basins). The predicted removal efficiencies for the 2-year frequency, 24-hour event are generally lower than the annual averages. This outcome is consistent with the notion that BMPs are generally more effective in treating runoff from the smaller, more typical rainfall events than large storm events.

The P8 results indicate that the average annual Total Phosphorous (TP) removals for the BMPs and natural wetlands within the ASDD range from one percent (underground treatment structure) to 96 percent (ponds/wetlands in the Minnesota River Valley that do not receive stormwater runoff from upstream drainage basins). The predicted removal efficiencies for the 2-year frequency, 24-hour event are also generally lower than the annual averages.

Through on-site and regional BMPs and naturally occurring wetlands, results from the P8 model indicate that 52% of the annual total phosphorus load generated from the Smith Pond and Airport South Drainage Districts is removed prior to discharge to Long Meadow Lake. Approximately 80% of the annual total suspended solids load is removed upstream of Long Meadow Lake.

5.3 Conclusions and Recommendations

5.3.1 Water Quantity (Hydrology & Hydraulics)

5.3.1.1 Storm Sewer System Level of Service

Level of service is defined as the capacity provided by a drainage system to remove runoff and prevent significant interference with normal daily transportation, commerce, or access that might result from a rainstorm. For example, gutters might run full, but when the runoff arrives at a catch basin it would enter the catch basin and be carried away by the storm sewer. Intersections would not flood, and public infrastructure would operate normally. The current standard of practice is usually that conveyance systems be designed for the “10-year” storm event, which means that there is roughly a 10% probability in any year that the system will be overtaxed and unable to meet these criteria. In many communities, older systems were designed for smaller storm events such as a “2-year” event or a “5-year” event. Intersection flooding is common in these areas.

The XP-SWMM modeling results indicate surcharged conditions for some manholes/catch basins within the modeled storm sewer system for events as frequent as a 2-year event (shown in red, Figure 3-1). Most of these manholes/catch basins shown in red are associated with ponding basins (where flood levels beyond normal water levels are expected) or located within parking lots. Parking lot storage can be an effective way to detain stormwater in large storm events. However, as redevelopment occurs in these areas, the City may wish to evaluate the desired frequency of surcharging in these parking areas, as well as the depth of ponding, to avoid significant damage to vehicles parked in the inundated lots.

The XP-SWMM modeling results indicate surcharged conditions for some manholes/catch basins for the 10-year event (shown in orange, Figure 3-1). Many of these surcharged manholes are located within the roadways of the drainage district, indicating that the trunk storm sewer does not provide a 10-year level of service in these areas. As redevelopment occurs in the area or roadway improvements are completed by the City or other entities (state or county), consideration should be given to reducing or eliminating the surcharged conditions for a 10-year frequency event.

5.3.1.2 Storm Sewer System Level of Protection

Level of protection is defined as the capacity provided by a municipal drainage system to prevent property damage and assure a reasonable degree of public safety following a rainstorm. For example, runoff might bypass a catch basin and collect in low-lying areas such as intersections, but would not cause flood damage to structures. Accumulated water might temporarily interfere with traffic or access, but public infrastructure should operate normally. The drainage system must have the capacity (in terms of pipe capacity and overland overflow capacity) to limit the flood elevation to acceptable levels for an event representing the protection criteria.

A 100-year frequency event is usually recommended as a standard for design of ponding basins, thus providing a 100-year 'level of protection' to adjacent property owners. Federal and state programs use criteria based on an event with 1% probability to define the floodplain along rivers and streams, and cities and other drainage authorities commonly extend this standard to other areas. The criterion for level of protection has broader application, as well. In addition to ponding areas, lakes, and streams, this criterion should be applied to all locations served by the drainage system where there are depressed intersections or other areas subject to temporary, unplanned flooding.

Figure 3-1 shows the approximate areas of inundation predicted for the 100-year frequency event. In general, the inundation areas represent ponding basins or natural depressions in the landscape, and significant inundation within the roadways was generally not predicted. However, the extent of inundation at the intersection of 80th Street and 28th Avenue is notable for the 100-year event. As redevelopment of this area occurs, it is recommended that the City evaluate options to reduce the extent of ponding at this intersection during large storm events.

100-year inundation areas are also identified in Figure 3-1 for several parking lot areas, including the parking areas and driving lanes surrounding the Mall of America. It is recommended that the City compare the predicted 100-year flood levels with the low entry elevations of structures nearby the inundation areas, to evaluate the potential for property damage.

5.3.1.3 Inflows from Smith Pond Drainage District

As previously noted, there is significant uncertainty in the predicted Wright's Lake outflows from the Smith Pond – Wright's Lake XP-SWMM model, due to the absence of an overflow conveyance from Wright's Lake to TH 77 and the loss of significant runoff volumes from the system (surcharged water was not 'captured'). As a result, the peak outflows, the 100-year high water levels and live storage results reported in Table 5-1 for the downstream Pond C (APS-74) should not be considered accurate and should be referenced with caution. Similarly, the XP-SWMM modeling results presented in Figure 3-1 for the storm water system downstream of the inflows from the Smith Pond Drainage District, such as inundation areas and surcharged conditions, should be considered approximate and should be referenced with caution. It is recommended that the City consider updating the Smith Pond – Wright's Lake XP-SWMM model to improve the accuracy of flood level predictions for waterbodies within the Smith Pond Drainage District and improve the accuracy of flows to the ASDD.

5.3.2 Water Quality

Because much of the ASDD developed well before the era of water quality treatment standards and requirements, stormwater runoff from portions of the drainage district receive little or no water quality treatment prior to discharge into downstream Long Meadow Lake. However, the City's efforts to require onsite water quality treatment for more recent development and redevelopment projects and construction of regional water quality treatment basins have resulted in a reduction in 'untreated areas' and improvements in the quality of water discharged to Long Meadow Lake.

Results from the P8 modeling indicate that approximately 52% of the total phosphorus load and 80% of the total suspended solids load generated from the Smith Pond and Airport South Drainage Districts are removed prior to discharge to Long Meadow Lake, based on existing land use conditions. Review of the predicted pollutant removal efficiencies indicates that infiltration BMPs are the most effective in removing both total suspended solids and total phosphorus from the stormwater runoff. Infiltration BMPs are especially fitting for the ASDD due to the sandy soils and the proximity to the airport (where standing water may encourage waterfowl populations). It is recommended that the City continue to encourage and/or require implementation of infiltration BMPs as the study area continues to redevelop. In addition, it is recommended that the City seek additional opportunities for infiltration BMPs when evaluating City-sponsored projects within the drainage district such as street reconstruction.

Table 5-1 XP-SWMM Modeling Results for the Airport South Drainage District													
		Sub-drainage Basin Data					Ponding Basin (and Inundation Area) Data						
Subwatershed / Node ID	Manhole ID	Sub-drainage Basin Area (ac)	Percent Impervious (%)	Sub-drainage Basin Peak Runoff Rates			Type	Peak Outflow			NWL (msl)	100-Year HWL (msl)	100-Year Live Storage (ac-ft)
				2-Year (cfs)	10-Year (cfs)	100-Year (cfs)		2-Year (cfs)	10-Year (cfs)	100-Year (cfs)			
APS-1	85M27-1-PVT13	4.4	1.1	1	7	16							
APS-10	85A05-1-PVT10	12.1	80.5	21	37	56	Pond	1	8	16	808.0	812.8	2.3
APS-101	02J24	6.1	79.9	11	19	29							
APS-102	02S15	5.9	40.0	6	15	26							
APS-103	01T18	7.2	80.0	14	23	34							
APS-104	01T26	2.4	80.0	5	8	11							
APS-105	01R26	2.1	80.0	4	7	10							
APS-106	01V26	1.8	80.0	3	6	9							
APS-107	02J25	1.2	80.0	2	4	6							
APS-108	01W39-1-PVT08	4.3	80.0	8	13	20							
APS-109		1.6	79.8	3	5	8	100 Year Inundation Area					806.9	
APS-11		30.0	74.4	52	93	141	Pond	26	45	79	803.3	808.1	3.6
APS-110	01H17-1-PVT01	9.5	80.0	18	30	45	100 Year Inundation Area					820.4	
APS-111[1]	01T08	17.2	55.6	24	51	78	100 Year Inundation Area					808.0	
APS-112	01I07-1-PVT04	6.9	80.0	13	22	33							
APS-113	01L26	3.3	67.9	5	10	15							
APS-114	01T10-2-PVT56	16.5	80.0	30	52	78							
APS-115	01H22	2.7	59.4	4	8	13							
APS-117	01J17	4.5	80.0	9	14	21							
APS-118	01M24	5.8	80.0	11	19	28							
APS-119	01K07	2.0	80.0	4	7	10							
APS-12	85L09-1-PVT05	6.1	41.0	7	17	27	100 Year Inundation Area					819.3	
APS-120		5.9	22.9	3	9	19	100 Year Inundation Area					813.2	
APS-121	02P15	9.4	80.0	18	31	45							
APS-122	02Q14	8.0	79.1	15	26	38							
APS-123	02R05	6.7	80.0	13	22	32							
APS-124	04I52	5.0	39.2	6	14	22							
APS-125[1]	02A06	6.7	55.5	10	20	31	100 Year Inundation Area					806.0	
APS-126	02U05	2.9	50.0	4	8	13							
APS-127	01T10	2.5	80.0	5	8	12							
APS-128	02K15	3.4	78.8	7	11	16							
APS-129	02L08	2.7	80.0	5	9	13							
APS-13	85L09	4.9	75.0	9	15	23							
APS-130		2.7	41.9	3	8	12							
APS-131		2.4	40.0	2	6	10	100 Year Inundation Area					809.7	
APS-132	02T02	5.2	40.4	5	13	22							
APS-133	02T15	15.4	37.2	14	36	64	100 Year Inundation Area					806.6	
APS-134	02M05	4.1	55.5	6	12	19							
APS-135	04N52	0.6	30.1	1	2	3							
APS-136	04T50	1.7	32.2	2	5	7							
APS-137	02T08	3.8	40.0	4	10	17							
APS-138[1]	02H05	6.0	55.4	9	18	27	100 Year Inundation Area					805.8	
APS-139	02M02	2.2	50.0	3	6	10							
APS-14	85L13	1.9	80.0	3	6	9							
APS-140	02S06	0.9	50.0	1	2	4							

Table 5-1
XP-SWMM Modeling Results for the Airport South Drainage District

Subwatershed / Node ID	Manhole ID	Sub-drainage Basin Data					Ponding Basin (and Inundation Area) Data						
		Sub-drainage Basin Area (ac)	Percent Impervious (%)	Sub-drainage Basin Peak Runoff Rates			Type	Peak Outflow			NWL (msl)	100-Year HWL (msl)	100-Year Live Storage (ac-ft)
				2-Year (cfs)	10-Year (cfs)	100-Year (cfs)		2-Year (cfs)	10-Year (cfs)	100-Year (cfs)			
APS-141[1]	02X02	1.6	50.0	2	5	7	100 Year Inundation Area					804.4	
APS-142[1]		3.4	49.9	4	10	15	100 Year Inundation Area					795.2	
APS-143	23E16	5.8	21.5	3	6	12							
APS-144[1]	23G05	7.9	40.5	8	21	34	100 Year Inundation Area					803.8	
APS-145	23N11	12.4	20.8	7	21	43	100 Year Inundation Area					808.2	
APS-146	23S03	6.4	49.6	9	19	29							
APS-147[1]	24B07	3.9	50.0	5	11	18	100 Year Inundation Area					797.2	
APS-148[1]	23U07-1-PVT02	3.8	35.3	4	10	17	100 Year Inundation Area					806.5	
APS-149	23R16-1-PVT01	1.3	40.1	1	4	6							
APS-15	85Y11	3.1	73.1	6	10	14							
APS-150	23N15	2.2	20.0	1	3	6							
APS-151[1]		3.1	50.0	4	9	14	100 Year Inundation Area					796.8	
APS-152	23W07	1.8	50.0	3	5	8	100 Year Inundation Area					796.5	
APS-153	23W08	1.4	50.0	2	4	6							
APS-154	24K13	2.1	36.2	3	6	9							
APS-155[1]	23D03	3.5	50.0	5	10	16	100 Year Inundation Area					804.3	
APS-156		7.8	34.8	7	20	33	Pond/Wetland	2	70	150	704.0	706.7	7.3
APS-157	01N39	2.2	74.5	4	7	10							
APS-158	01V26-1-PVT19	1.5	80.0	3	5	7							
APS-159	01V26-1-PVT06	1.7	100.0	3	5	8							
APS-16	23S16	2.3	32.7	2	5	10							
APS-160	01V26-1-PVT03	0.7	70.0	1	2	3							
APS-161	01Z26-3	5.6	100.0	12	18	27							
APS-162	02J25-1-PVT49	9.7	100.0	20	31	46	100 Year Inundation Area					805.6	
APS-163	02J25-1-PVT19	5.2	100.0	10	15	23	100 Year Inundation Area					822.7	
APS-164	02J25-1-PVT25	0.9	100.0	2	3	5							
APS-165	02J25-1-PVT01	1.7	100.0	4	6	8							
APS-166	02J25-1-PVT04	5.4	100.0	10	17	25							
APS-167	02J25-1-PVT11	1.8	90.0	4	6	9							
APS-169	02J25-1-PVT40	5.4	85.0	10	17	26							
APS-17	86G02-1-PVT17	4.6	81.2	9	15	22							
APS-170	02J25-1-PVT31	3.5	100.0	7	11	16							
APS-171	02J25-1-PVT42	3.8	100.0	8	12	18	100 Year Inundation Area					805.6	
APS-172	02J25-1-PVT44	3.8	95.0	8	13	19	100 Year Inundation Area					805.6	
APS-173	01T10-1-PVT19	2.0	100.0	4	7	10	100 Year Inundation Area					823.0	
APS-174	01T10-1-PVT04	2.5	95.0	5	8	12							
APS-175	01T10-1-PVT14	2.4	90.0	5	8	11	100 Year Inundation Area					820.2	
APS-18	02C52	3.5	79.9	7	11	17							
APS-19	01G43-1-PVT02	5.0	80.0	10	16	24	100 Year Inundation Area					810.2	
APS-2	85L24	3.2	74.5	6	10	15							
APS-20	01M48-1-PVT06	5.9	80.0	12	19	28	100 Year Inundation Area					806.7	
APS-21	01M47	1.3	80.0	3	4	6							
APS-22		3.7	79.6	7	12	18	100 Year Inundation Area					809.3	
APS-23	01G48	3.7	80.0	7	12	17							
APS-24	01L52-1-PVT04	2.8	80.0	6	9	14							

Table 5-1
XP-SWMM Modeling Results for the Airport South Drainage District

Subwatershed / Node ID	Manhole ID	Sub-drainage Basin Data					Ponding Basin (and Inundation Area) Data						
		Sub-drainage Basin Area (ac)	Percent Impervious (%)	Sub-drainage Basin Peak Runoff Rates			Type	Peak Outflow			NWL (msl)	100-Year HWL (msl)	100-Year Live Storage (ac-ft)
				2-Year (cfs)	10-Year (cfs)	100-Year (cfs)		2-Year (cfs)	10-Year (cfs)	100-Year (cfs)			
APS-26	01G52-1-PVT01	1.8	80.0	4	6	9							
APS-27	01G43	0.6	80.0	1	2	3							
APS-28	85G04	1.4	80.0	3	4	6							
APS-29	01M40	2.3	79.1	5	7	11							
APS-3	85L24-1-PVT06	2.7	80.6	5	9	13	Pond	4	5	7	800.0	802.1	0.3
APS-30	01M51	6.7	79.7	13	21	32							
APS-31	01L52	1.5	79.9	3	5	7							
APS-32	01M43	2.4	80.0	5	8	11							
APS-33	01N47	6.2	80.0	11	19	29							
APS-34	01M38-1-PVT03	30.1	0.2	2	25	68							
APS-35	01G42	4.5	82.3	9	15	22							
APS-36	01M29	3.7	80.0	7	12	18							
APS-37	01V39-2-PVT03	1.1	80.0	2	3	5							
APS-38	01R39-1-PVT04	3.1	80.0	6	10	15	100 Year Inundation Area					808.1	
APS-39	01M32-1-PVT02	3.3	80.0	6	11	16							
APS-4	85L21	3.5	80.0	6	11	17							
APS-40	01P39-1	3.8	80.0	7	12	18	100 Year Inundation Area					808.3	
APS-41	01V31	5.0	80.0	9	16	24							
APS-42		30.4	82.1	56	96	144	Infiltration Basin	0	-8	-18	794.2	803.2	10.4
APS-43	02J39	4.1	79.9	8	13	20							
APS-44	01Z39	5.2	80.0	10	17	25							
APS-45	01Z28	3.8	80.0	7	12	18							
APS-46	01Y27-1-PVT12	5.3	80.0	10	17	25	100 Year Inundation Area					807.7	
APS-47	01Z31	4.9	80.0	9	15	23							
APS-48	01M30	1.8	72.4	3	6	8							
APS-49	01M38	1.2	63.8	2	4	5	100 Year Inundation Area					809.7	
APS-5	85L19-1	11.4	79.5	21	37	54							
APS-50[2]	01M31	1.4	70.6	3	5	7	100 Year Inundation Area					809.8	
APS-51	02B39-1-PVT05	6.9	80.9	13	22	33	Pond	2	3	4	808.5	812.1	1.4
APS-52	01Z35-1-PVT08	2.9	80.0	6	9	14							
APS-53	02I45	5.7	39.9	6	15	25							
APS-54	01V39-1-PVT01	8.0	80.7	14	25	38	Pond	10	13	19	801.9	813.2	0.6
APS-55	02M29	15.6	19.7	7	19	40	100 Year Inundation Area					806.7	
APS-56	23U03	2.8	44.3	3	7	11							
APS-57	02F46	3.3	80.0	6	11	16							
APS-58	02J39-1-PVT05	2.2	80.4	4	7	11	Pond	3	4	5	808.5	811.4	0.3
APS-59		6.3	34.6	6	16	27	Pond	4	8	11	803.5	807.0	0.9
APS-60	01M41	8.1	41.7	8	17	30							
APS-61	23A32-1-PVT15	10.8	80.1	21	35	51	Pond	20	31	42	796.0	801.3	0.6
APS-62	23A28	1.7	23.6	1	3	6							
APS-63	02W27-1	9.3	26.2	6	14	27							
APS-64	02Q26	1.5	3.2	1	4	6							
APS-65	23L40	77.9	35.1	85	184	314	Wetland	4	12	24	702.0	704.3	22.3
APS-66	23L43	0.6	47.6	1	2	3	Pond	249	253	256	696.0	704.3	1.2
APS-67	23V36	11.3	93.1	25	38	55	Pond	9	11	13	696.0	704.1	56.6
APS-68	23E41	29.7	0.0	4	43	99							

Table 5-1
XP-SWMM Modeling Results for the Airport South Drainage District

Subwatershed / Node ID	Manhole ID	Sub-drainage Basin Data: Sub-drainage Basin Area (ac)	Percent Impervious (%)	Sub-drainage Basin Peak Runoff Rates: 2-Year (cfs)	10-Year (cfs)	100-Year (cfs)	Ponding Basin (and Inundation Area) Data: Type	Peak Outflow: 2-Year (cfs)	10-Year (cfs)	100-Year (cfs)	NWL (msl)	100-Year HWL (msl)	100-Year Live Storage (ac-ft)
APS-69		9.9	51.6	17	30	46	Pond/Wetland	1	2	3	701.4	702.0	2.0
APS-70	24A15	3.2	25.1	2	7	13							
APS-71	24M19	10.8	47.7	14	32	49							
APS-72	24O18	1.7	50.0	3	5	8							
APS-73	24Q18	1.2	50.0	2	4	6							
APS-74[1]		25.8	42.7	40	76	117	SW	101	135	168	708.5	727.4	252.7
APS-75[1]		11.8	61.4	21	37	55	Pond	-9	-48	-393	698.5	704.1	18.5
APS-76	23R19	2.4	20.0	2	6	10	Depression	2	6	17	783.9	789.8	0.9
APS-77	23Q23	6.4	19.8	3	9	20							
APS-78	23R02	2.0	30.5	2	5	9							
APS-79	23S20	1.0	20.0	1	2	4							
APS-8	85L20	2.5	80.0	5	8	12							
APS-80		0.9	40.0	1	3	4	100 Year Inundation Area					810.1	
APS-81		3.2	20.0	2	6	11	100 Year Inundation Area					810.8	
APS-82	23I16	10.5	20.0	5	16	33							
APS-83	02R24	4.0	49.0	5	11	18							
APS-84	23I20	1.5	19.6	1	4	6							
APS-85	23L22	10.7	19.3	5	17	35							
APS-86	23N17	6.0	20.1	3	9	19							
APS-87	23B23	1.9	29.8	2	4	8							
APS-88	23N22	5.3	20.4	3	9	18							
APS-89	23A16	7.1	28.0	5	16	29							
APS-9	85M15	7.1	79.6	14	23	34							
APS-90	23W13	2.2	45.0	3	6	10							
APS-91	23N19	2.0	20.0	1	5	8	100 Year Inundation Area					801.0	
APS-92	01V26-6	1.6	85.0	3	5	8							
APS-93	01R23	23.5	80.0	27	50	84	100 Year Inundation Area					816.0	
APS-94	02J25-1-PVT55	8.9	100.0	18	29	42	100 Year Inundation Area					805.8	
APS-95	02J26	1.7	80.0	3	5	8							
APS-96	01Z26	1.0	80.0	2	3	5							
APS-97	01T10-1-PVT22	4.6	84.9	9	15	22							
APS-98	02N15	2.6	80.0	5	8	12							
APS-99	02N27-1-PVT04	4.0	0.5	1	6	13							

[1] Basin Data results may be inaccurate due to uncertainty of predicted inflows from the Smith Pond-Wright 's Lake XP-SWMM model.

[2] 100 year inundation elevation is from manhole 01M35.

Table 5-2 Pollutant Removal Efficiencies

ID	Type of Water Quality Treatment Device	Annual TSS Load Reduction (%)[1]	Annual TP Load Reduction (%)[1]	2-year, 24-hour TSS Load Reduction (%)	2-year, 24-hour TP Load Reduction (%)
OVERALL		80	52	69	31
204-SP	Pond	76	45	65	24
303b-WL	Pond	64	31	44	12
APS-10	Pond	87	58	76	35
APS-11	Pond	81	50	61	23
APS-156	Pond	99	70	99	48
APS-3	Pond	87	57	69	29
APS-42[2]	Infiltration Basin	99	95	100	95
APS-51	Pond	76	45	57	19
APS-58	Pond	68	35	58	19
APS-59	Pond	62	28	57	19
APS-61	Pond	46	16	35	7
APS-63	Pond	27	4	27	3
APS-65	Pond	99	71	98	48
APS-75	Pond	100	16	98	20
APS-104a	Infiltration Basin	99	96	100	96
APS-104b	Underground Structure	46	19	31	6
APS-104c	Infiltration Basin	99	96	100	96
APS-69	Pond	100	15	100	14
APS-66	Pond	20	3	14	1
APS-67	Pond	65	37	62	26
APS-74	Pond	79	44	64	24
APS-2	Underground Structure	8	1	5	<1

1 - Based on 1995 Water Year (October 1/1994 - 9/30/1995) which is considered an average precipitation year

2 - Results reflect removal from direct drainage area only, and do not reflect that stormwater from the trunk storm sewer system will backflow into the basin during larger storm events

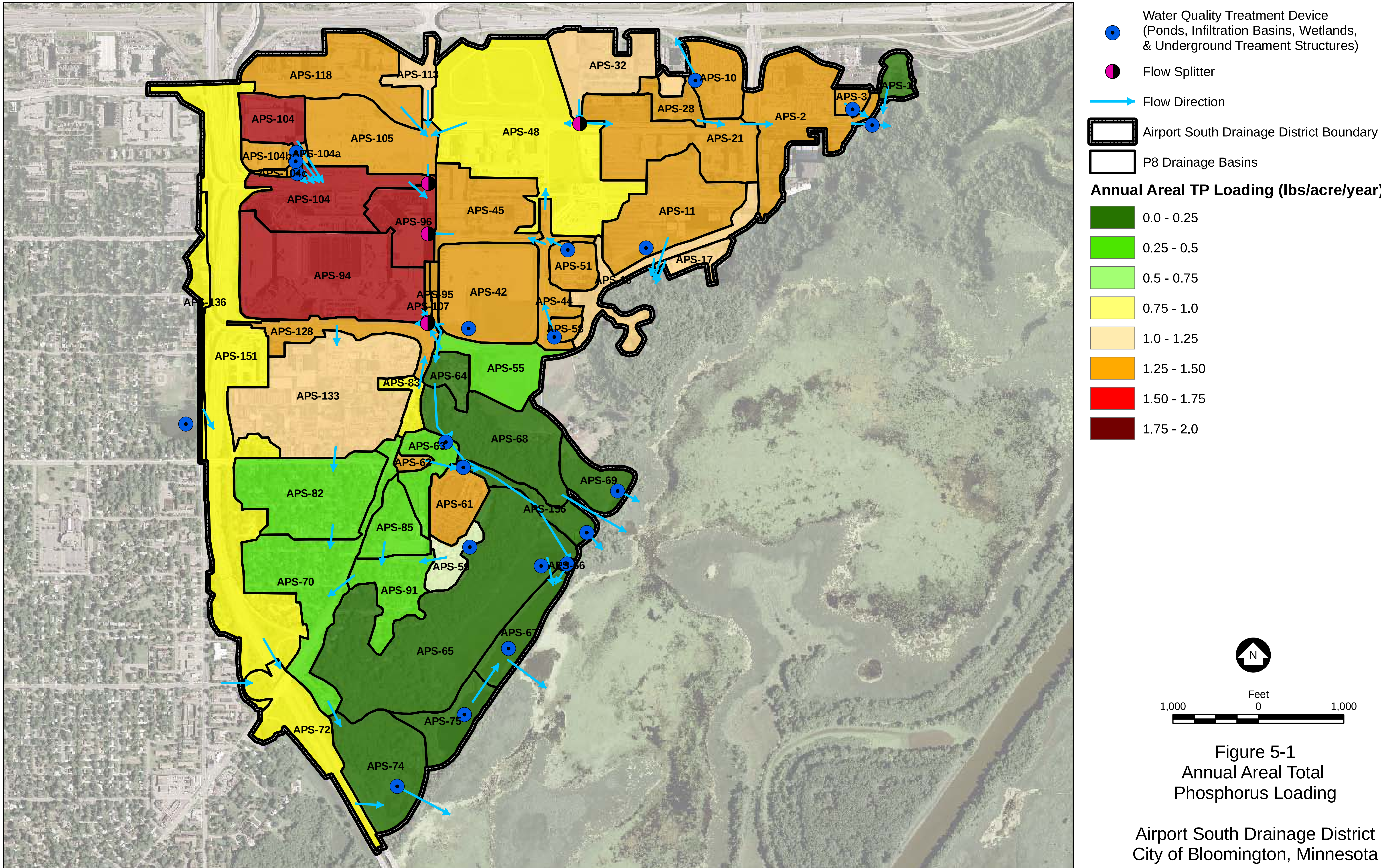

Figure 5-1
Annual Areal Total
Phosphorus Loading

Airport South Drainage District
City of Bloomington, Minnesota

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Appendix A

December 2, 1998 Memorandum from Montgomery Watson regarding XP-SWMM Model Calibration

MEMORANDUM

MONTGOMERY WATSON

To:	Shelly Pederson, Bloomington Scott Anderson, Bloomington Jenny Ross, SRF Dave Filipiak, SRF	**Date:**	December 2, 1998
From:	Eric Thompson	**Reference:**	
Subject:	XP-SWMM Model Calibration		

The following memorandum is a brief discussion of the efforts undertaken to calibrate the XP-SWMM model covering the eastern portions of Bloomington, specifically the Airport South (Mall of America) drainage area.

The memorandum briefly discusses rain and flow gauge data and the quality of the data, and summarizes the specific alterations made to the model to calibrate the model to gauged events.

At this time, I feel that the XP-SWMM model is satisfactorily calibrated. This conclusion is supported by Figures 13-17 at the end of this memorandum which verify model performance against gauged data.

RAIN DATA

A single continuous recording tipping bucket rain gauge was used to gather rainfall intensity data for use in calibrating the XP-SWMM model. The gauge was located in the right turn lane island at northbound Old Shakopee Road where it meets Killebrew Drive at the southeast corner of the Mall of America.

The gauge was installed in early May 1998 and was removed in late July 1998. The gauge measured incremental rainfall depths at 5-minute increments with a minimum sensitivity of 0.01 inches. During the monitoring period, the gauge captured five rainfall events for which there was also good flow data at both flow gauging locations (discussed below).

The five rainfall events cover a very wide range of storm types. For example, the 6-24 storm is a long duration storm of rather light intensity. Conversely, there is the 6-25 storm which is a short duration but high intensity. The remaining three storms cover a wide range of durations but all have comparatively moderate intensities.

Table 1

Date	Rainfall Depth (inches)	Rainfall Duration (hours)	Average Intensity (inches/hour)
6-20-98	0.54	1:50	0.29
6-24-98	0.99	7:00	0.14
6-25-98	0.45	0:55	0.49
6-26-98	3.01	7:45	0.39
7-14-98	1.38	4:40	0.30

FLOW DATA

Two continuous recording area-velocity meters were used to gather flow data for calibrating the XP-SWMM model. The gauges were located in isolated subwatersheds chosen to provide insight into portions of the overall Airport South drainage area with different development conditions.

To measure flows from a largely commercial area with very high percentages of impervious ground cover, a gauge was placed at Metro Drive just north of 80th Street. The gauge was placed within the 36-inch RCP storm sewer running north to south at the manhole approximately 25 feet north of 80th Street. This gauge had a tributary area of approximately 25.4 acres.

To gauge flows from a residential area with very low impervious cover, a gauge was placed within the 27-inch RCP storm sewer flowing from east to west towards Old Shakopee Road at 88th Street. The gauge was located within a manhole in the backyard of the residence at the intersection of Old Shakopee Road and 88th Street. This gauge had a tributary area of approximately 18.6 acres.

The gauges were installed in early May 1998 and were removed in late July 1998. The gauges measured flow depth and velocity at 5-minute increments with a manufacturer-specified measurement accuracy of 12%. Flow depth was internally converted to flow area electronically within the gauge. Flow rate was later calculated using a spreadsheet by multiplying the incremental velocity measurement by the flow area. During the monitoring period, the gauges captured five runoff events for which there was good flow data at both flow gauge locations and good rainfall data.

FLOW GAUGE DATA ANALYSIS

When analyzing the flow data for anomalies, it was found that on a few occasions measured flow values would oscillate from positive flow valves to zero and back again. This behavior was evident at both gauge locations during the 6-26-98 rainfall event which had a period of extreme intensity. It was felt that the zero readings were false values which could bias the model calibration.

To best deal with the zero value data points, a rating curve of flow as a function of depth was developed for both gauge sites. This was accomplished by plotting all measured data for each gauge on a single graph. Figures 1 and 2 represent the respective rating curves for each gauge site when the statistical outliers were removed from the equation. The resulting fifth order equations fit through the data points had very good accuracy.

Figures 3 through 12 show plots of the raw data and the data adjusted according to the rating curves developed for each gauge. As can be seen, the rating curve adjustments had little effect on any hydrograph other than the 6-26-98 rainfall.

MODEL CALIBRATION

The XP-SWMM model was calibrated through alteration of the runoff subroutine which contains the majority of the qualitative parameters controlling rainfall runoff volume and discharge rates. Specifically, calibration was accomplished through alteration of the values for directly connected impervious area (DCIA), initial and final infiltration rates, the infiltration decay rate, and the initial abstractions.

It was found through spreadsheet analysis and some trial and error fine tuning that the following input values worked best:

Parameter		Original Value	Calibrated Value
DCIA Multiplier		1.00	0.70
Initial Infiltration	f_o	5.00 in/hr	6.00 in/hr
Final Infiltration	f_c	0.45 in/hr	0.20 in/hr
Infiltration Decay Rate	K	0.00056 sec^{-1}	0.00139 sec^{-1}
Impervious Area Initial Abstractions	I_a	0.017-0.042 in	0.05 in
Pervious Area Initial Abstractions	I_a	0.200 in	0.10 in

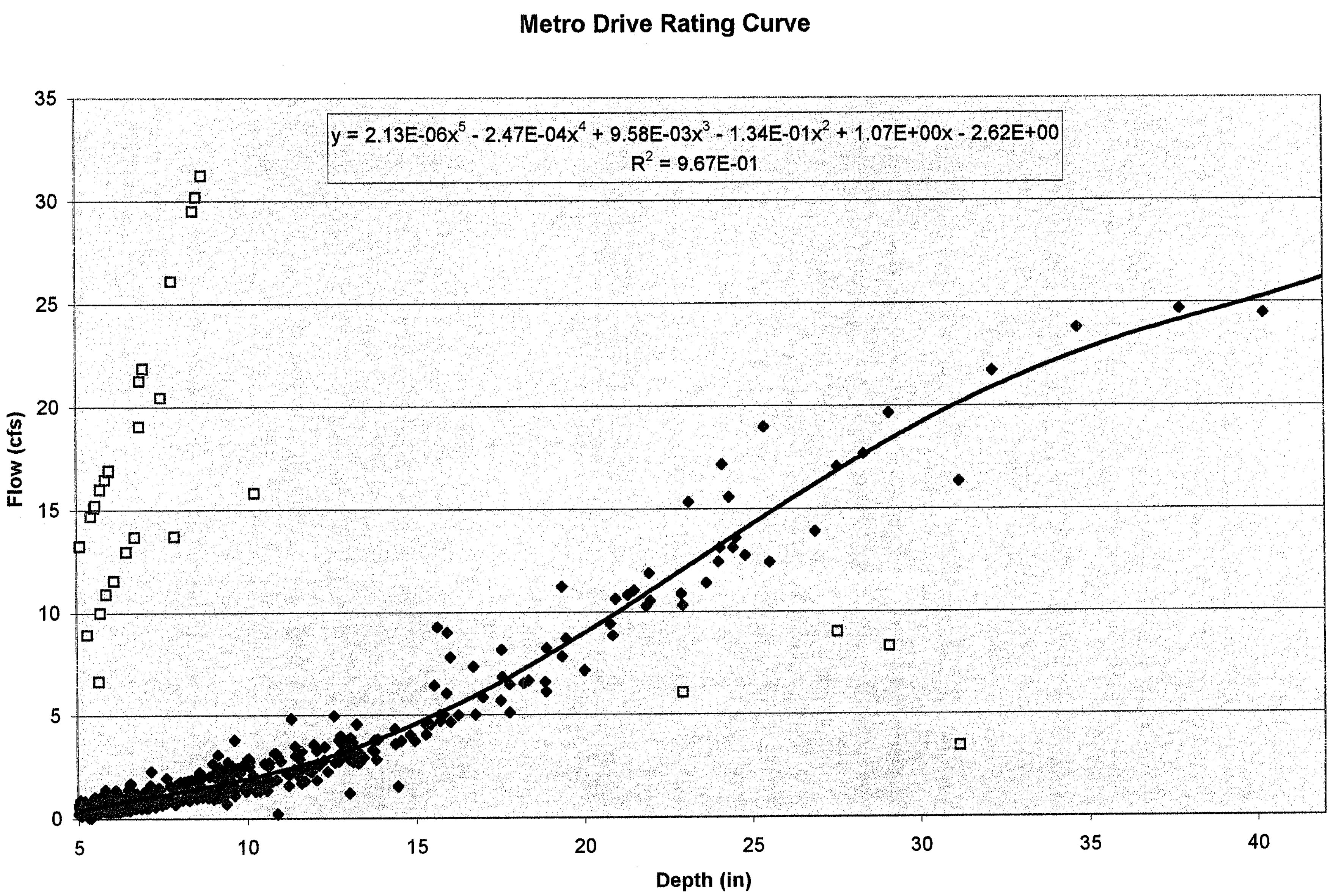
These values resulted in model input values well within accredited reference values and produced model outputs which closely matched gauged data.

The use of a DCIA multiplier resulted from the need to extrapolate the calibration of the model to other subwatersheds with different land uses. Multiplying the original model input values for DCIA by 0.70 (i.e., reducing them by 30%) was an integral part of the calibration and was felt to be acceptable as it worked well for both gauged sites.

Figures 14 through 18 show plots of the model output and the adjusted (rating curve) flow data for each storm event gauged. As can be seen, the model does a very good job of reproducing gauged behavior for each of the five storms analyzed. It is felt that the XP-SWMM model has been calibrated and its behavior has been verified by these graphs.

cc: Paul Nelson

Enclosure: Calibrated model files



Metro Drive Rating Curve
$y = 2.13E\text{-}06x^5 - 2.47E\text{-}04x^4 + 9.58E\text{-}03x^3 - 1.34E\text{-}01x^2 + 1.07E{+}00x - 2.62E{+}00$
$R^2 = 9.67E\text{-}01$
Flow (cfs)
Depth (in)
0
5
10
15
20
25
30
35
40

FIGURE 1

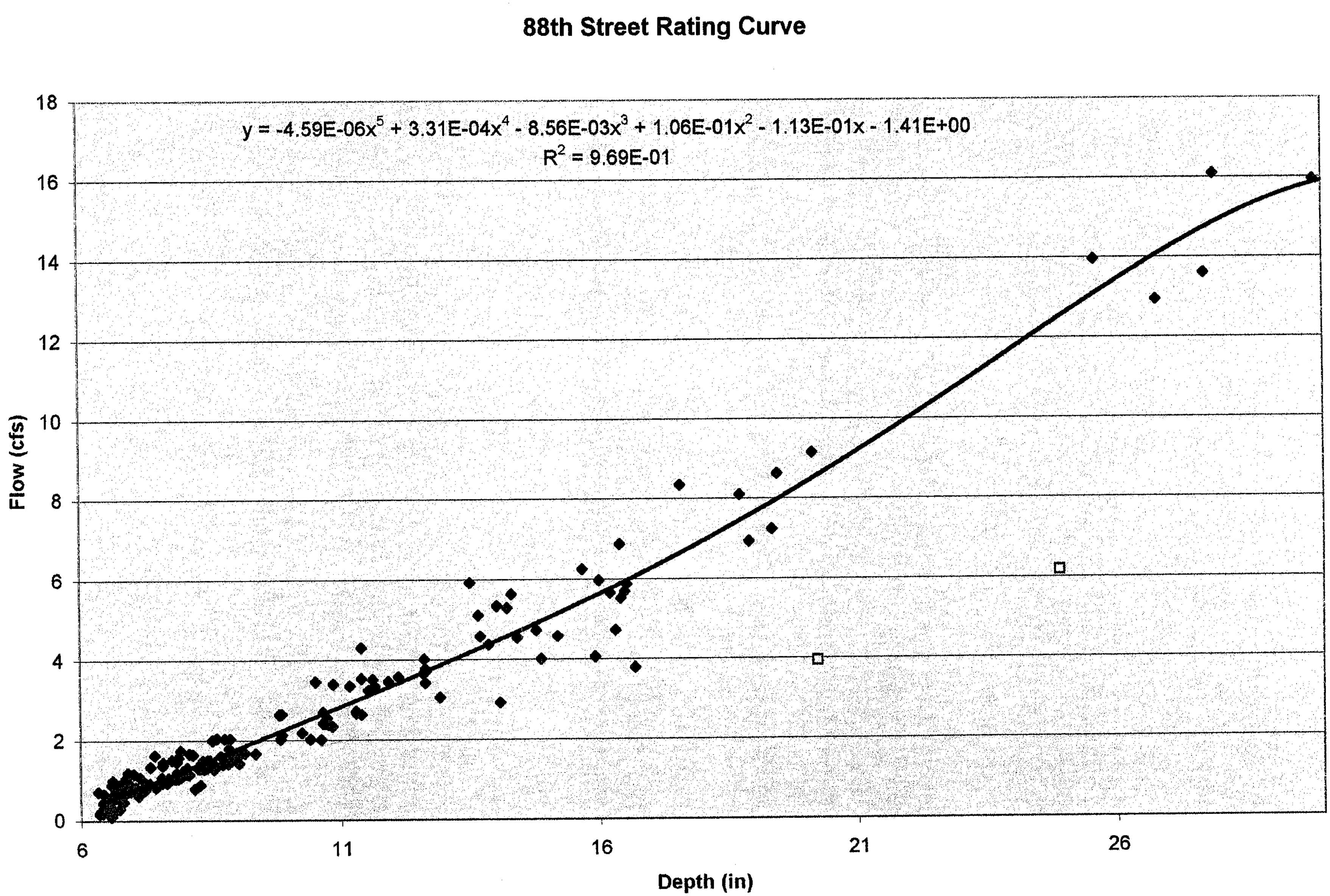

88th Street Rating Curve
y = -4.59E-06x^5 + 3.31E-04x^4 - 8.56E-03x^3 + 1.06E-01x^2 - 1.13E-01x - 1.41E+00
R^2 = 9.69E-01
Flow (cfs)
0
2
4
6
8
10
12
14
16
18
6
11
16
21
26
Depth (in)

FIGURE 2

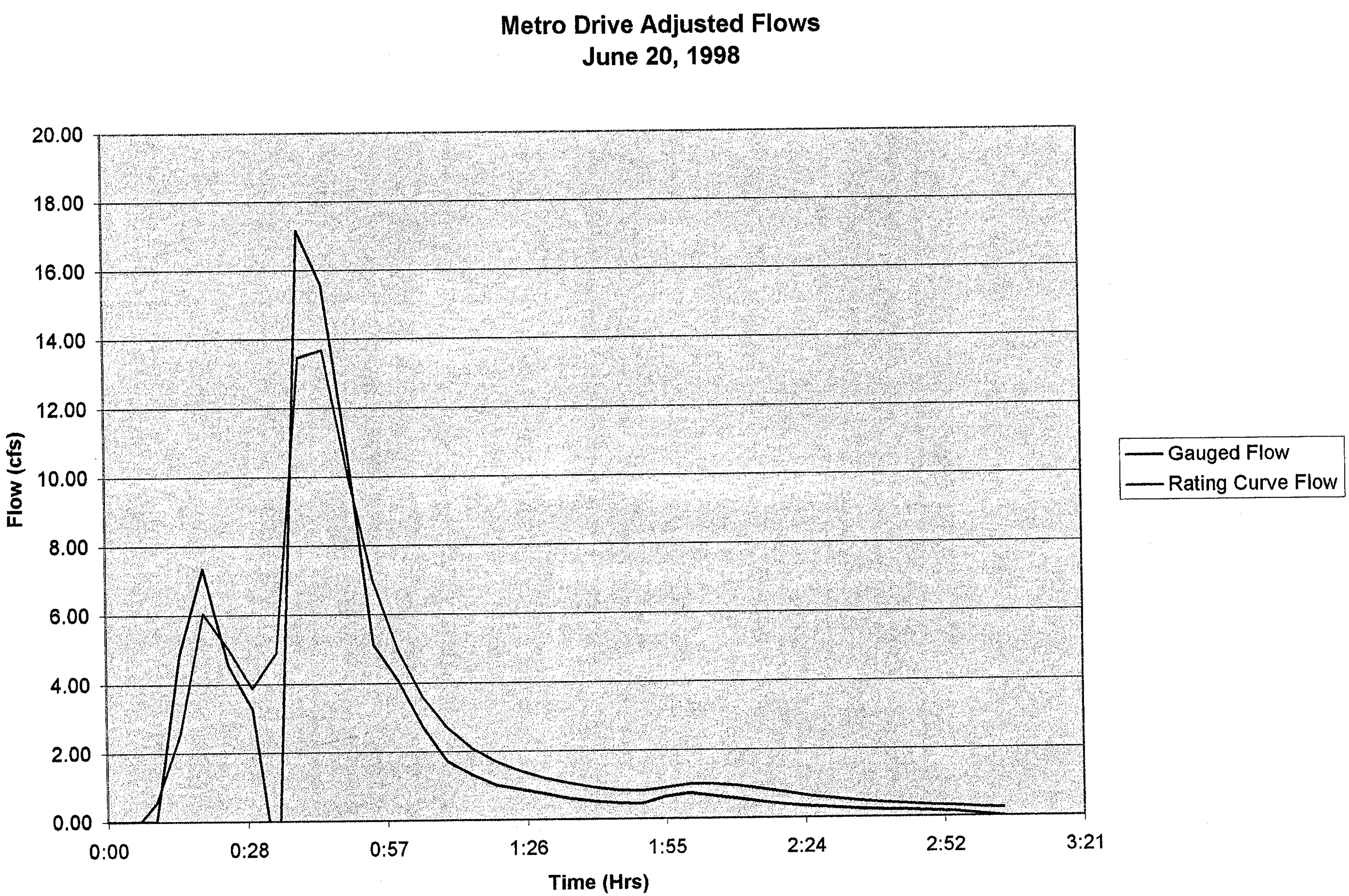

Metro Drive Adjusted Flows
June 20, 1998
20.00
18.00
16.00
14.00
12.00
10.00
8.00
6.00
4.00
2.00
0.00
Flow (cfs)
0:00
0:28
0:57
1:26
1:55
2:24
2:52
3:21
Time (Hrs)
Gauged Flow
Rating Curve Flow

FIGURE 3

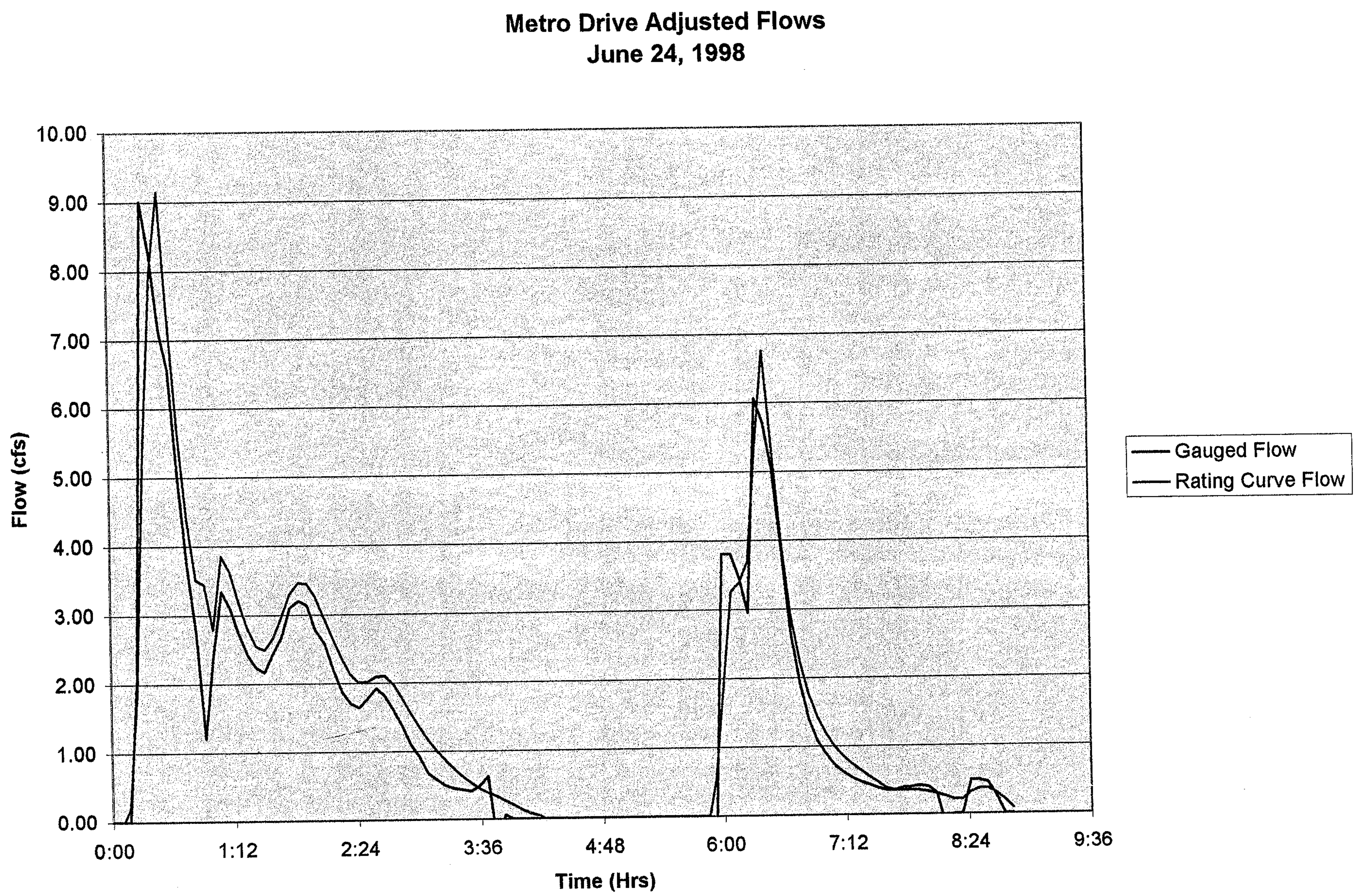

Metro Drive Adjusted Flows
June 24, 1998
Flow (cfs)
10.00
9.00
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
0:00
1:12
2:24
3:36
4:48
6:00
7:12
8:24
9:36
Time (Hrs)
Gauged Flow
Rating Curve Flow

FIGURE 4

Metro Drive Adjusted Flows
June 25, 1998

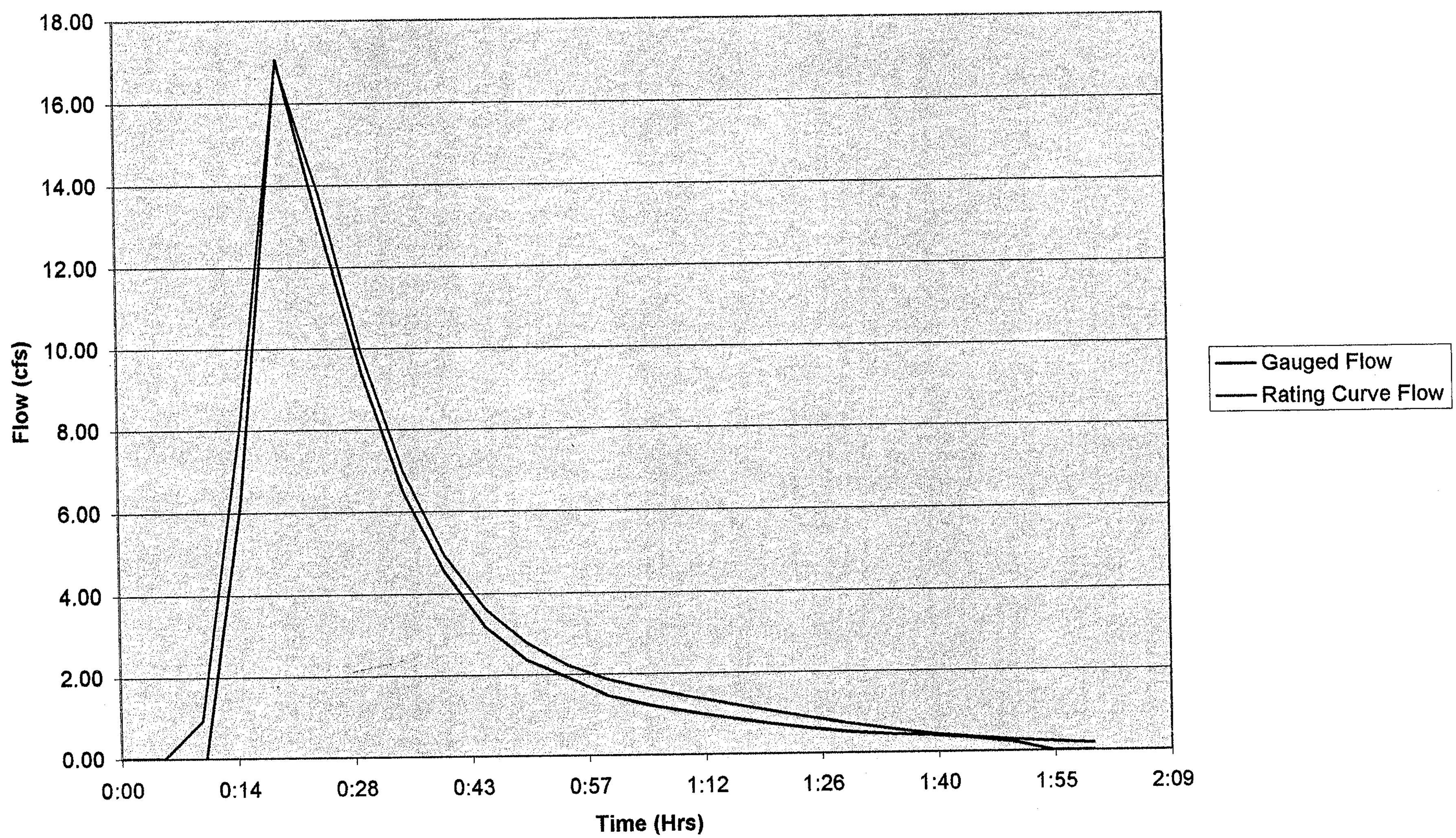


FIGURE 5

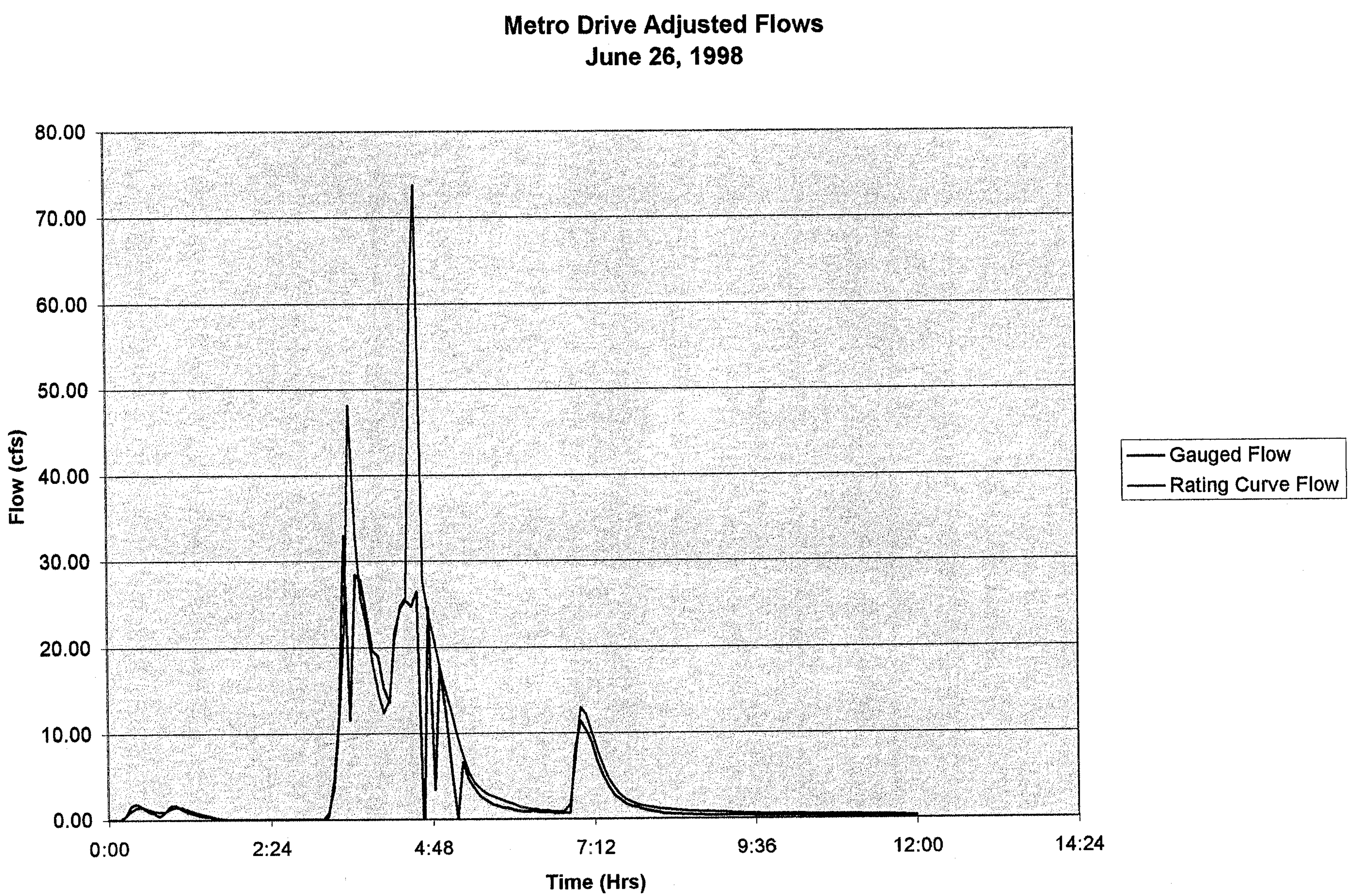

Metro Drive Adjusted Flows
June 26, 1998
80.00
70.00
60.00
50.00
40.00
30.00
20.00
10.00
0.00
Flow (cfs)
0:00
2:24
4:48
7:12
9:36
12:00
14:24
Time (Hrs)
Gauged Flow
Rating Curve Flow

FIGURE 6

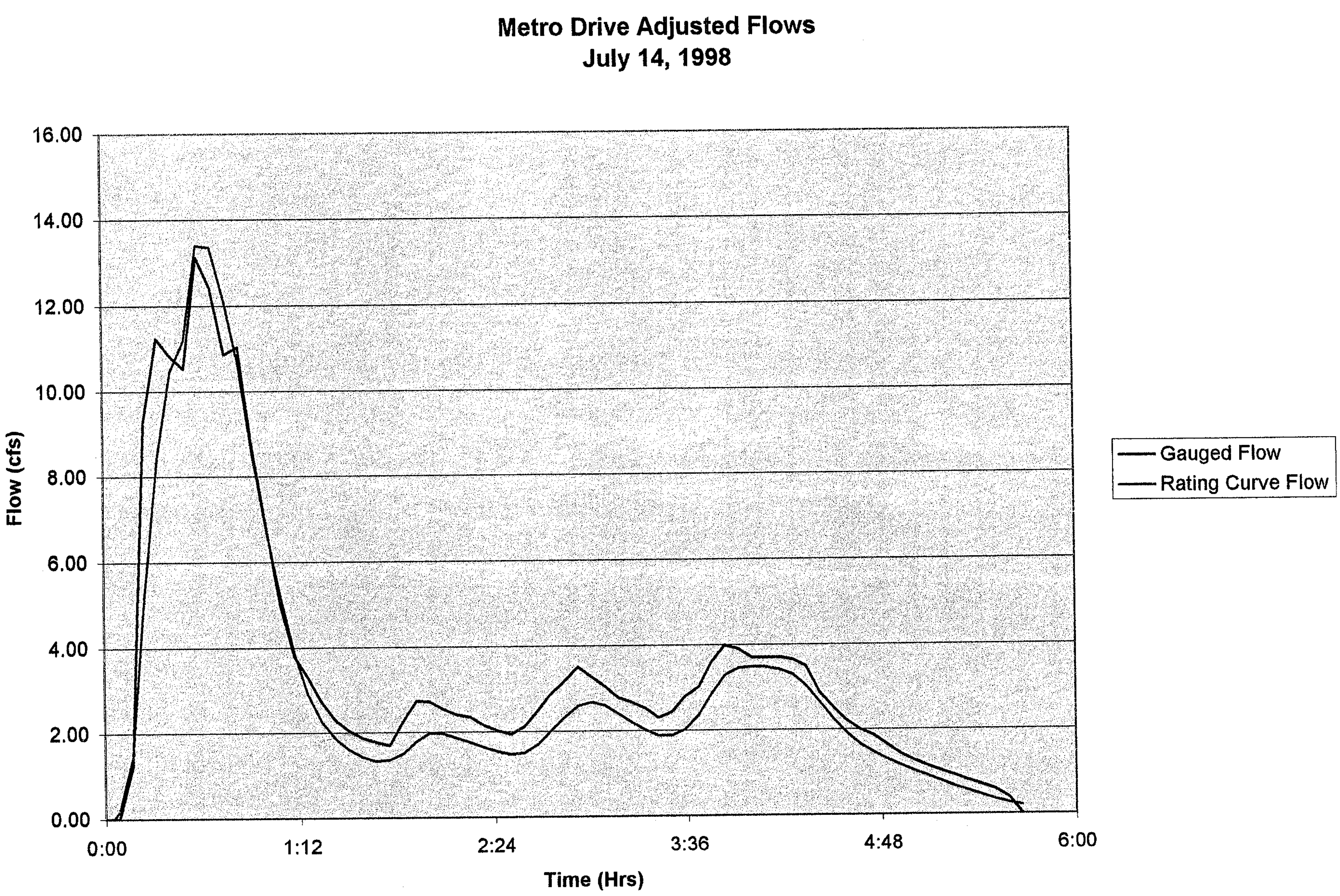

Metro Drive Adjusted Flows
July 14, 1998
16.00
14.00
12.00
10.00
8.00
6.00
4.00
2.00
0.00
Flow (cfs)
0:00
1:12
2:24
3:36
4:48
6:00
Time (Hrs)
Gauged Flow
Rating Curve Flow

FIGURE 7

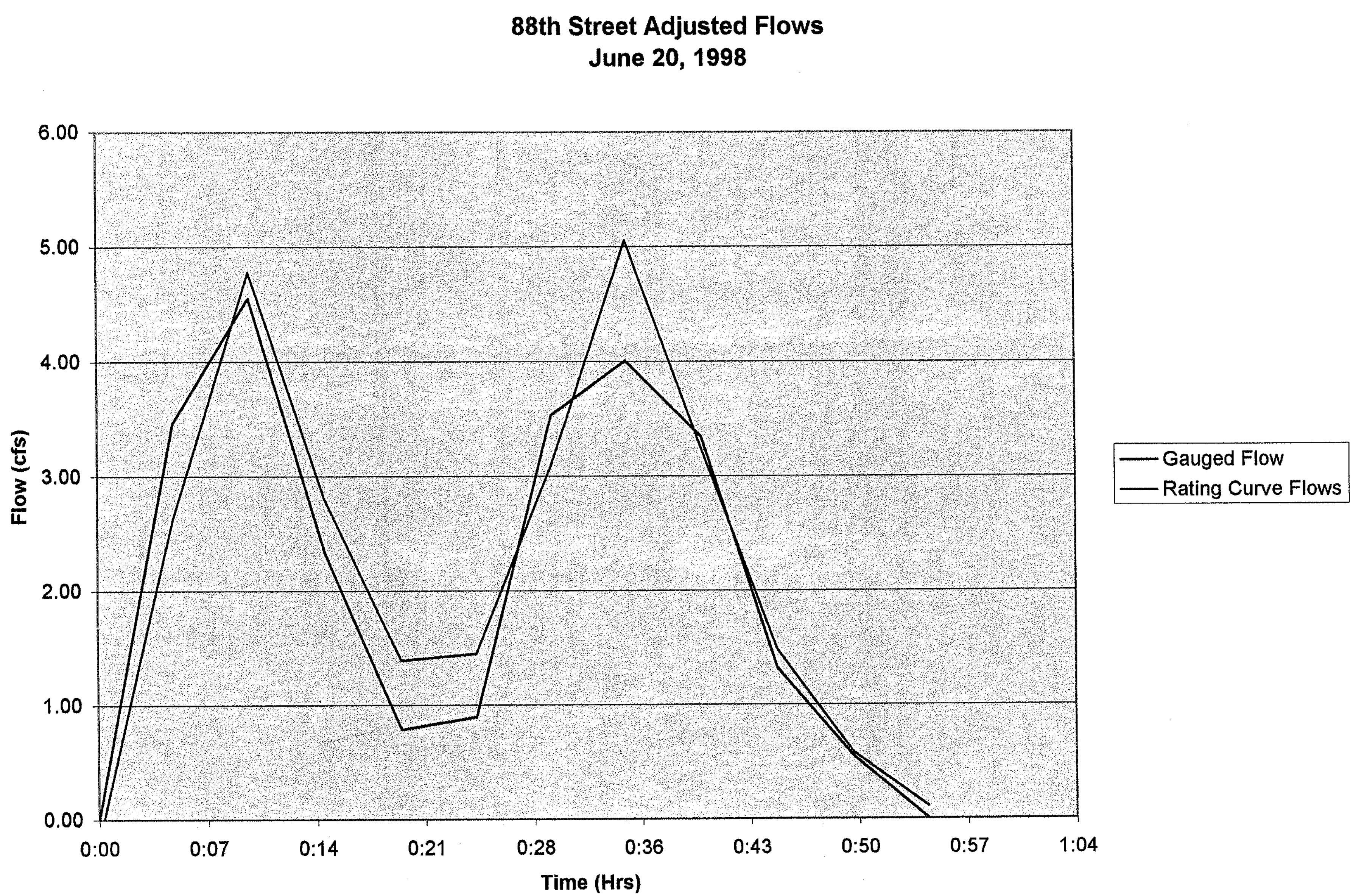

88th Street Adjusted Flows
June 20, 1998
6.00
5.00
4.00
3.00
2.00
1.00
0.00
Flow (cfs)
0:00
0:07
0:14
0:21
0:28
0:36
0:43
0:50
0:57
1:04
Time (Hrs)
Gauged Flow
Rating Curve Flows

FIGURE 8

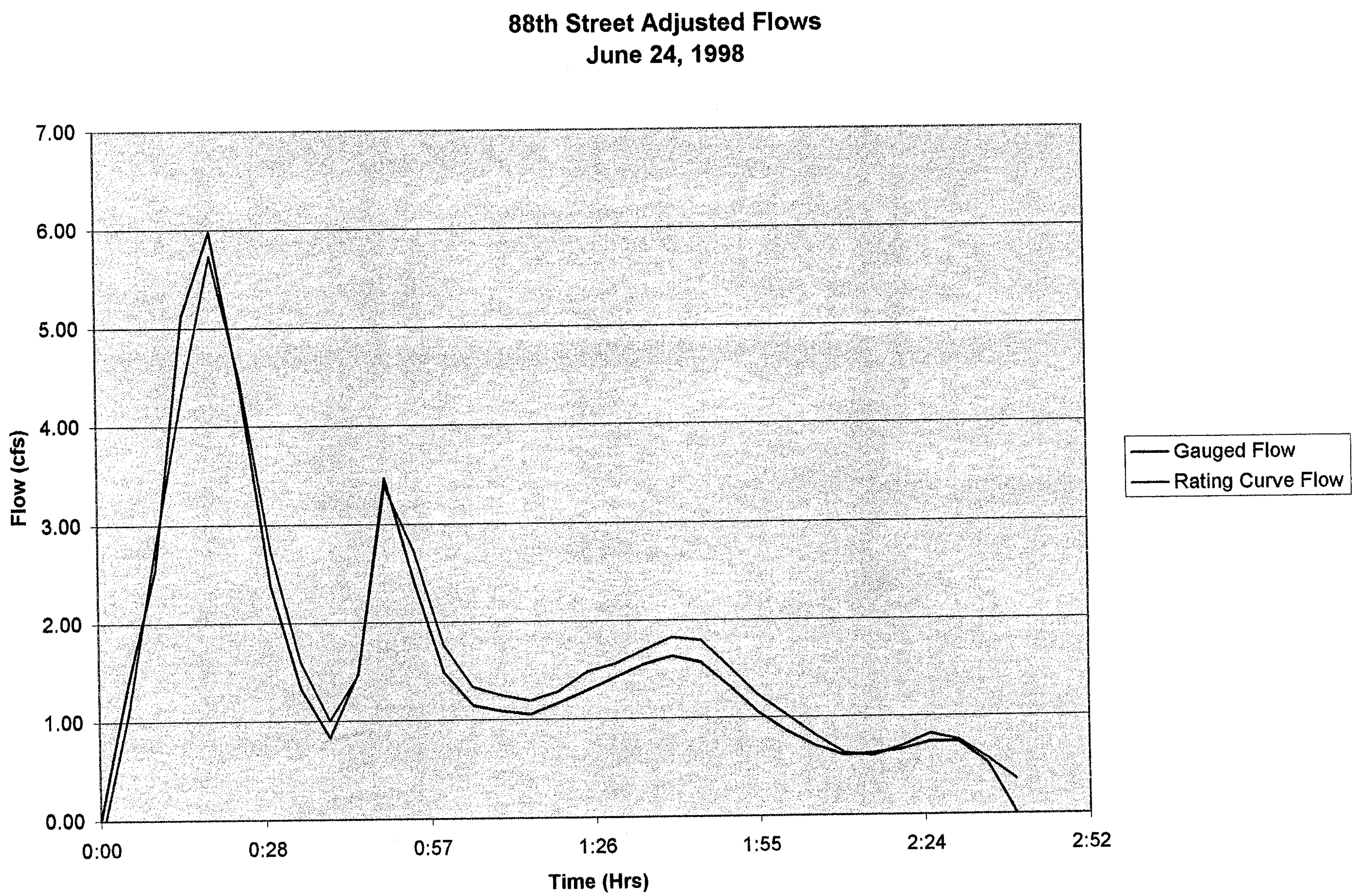

88th Street Adjusted Flows
June 24, 1998
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
Flow (cfs)
0:00
0:28
0:57
1:26
1:55
2:24
2:52
Time (Hrs)
Gauged Flow
Rating Curve Flow

FIGURE 9

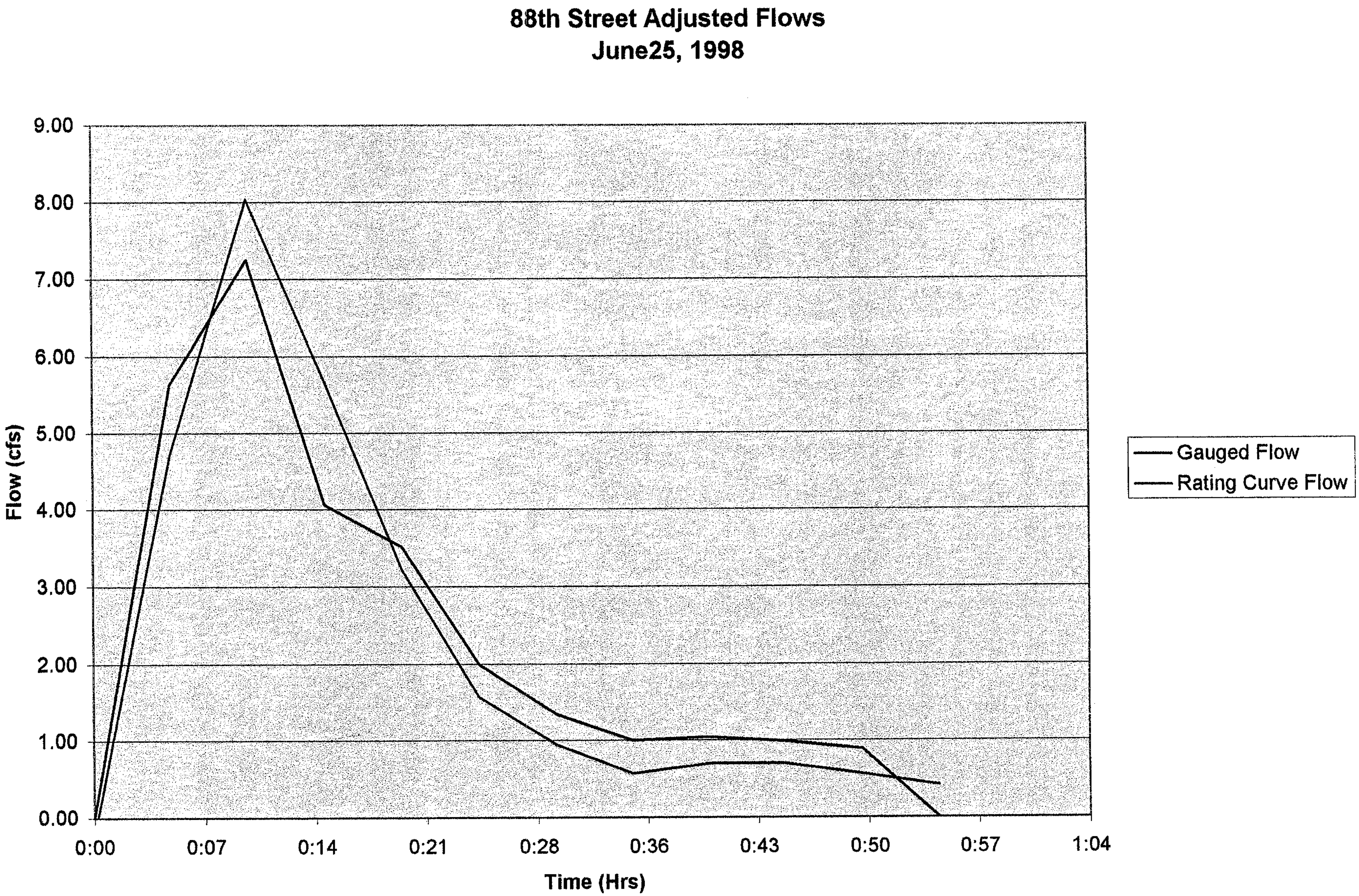
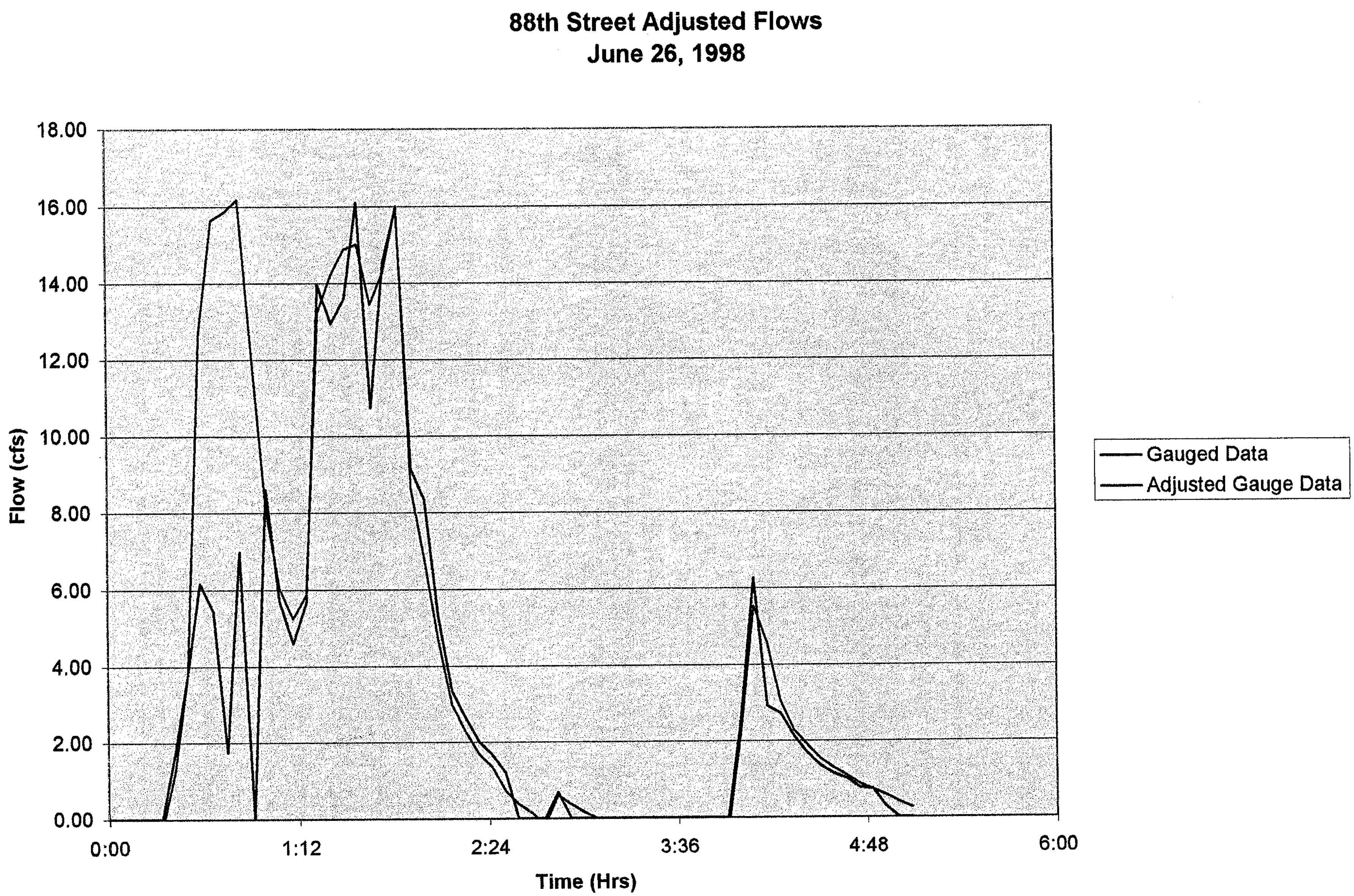

88th Street Adjusted Flows
June25, 1998
9.00
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
Flow (cfs)
0:00
0:07
0:14
0:21
0:28
0:36
0:43
0:50
0:57
1:04
Time (Hrs)
Gauged Flow
Rating Curve Flow

FIGURE 10



88th Street Adjusted Flows
June 26, 1998
18.00
16.00
14.00
12.00
10.00
8.00
6.00
4.00
2.00
0.00
Flow (cfs)
0:00
1:12
2:24
3:36
4:48
6:00
Time (Hrs)
Gauged Data
Adjusted Gauge Data

FIGURE 11

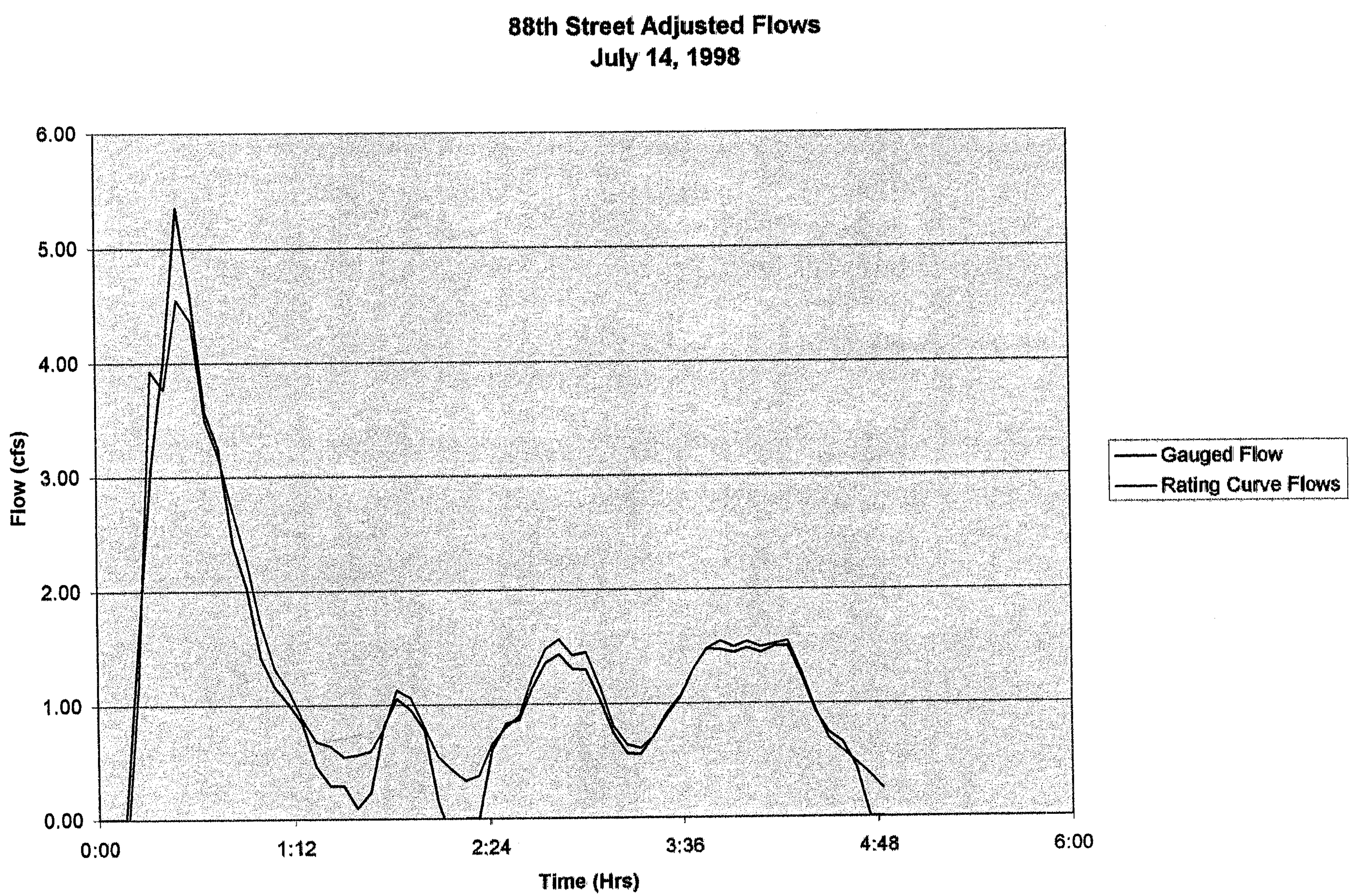

88th Street Adjusted Flows
July 14, 1998
6.00
5.00
4.00
3.00
2.00
1.00
0.00
Flow (cfs)
0:00
1:12
2:24
3:36
4:48
6:00
Time (Hrs)
Gauged Flow
Rating Curve Flows

FIGURE 12

June 20, 1998
Calibrated

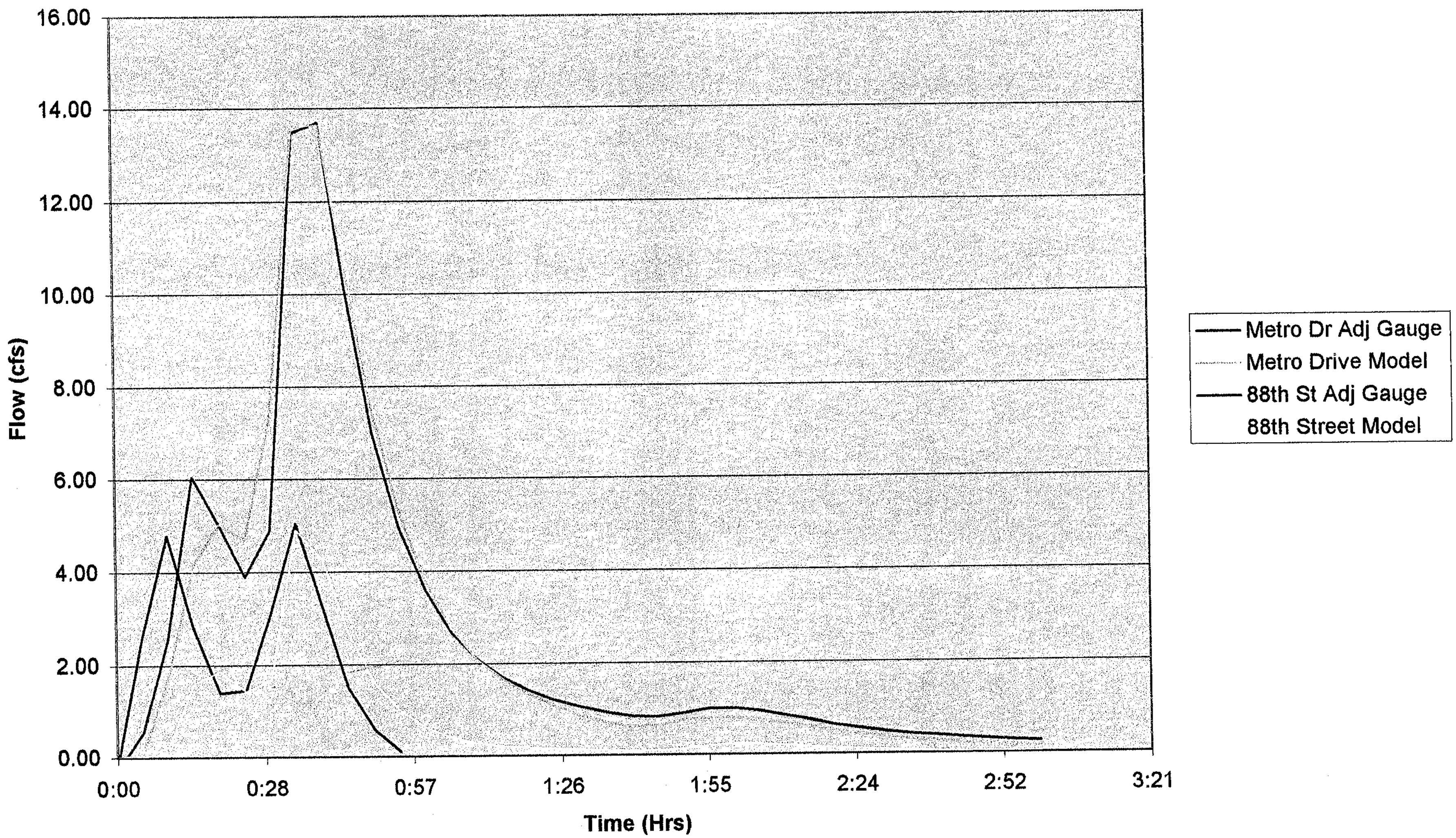


FIGURE 13

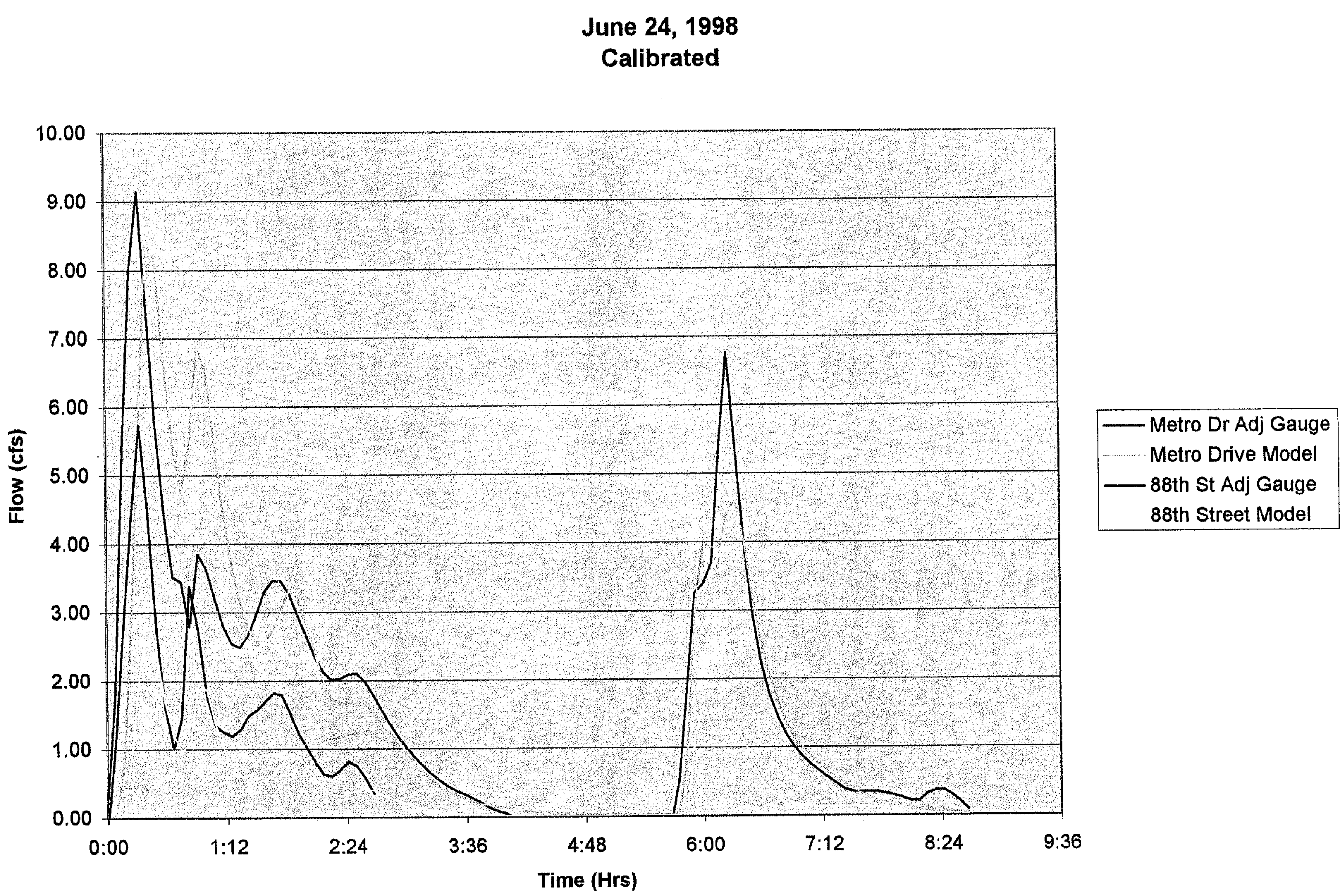

June 24, 1998
Calibrated
10.00
9.00
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
Flow (cfs)
0:00
1:12
2:24
3:36
4:48
6:00
7:12
8:24
9:36
Time (Hrs)
Metro Dr Adj Gauge
Metro Drive Model
88th St Adj Gauge
88th Street Model

FIGURE 14

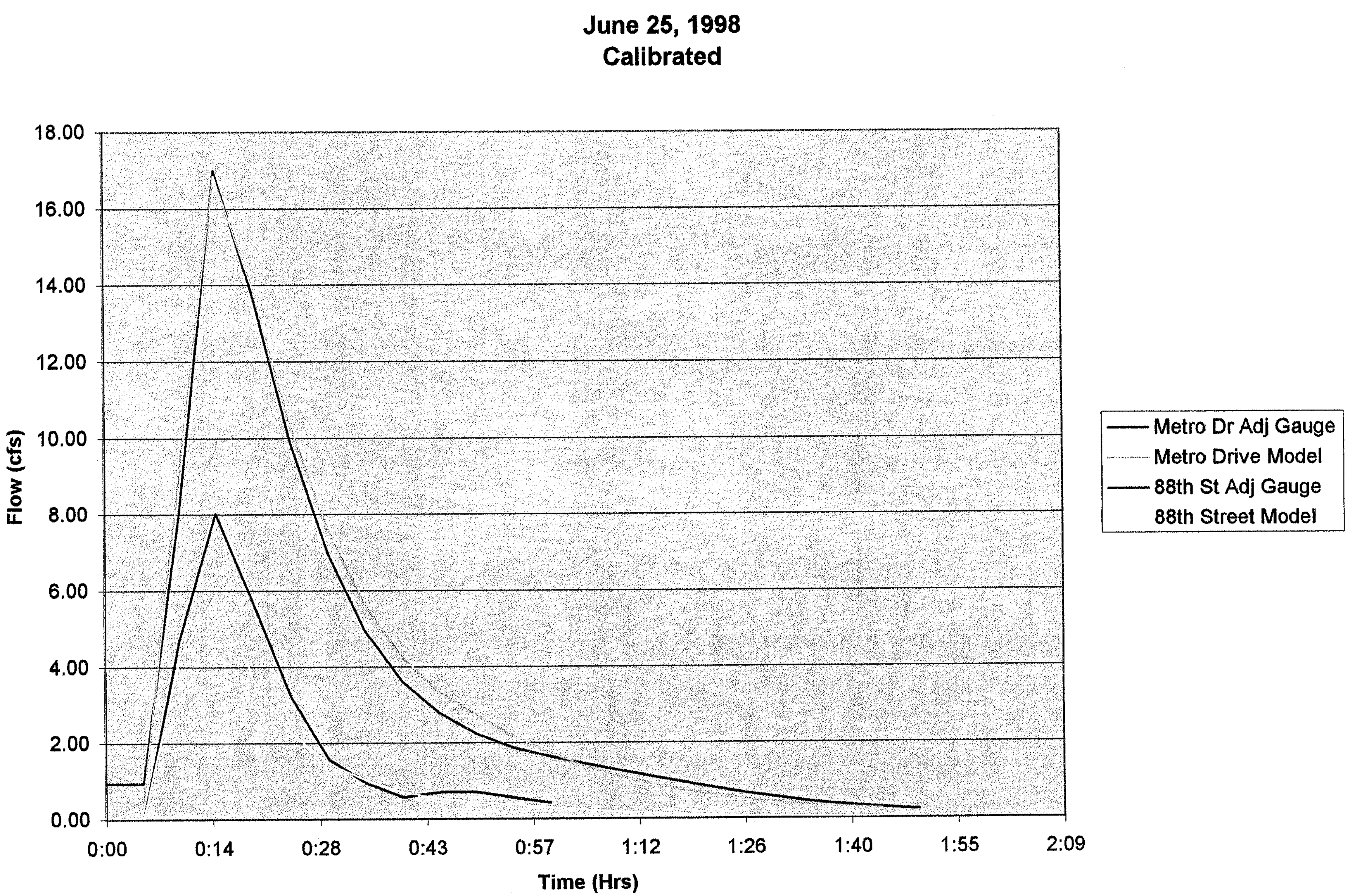

June 25, 1998
Calibrated
18.00
16.00
14.00
12.00
10.00
8.00
6.00
4.00
2.00
0.00
Flow (cfs)
0:00
0:14
0:28
0:43
0:57
1:12
1:26
1:40
1:55
2:09
Time (Hrs)
Metro Dr Adj Gauge
Metro Drive Model
88th St Adj Gauge
88th Street Model

FIGURE 15

June 26, 1998
Calibrated

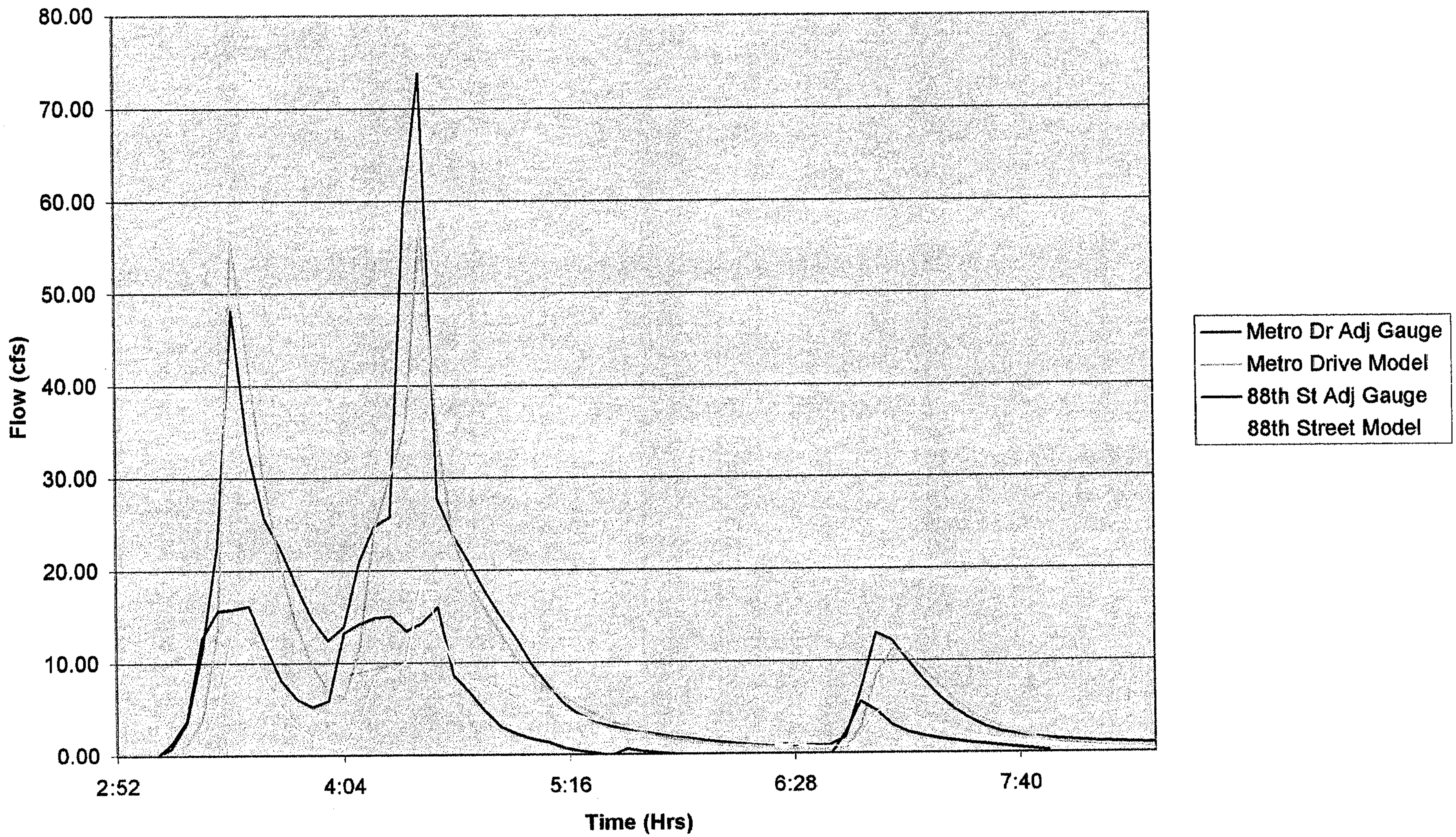


FIGURE 16

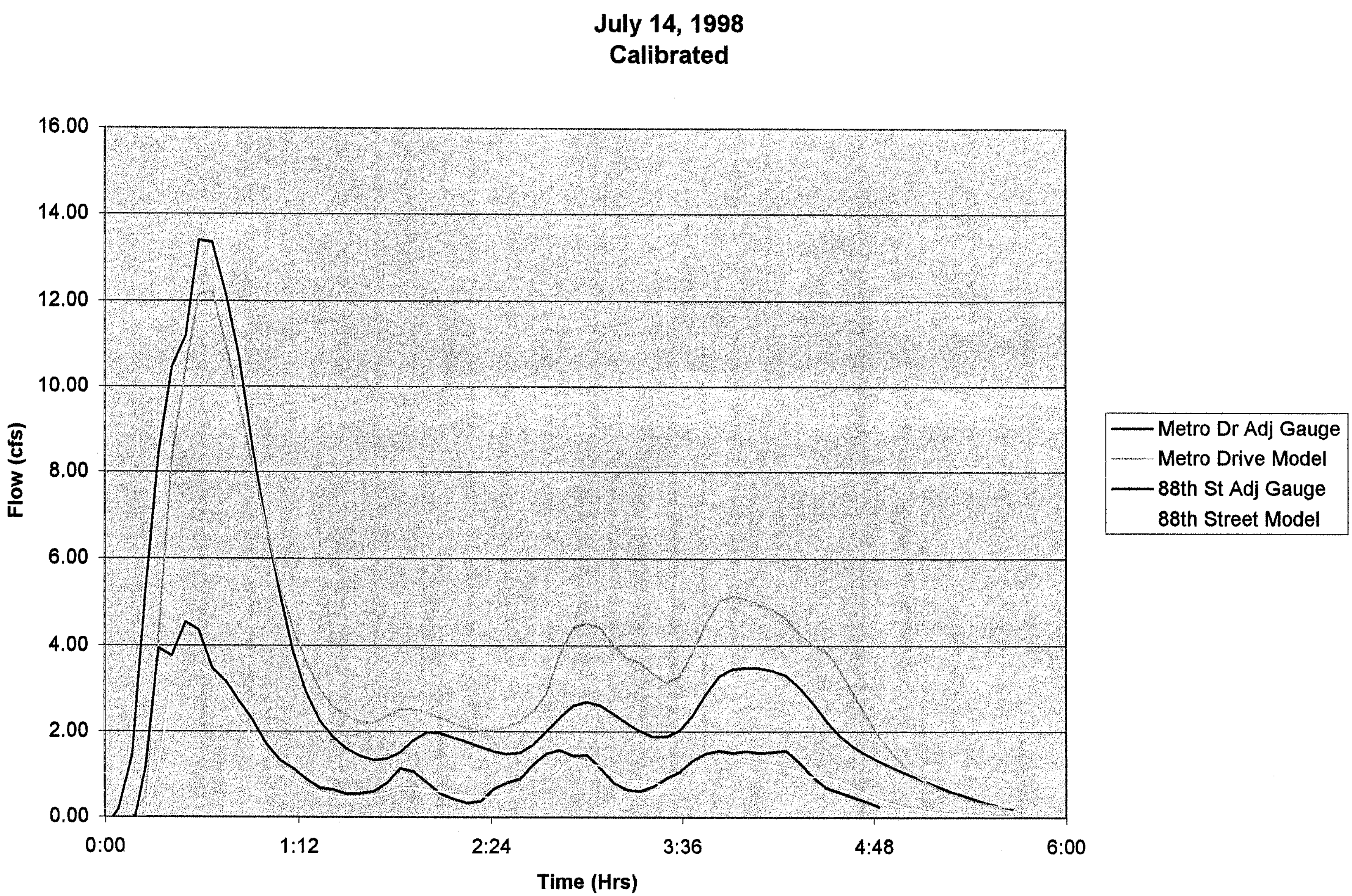

July 14, 1998
Calibrated
16.00
14.00
12.00
10.00
8.00
6.00
4.00
2.00
0.00
Flow (cfs)
0:00
1:12
2:24
3:36
4:48
6:00
Time (Hrs)
Metro Dr Adj Gauge
Metro Drive Model
88th St Adj Gauge
88th Street Model

FIGURE 17

Appendix B

Bloomington Airport South District Storm Water Treatment Feasibility Study, Prepared by SRF Consulting Group, Inc and Montgomery Watson Harza, March 12, 2002

Bloomington
Airport South District

Storm Water Treatment Feasibility Study

Prepared for

City of Bloomington

DRAFT

Prepared by

SRF Consulting Group, Inc
Montgomery Watson Harza

MARCH 12, 2002 – DRAFT FOR REVIEW

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1.0 Introduction/Study Goals

1.1 *Background*

The Airport South District (ASD), bounded by I-494 to the north, TH-77 (Cedar Avenue) to the west, and the Minnesota River to the east and south, is a portion of the City of Bloomington that is expected to see changes in land use over the next twenty years. It's estimated that approximately 272 acres will likely redevelop during this time, i.e., just under 38 percent of the total developable area within the District.

Runoff from the ASD ultimately discharges into Long Meadow Lake through four outfalls. Two of the outfalls, identified as 80th Street and Ceridian outfalls in this study, drain to the Lake through a variety of floodplain wetland complexes. A third outfall discharges to Hogback Pond. It takes runoff from a small area identified as the Adjoining Lands and a portion of Old Shakopee Road, as well as overflow from the area north of Old Shakopee Road, during large storm events via a flow splitting device. These three outfalls account for roughly 8 percent of the drainage area of the four outfalls to Long Meadow Lake, and 27 percent of the ASD. The last outfall, which discharges to Pond C, accounts for 92 percent of the drainage area of the four outfalls to Long Meadow Lake, including a drainage area west of TH 77.

The ASD comprises 29 percent (718 acres) of the total area from Bloomington discharging to the Lake through these four outfalls. The remaining 71 percent, almost 1,800 acres of mostly residential area west of TH 77 also discharges to the Lake via Pond C. This area is not expected to redevelop within the 20-year study period.

Long Meadow Lake lies within the floodplain of the Minnesota River and within the Minnesota Valley National Wildlife Refuge, managed by the U.S. Fish and Wildlife Service (USFWS). The Long Meadow Lake/marsh area is 1,188 acres in size (Table 3-4, Lower Minnesota River Watershed District (LMRWD) Water Management Plan, September, 1999), and is typically inundated for a period of weeks during normal springtime runoff conditions. Refuge staff have identified issues and concerns regarding potential water quality impacts to the Refuge from ASD storm water discharges, including: lack of 'above the bluff' storm water treatment ponds, low efficiency of treatment in Pond C, and past spills from industrial/commercial properties in ASD that had resulted in pollutant transport to Refuge water bodies via the City storm sewer.

1.2 *Study Goals*

City staff initiated this Feasibility Study to take a comprehensive look at the effectiveness of existing storm water treatment facilities and to assess future treatment demands and opportunities, in order to develop a strategy for maintaining and improving water quality level in discharges from ASD to the Refuge/Long Meadow Lake. This Feasibility Study builds on, but looks beyond the 2007 development conditions studied during the ASD Alternative Urban Areawide Review (AUAR) process (conducted in 2001-2002), to address anticipated land use changes through year 2020. This Study also looks in greater detail at alternative strategies for addressing storm water quality, including on-site treatment, regional pond treatment and incorporation of other Best Management Practices (BMPs). In addition to this Feasibility Study, a separate review of commercial/industrial spill prevention practices was conducted to address Refuge concerns related to past spills, and recommendations were made for practices that the City and ASD property-owners could implement to prevent spills from being discharged to the City storm sewer system. Bloomington staff will be following up with commercial and industrial property owners within Airport South and the remainder of the City during 2002-2003 to review their facilities and discuss implementation of spill prevention plans and containment measurements.

The City of Bloomington retained SRF Consulting Group, Inc. and Montgomery Watson Harza to study the following in this Feasibility Study:

1. Evaluate the water quality changes within the ASD watersheds given a list of potential land use changes expected prior to the year 2020. Initial modeling assumptions for 2020 conditions include implementation of NURP-level treatment at all new development/redevelopment sites in ASD. This will allow assessment of whether adequate treatment can be provided by adding on-site ponding only, or if additional regional ponding capacity is also needed.

2. Evaluate the feasibility and cost effectiveness of constructing regional water quality treatment facilities to treat runoff at a number of potential sites within the ASD.

3. Evaluate the effectiveness of implementation of phosphorus reduction and street sweeping BMPs in ASD in reducing pollutant loadings to Long Meadow Lake.

4. Evaluate the impacts of the various proposed facilities and BMPs on the overall ASD loading to Long Meadow Lake.

5. Suggest potential combinations of regional water quality improvement measures that provide an effective, responsible approach to addressing water quality concerns and reducing the pollutant loading for the runoff flowing to Long Meadow Lake through the ASD.

The Feasibility Study's main purpose was to identify and evaluate the feasibility of measures that could be implemented to improve the quality of surface water discharges to Long Meadow Lake from ASD. As with the Airport South District AUAR storm water analyses, this Feasibility Study evaluated the impacts and improvements to water quality of the runoff discharging to the Lake via the four outfalls, and did not directly evaluate the improvements to the ecology to the Lake. This allows for comparison of existing and proposed conditions and potential improvements, but does not attempt to evaluate the impacts to the lake from other potential sources of sediment loading (e.g. from spring flooding).

2.0 Modeling Background/Data Sources

2.1 *Modeling Background*

The ASD is defined by I-494 to the north, TH 77 to the west, and the Minnesota River to the south and east. Previously, models have been developed for this area in order to determine the storm water runoff water quality. An Alternative Urban Areawide Review (AUAR) process was initiated for the ASD by the City of Bloomington to identify and document potential cumulative environmental impacts and infrastructure needs related to anticipated development and redevelopment in the ASD through year 2007. The modeling analyses performed for the AUAR provided a comparison of surface water quality for existing (2000) and post-AUAR (2007) development conditions in the ASD drainage areas to allow for assessment of potential cumulative surface water impacts. The water quality assessment conducted for the AUAR was a continuation of water quality studies conducted previously by Montgomery Watson Harza for the ASD.

The effects of the proposed ASD development were assessed using the P-8 Urban Catchment Model (W. Walker, Jr. 1998) previously developed for the study area in 1998. The model was updated to reflect current and proposed 2007 development conditions. These studies also incorporated modeling from the area west of TH 77 (Cedar Avenue) that drains to Pond C, including the drainage areas for Smith Pond and Wrights Pond. The Feasibility analyses build on the previous AUAR modeling to reflect 2020 development conditions.

2.2 *Background Data*

The following list includes the various sources utilized to provide the information required for the study.

- City of Bloomington storm sewer mapping
- 2-foot GIS contour mapping
- Mall of America Expansion – Met Center Site EIS (2000)
- Bloomington Airport South District Draft AUAR (2001)
- Field walks of each site
- Comprehensive Land Use information
- Discussions with City staff regarding potential land use changes
- Meeting with USFWS and stakeholders

2.3 *Other Resources*

The P-8 Urban Catchment Model vr. 2.2 (1998) was developed in the later 1980's by William Walker, Jr., utilizing information developed during the National Urban Runoff Program (NURP) studies in the later 1970's and early 1980's. P8 is a model to predict the generation and transport of storm water runoff pollutants in an urban watershed. This model is widely used for determining relative impacts of land use changes and best management practices on urban storm water quality.

The model incorporates modeling for the ASD as well as modeling developed for the area west of TH 77 which discharges to Long Meadow Lake through Pond C. Modeling for the area west of highway 77 was provided by City staff and was not changed or manipulated as a part of this study. It was, however, important to include this area in the modeling because it drains to Pond C, which is a device evaluated as a part of this study.

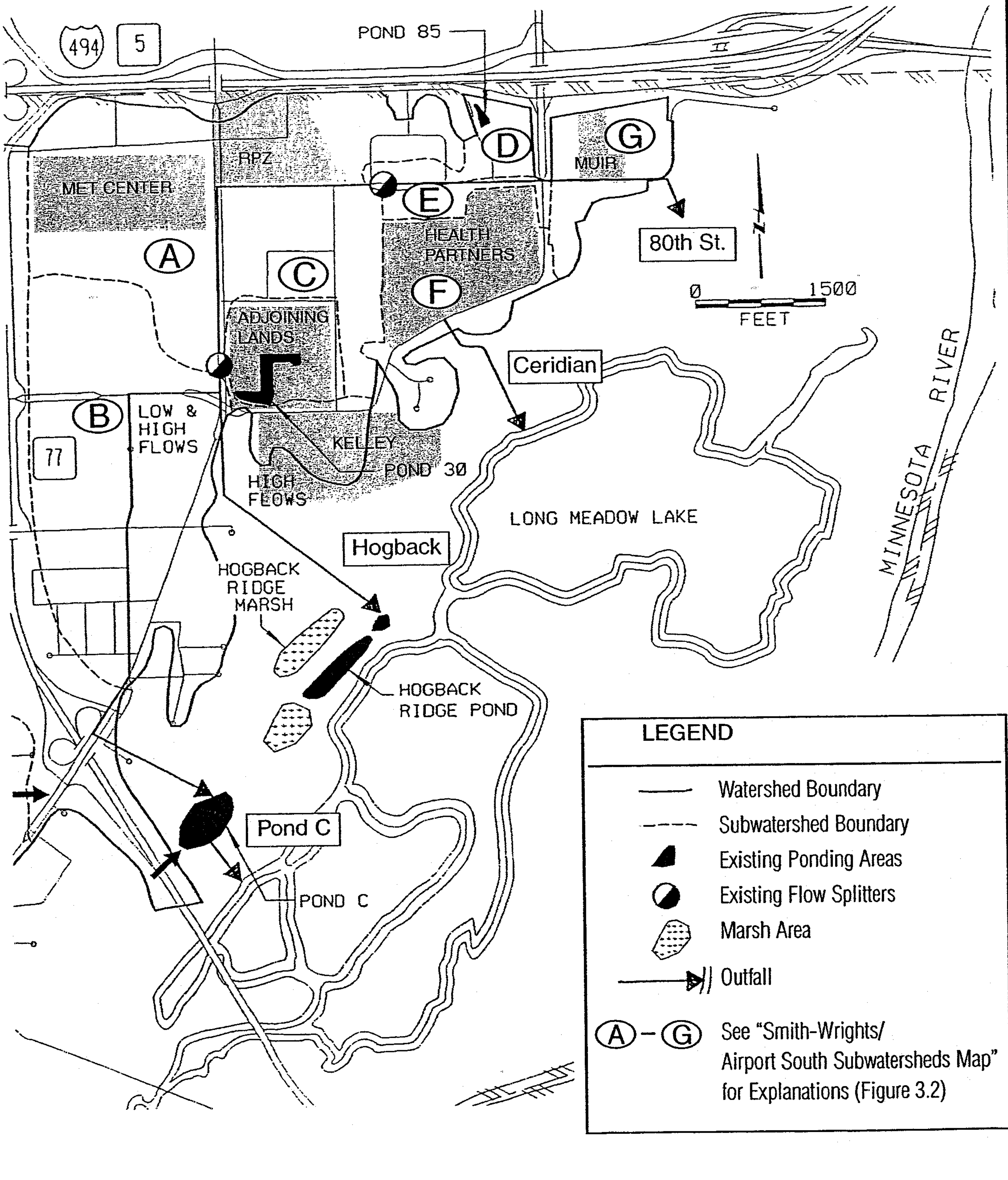
3.0 Existing Conditions

Storm water runoff from the study area currently enters Long Meadow Lake through one of four outfalls, as shown on Figure 3.1 and described in Section 1.1.

Storm water treatment currently takes place within four ponding areas, identified on Figure 3.1. These include two ponding areas below the bluff, Pond C and Hogback Pond, which operate as regional facilities. Pond C (located within Mn/DOT right-of-way and operated by the City) receives runoff from a large drainage area west of TH 77 in addition to runoff from the majority of ASD. Hogback Pond, located within the Minnesota Valley Refuge, receives water from the storm sewer that serves the Adjoining Lands parcel and overflow from the portion of the ASD north of Old Shakopee Road.

Two additional ponds are located within the developed areas of ASD: Pond 85 and Pond 30. Pond 85 serves a small parking area (presently developed as a park-and-ride) near I-494, and Pond 30 serves the existing parking area on the Adjoining Lands parcel located east of the Mall of America.

The individual drainage areas that flow to Long Meadow Lake through the four outfalls are shown in Figure 3.2. Land uses and corresponding acreages within the areas for each outfall are shown in Table 3.1. The ASD comprises 29 percent of the total area draining to Long Meadow Lake through these outfalls, with the remaining 71 percent originating from the area west of TH 77 that drains to Pond C.



SRF Consulting Group, Inc.

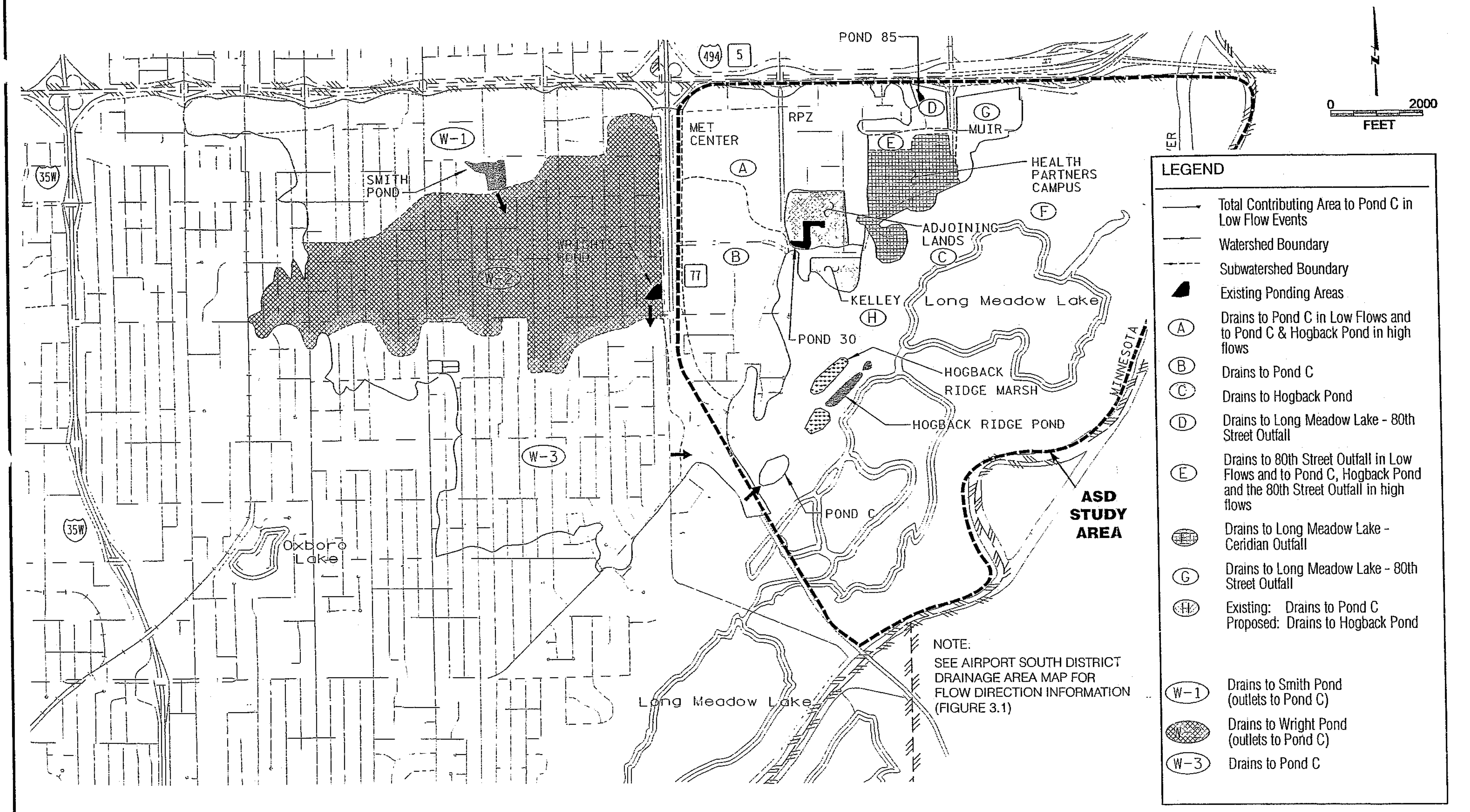
SRF NO. 0983008.7

Airport South District Outfall Location Map

City of Bloomington

FIGURE

3.1



SRF CONSULTING GROUP, INC.

SRF NO. 0983008.7

Smith-Wrights/ Airport South Subwatersheds

City of Bloomington

FIGURE

3.2

Table 3.1
Existing Drainage Area Characteristics

Outfall Location	Total Area (acres)	Impervious Area (acres)	Land Use (Percentage)		
			Commercial/ Industrial	Low density Residential	Office/High Density Residential
80th Street Outfall	76	62	75%	-	25%
Ceridian Outfall	69	44	50%	15%	35%
Hogback Pond Outfall	49	36	100%		
Pond C Outfall – ASD	524	370	70%	20%	10%
Total ASD	718	512	71 %	16 %	13 %
Pond C Outfall – West of TH 77	1,796	800	15%	80%	5%
Totals	2,514	1,312	31%	62%	7%

As Table 3.1 illustrates, the majority of the existing drainage area to Long Meadow Lake is low density residential, primarily located west of TH 77. These residential areas are not expected to redevelop within the 20-year study period. In contrast, the majority of the ASD is higher density development in commercial/industrial and office/high density residential land uses, and 272 of the 718 acres of developed/developable land is anticipated to redevelop within 20 years.

4.0 Anticipated Land Use Changes/Hydrology

4.1 *Changes in Land Use*

The properties within ASD that are anticipated to change land use prior to 2020 are shown in Table 4.1.

The total area which will likely change land use from the existing conditions in 2001 to the year 2020 is 272 acres. This is just under 38 percent of the 718 acres of developable area of ASD, but represents only about 11 percent of the overall drainage area to Long Meadow Lake through the four outfalls.

2001 → 2020 272 acres

Table 4.1
2020 Redevelopment Areas

Location	Total Area (acres)	(2020) Impervious Area/Site (acres)	NURP Pond Surface Area Required (acres)	NURP Pond Permanent Pool Required (acre-feet)
Met Center/ Thunderbird/Marriot [1]	87	54.2	3.9	13.6
Adjoining Lands	34.1	32.6	1.7	5.9
Robert Muir	12.3	11.8	0.7	2.1
Health Partners	44.7	31	1.5 [3]	4.9 [3]
Kelley Property	43 [2]	26.1	1.2	4.0
VTC	16.4	11.2	0.6	1.7
Long Meadow Area	4.5	3.1	0.2	0.5
Total Area	272	170	10	33
Percent of ASD	38%			

[1] Areas/computations include all three sites.

[2] Does not include 17.1 acres that is assumed to be unbuildable.

[3] Pond area and volume includes the total of a pond system taken from a proposed site plan.

For the initial modeling performed for this study it was assumed as these properties were redeveloped, water quality treatment measures were incorporated such that NURP levels of Total Suspended Solids (TSS) removal were attained (70-80 percent), to provide an estimate of the amount of treatment that could be anticipated by incorporating 'above the bluff treatment.' The ponding requirements shown in Table 4.1 describes a pond designed in accordance with the standards set in the City of Bloomington Water Management Plan (2001), which is consistent with standard design practices for the 'Walker' pond method. This method is consistent with the City's Storm water Management Plan as well as the LMRWD plan.

5.0 Potential Water Quality Treatment Sites/Site Evaluation

5.1 *Selection of Water Quality Treatment Measures*

There are many choices for water quality treatment measures for use on small sites, as identified in a number of recent publications (Protecting Water Quality in Urban Areas, MPCA 2000, Minnesota Urban Small Sites BMP Manual, Metropolitan Council, Barr Engineering, 2001). These include a plethora of aboveground and underground systems incorporating a variety of ponds, infiltration measures, and others. The appropriate choice of BMP is often connected with the physical site characteristics, site size and layout, construction methods, target pollutant, short and long-term maintenance requirements and cost. This study assumed the use of on-site ponds (see

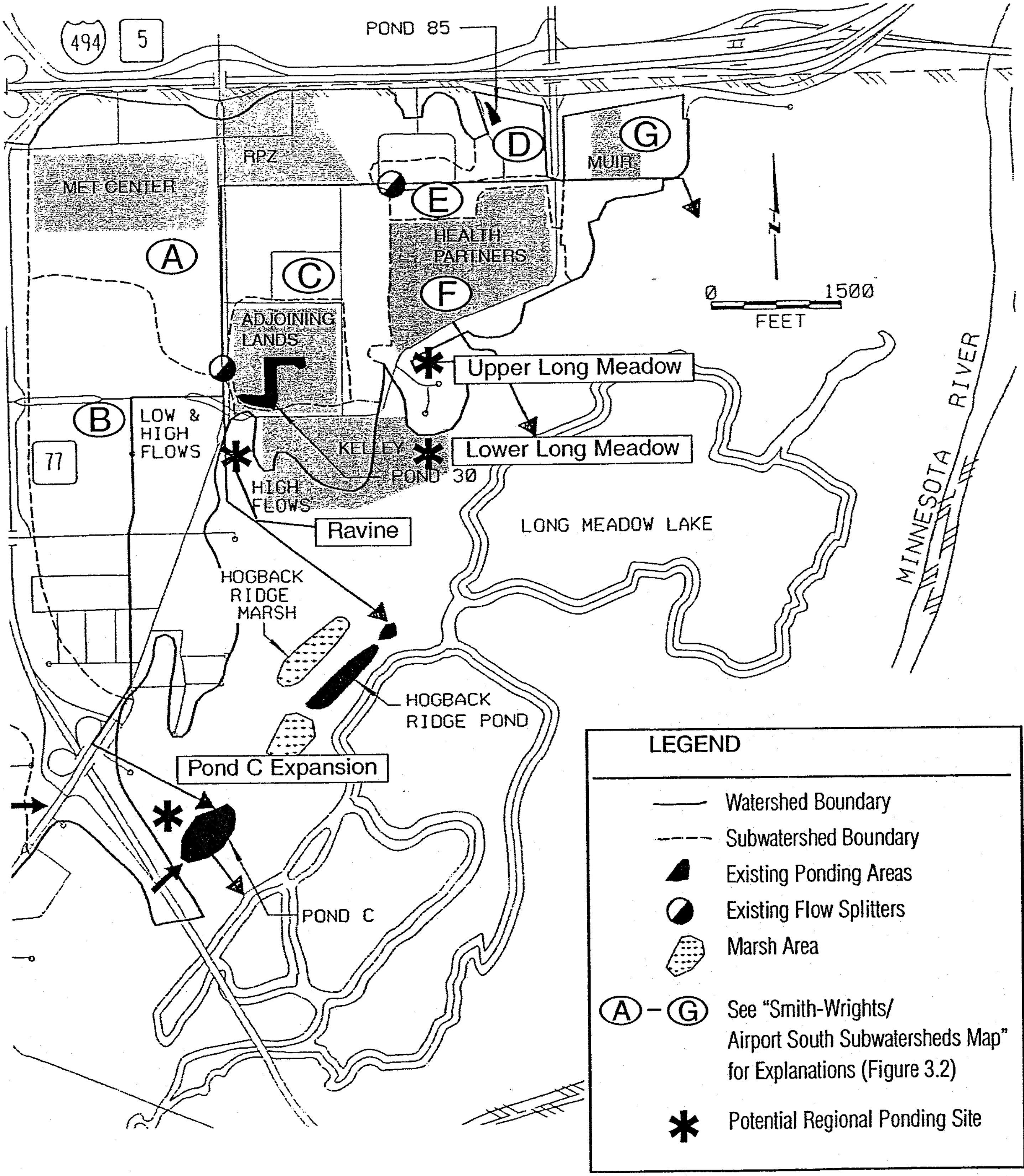
Table 4.1) as the means of providing treatment at each redevelopment site because they generally minimize the site area required for treatment. However, other on-site treatment methods such as infiltration basins could be utilized in addition to or in lieu of ponding to achieve the 70-80 percent target TSS removal levels.

While these various methods may be feasible means for providing on-site treatment at proposed redevelopment sites within the ASD, it is generally accepted that for a large regional treatment system, some type of ponding provides the most cost effective approach to providing a high level of pollutant removal. Wet detention ponds are relatively easy to construct, require a low level of maintenance, and can provide a high level of treatment of both soluble and particulate pollutants when sized properly. Therefore, this study analyzed the feasibility of using wet detention ponding for regional water quality treatment, in addition to studying the effectiveness of providing on-site ponding within the proposed redevelopment sites.

5.2 *Potential Regional Ponding Sites*

The ASD was reviewed from a hydrologic perspective to identify potential regional ponding sites to supplement and/or substitute for on-site ponding, as needed. The primary consideration in this process was the configuration of the existing conveyance systems and contributing drainage areas that discharge to Long Meadow Lake. Given the depth of the existing storm sewer (up to 15 feet below the ground surface) and its relationship to contributing drainage areas, as well as the physical characteristics of the ASD area (including bluff/ravine areas and intensively developed areas), the list of potential regional pond locations was narrowed to the sites shown on Figure 5.1.

Each of the sites identified was evaluated based on criteria in three main categories: Design/Effectiveness, Physical Feasibility, and Long Term Operation Issues. The criteria are listed in Table 5.1. Design criteria included initial analyses to determine the pond size required to treat the contributing drainage area (used as the basis for the physical feasibility evaluation) and, later, analysis of construction cost and treatment efficiency, measured in cost per pound of TSS removed. Physical feasibility included assessment of issues related to 'constructability' of a pond in each potential location (i.e., is there enough land available, are there environmental constraints, are there soil or topographic constraints, etc.) Long-term operation issues are not critical to assessing the physical feasibility, but are factors that should be considered in overall operation of a pond over time.



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Airport South District
Potential Regional Ponding Sites

City of Bloomington

FIGURE

5.1

Table 5.1
Pond Feasibility Criteria

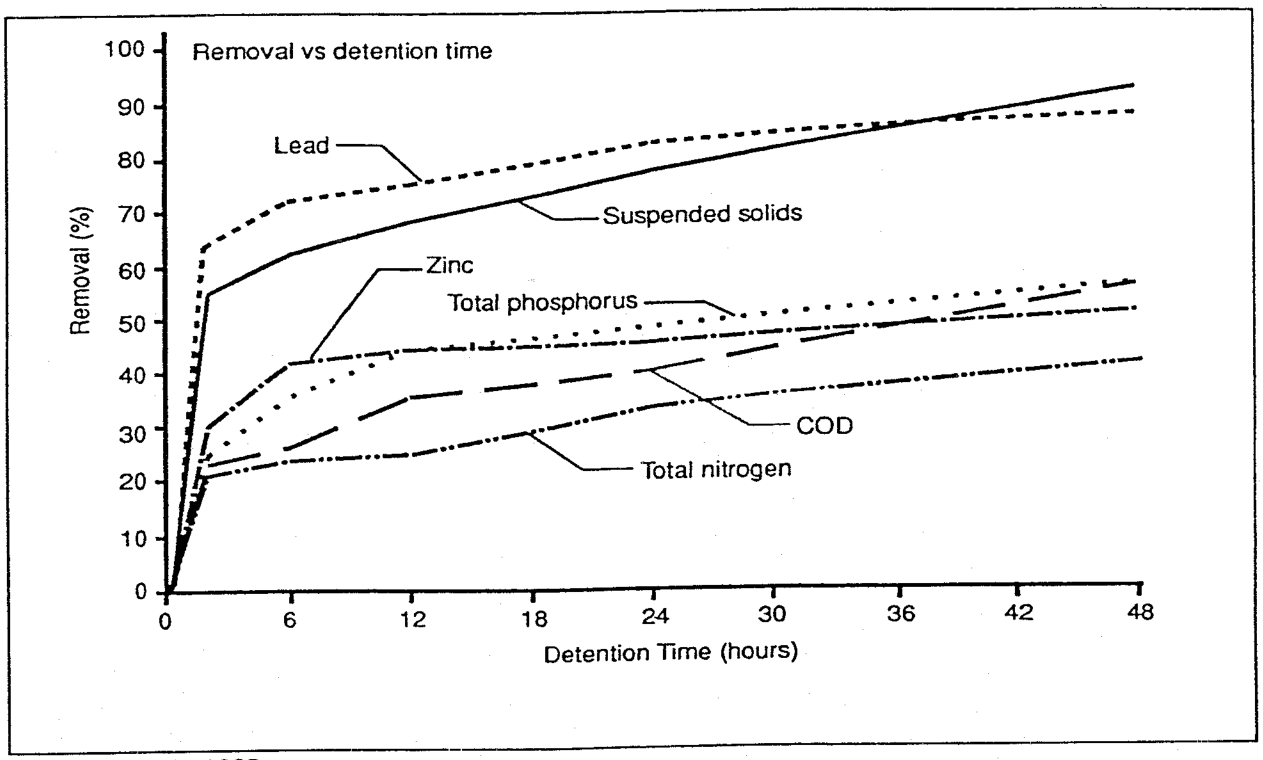
Design/Effectiveness
• Contributing Drainage Area (based on existing trunk storm sewer)
• Proposed Normal Pool Depth/Surface Area
• Potential Dead Pool/Active Pool Storage
• Allowable Flood Elevation
• Pollutant Removal (lbs./year)
• Estimated Construction Costs
Physical Feasibility
• Physical Accessibility
• Land Ownership
• Proximity to Existing Storm Sewer (horizontal location and depth)
• Soil Conditions
• Constructability
• Wetland Impacts
• Permitting Requirements
• Cultural Resources
Long Term Operation Issues
• Maintenance Access
• Aesthetics
• Maintenance Issues

5.3 *Pond Treatment Effectiveness Criteria*

The current treatment standards in the Twin Cities metro area include providing ponding surface areas that are approximately 1 percent of the drainage area, given a permanent pool volume of runoff from the 2.5-inch storm event (Walker, 1987). Additional guidance in planning regional treatment is provided by Robert Pitt, who has studied pond sizing in terms of small storm events and found the ratio to vary between 0.8 for residential areas to 3.0 for totally paved areas (Pitt, 1998)

Storm water treatment devices designed according to NURP standards (MPCA criteria) will have long-term average phosphorus removal efficiencies of 47 percent to 68 percent for the Twin Cities area (W. Walker Jr., 1987). Detention ponds are designed to remove pollutants from surface waters as a result of physical settling and are most effective for controlling those pollutants typically associated with sediment particles, including lead, phosphorus and zinc. These pollutants attached to sediment particles exhibit settling characteristics similar to those of sediment as illustrated in Figure 5.2. Consequently, if removal efficiencies are reached for TSS, then appropriate removal efficiencies will typically be reached for the other constituents of concern. Therefore, TSS was used as an indicator of pond effectiveness.

Figure 5.2
Results of Settling Column Study on Urban Runoff



Source: OWML, 1983

Once the initial pond sizing was completed, a review of the physical feasibility of each potential pond location was conducted. This review indicated that there are physical barriers to providing regional ponding at some of the sites originally identified. First, existing outfalls at 80th Street and Ceridian have very limited drainage areas, and very limited undeveloped space for pond construction above the bluff. The only opportunity for water quality treatment exists within the redevelopment sites (i.e., Long Meadow Upper Pond). Second, the existing storm sewer system is typically quite deep to Hogback Pond or Pond C, which creates very deep regional ponding that require considerable property within areas on the bluff. Third, the ponding sites available deep enough to serve the storm sewer system (Ravine pond/Long Meadow Lower) have a variety of technical issues, including sidehill seeps that would change the effectiveness of settling in the basin, or resulted in other potential impacts (archeological, heavily wooded areas, etc.)

The physical feasibility matrices are found in Table A.1 in Appendix A. The matrices provide details of the factors evaluated for each of the regional ponding alternatives studied, and resulted in the conclusion that regional ponds at the 80th Street outfall and the Long Meadow Lower Pond sites were not feasible due to physical land constraints.

The remainder of the ponding sites were carried through the modeling processes to evaluate their overall effectiveness as a regional facility. These represent regional sites that would provide supplemental treatment for two of the four outfalls, and include those shown in Table 5.2.

Table 5.2
Regional Pond Locations

Pond Location	Description	Outfall Location
Pond C Expansion	Expand Pond C to the north within existing Mn/DOT right-of-way and physical constraints. Pond C is expanded by about 2.5 times in surface area and roughly 5 times in dead pool volume.	Pond C Outfall
Ravine Pond	Pond would be located in the ravine area along the access road to the Refuge. Direct drainage area would be from the Adjoining Lands and the outfall from Kelley Farms, which is currently directed to Hogback Pond.	Hogback Pond Outfall
West 77 pond	Pond sized to provide treatment for the untreated area currently flowing to Pond C from area W-3.	Pond C Outfall

6.0 Water Quality Modeling

6.1 *Methodology*

The water quality studies performed for the Met Center EIS and the ASD AUAR provide the backdrop for the modeling effort contained within this study. The P-8 model developed for these documents was expanded to evaluate the regional facilities.

As with the ASD AUAR, the 2.5-inch type II single event storm was used to evaluate the system. This is consistent with the design required for NURP ponds that would be found within the various redevelopment sites. The P-8 modeling performed for the Met Center Site EIS (May 18, 2000) evaluated the impact and effectiveness of three rainfall scenarios of the redevelopment and mitigation respectively. In all cases the large, single event resulted in removal efficiencies 15-25 percent lower than for the normal rainfall year. Since the modeling was completed for a much larger design storm this analysis provides a conservative approach for the evaluation of the devices on an annual basis.

The average annual storm in Minneapolis is approximately 0.34 inches in magnitude. It is likely that the devices will actually perform much better under the majority of storm events likely to occur. The particle class used for the analysis was the 50th percentile class derived from the National Urban Runoff Program. The model also includes data for the area west of 77 provided by City staff as it affects the functioning of Pond C.

6.1.1 Storm Water Treatment Scenarios Modeling

A number of scenarios were assembled to evaluate the effectiveness of the overall system efficiency (i.e., the combined effect of all assumed treatment facilities on the discharge to Long Meadow Lake), as well as the individual components. The baseline condition included NURP ponding within all of the redevelopment areas as a means of assessing the ability of treatment at new development sites to provide adequate treatment (i.e. without additional regional ponding capacity). The model evaluates the treatment components both individually and as a system, which provides the opportunity to observe how each component contributes to the overall protection of Long Meadow Lake.

The four potential regional pond facilities listed in Table 5.2 were incorporated – along with other ponding/BMP strategies – in combinations to create eight treatment scenarios (see Table 6.1) that were modeled for overall effectiveness in TSS removal. Most of the scenarios focus on evaluation of options that address the area draining to the Pond C outfall, given that Pond C provides treatment for the majority of the drainage area. It should also be noted that although the 80th Street and Ceridian outfalls do not have regional ponding facilities, onsite NURP ponds are included on redeveloped properties within these subwatersheds for the Baseline condition.

Table 6.1
2020 Model Scenarios

1 2020 Baseline Conditions	NURP-level treatment ponds constructed on all new development properties. This model is the baseline for the remaining 2020 model scenarios.
2 Pond C Expansion	Maximized the expansion of Pond C to the north within existing Mn/DOT right-of-way and physical constraints. Pond C is expanded by about 2.5 times in surface area and roughly 5 times in dead pool volume.
3 Ravine Pond	Routed overflow from the splitter through a pond in the ravine area. Direct drainage area is from the Adjoining Lands and the outfall from Kelley property, which currently flows to Hogback Pond.
4 Wrights Lake and Smith Pond bypass	Bypassed the Wrights Lake and Smith Pond outlets directly to Long Meadow Lake. Model intended to test effectiveness of removing hydrologic loading from Pond C.
5 West 77 pond	Routed watershed W-3 (currently untreated) through a NURP pond and then into Pond C.
6 West 77 area through a NURP pond with Pond C expansion	Routed all of the area west of 77 through the W77 pond, including the outlet of Wrights Lake and watershed W-3. Outlets directly to Long Meadow Lake, reducing hydraulic loading on Pond C. Pond C expansion treats ASD water only.
7 Wrights Lake NURP pond with Pond C expansion	Routed the outlet of Wrights Lake through a W77 pond, then directly to Long Meadow Lake. Area W-3 flows to Pond C.
8 2020 combo of TP ban, street sweeping, Pond C expansion	This model is intended to demonstrate the effectiveness of additional housekeeping measures, compared to Scenario 2.

6.2 *Analysis Results*

6.2.1 Existing Conditions

The existing systems include storm water treatment at Hogback Pond and Pond C, as well as two ponds currently within the ASD (Pond 30 and Pond 85). The model results for these ponds in the current condition are shown in Table 6.2. The results show that the Smith Pond/Wrights Lake and Pond C treatment systems are below the 70 percent removal threshold.

TABLE 6.2
DEVICE REMOVAL EFFICIENCIES FOR TOTAL SUSPENDED SOLIDS
2000 CONDITIONS

Pond	Runoff Volume (ac-ft)	TSS input load (lb)	TSS output load (lb)	TSS removed (lb)	Percent (%) Reduction TSS	Percent (%) Reduction TP
Existing Treatment Ponds (Regional and Site Specific)						
Pond 30	4.9	615	121	494	80	35
Pond 85	2.1	318	98	219	69	29
Hogback Pond	34	3,816	722	3,077	80	34
Pond C	220	19,027	10,176	8,852	47	12
Smith Pond	63.2	7,961	2,890	5,071	64	21
Wrights Lake	119	10,126	6,017	4,109	41	9

6.2.2 On-Site Development Ponds

The effectiveness of on-site treatment devices within the redevelopment sites assumed for modeling Scenarios 1-8 are shown in Table 6.3. Each new device provides a removal efficiency of 70-80 percent, at or above minimum NURP requirements for a Type II storm event.

TABLE 6.3
DEVICE REMOVAL EFFICIENCIES FOR TOTAL SUSPENDED SOLIDS REDEVELOPMENT SITES

Pond	Runoff Volume (ac-ft)	TSS input load (lb)	TSS output load (lb)	TSS removed (lb)	Percent (%) Reduction TSS	Percent (%) Reduction TP
Airport South District 2020 Redevelopment On-Site Ponds						
Met Center/ Thunderbird/ Marriott	15	1,850	444	1,406	76	31
Adjoining Lands	5.3	666	140	525	79	34
Health Partners	7.0	1,142	328	815	71	33
Muir Pond	2.3	318	80	238	75	33
Kelley Ponds	5.5	691	202	490	71	28
VTC	2.2	280	72	208	74	30
LMA	0.6	78	20	58	74	30

6.2.3 Results of Regional Ponding Combinations

The scenarios described in Table 6.1 were modeled to evaluate the effectiveness of each scenario combination on the total loading from the four outfalls to Long Meadow Lake. Results of these model runs are shown in Table 6.4.

TABLE 6.4
DEVICE REMOVAL EFFICIENCIES FOR TOTAL SUSPENDED SOLIDS (TSS) 2020 Scenarios

Treatment Scenario	Runoff Volume (ac-ft)	TSS input load (lb)	TSS output load (lb)	TSS removed (lb)	Percent (%) Reduction TSS	Percent (%) Reduction TP
1 2020 Baseline	296	38,097	13,363	24,737	65	24
2 Pond C Expansion	296	38,097	11,117	26,981	71	29
3 Ravine Pond	296	38,097	13,161	24,936	65	24
4 Wrights/Smith Lake bypass	296	38,097	14,369	23,728	62	22
5 West 77 pond	296	38,097	10,944	27,152	71	30
6 West 77 area through NURP pond w/ Pond C expansion	296	38,097	8,310	29,787	78	35
7 Smith/Wrights Lake NURP pond w/Pond C expansion	296	38,097	8,834	29,263	77	34
8 2020 Combo-w/Pond C Expansion	296	38,097	11,176	26,905	71	28

A review of the results provides the following comments.

- Ponding scenario 1 includes ponding within all of the redevelopment areas. The addition of NURP ponding within all of the development areas expected to redevelop within the next twenty years does not bring the overall system removal efficiency to within the 70 percent minimum TSS removal NURP treatment goal (only 65 percent TSS removal is achieved in Scenario 1).

- Expanding Pond C, shown in Scenario 2, in conjunction with the development ponding, removes 71 percent of the TSS load to Long Meadow Lake, within the range of NURP guidelines.

- Construction of Ravine Pond (Scenario 3) does not contribute appreciably to TSS removal in the system (compare to Scenario 1).

- Scenario 4 was included to evaluate the impact of removing hydraulic loading from Pond C by routing Smith/Wrights (treated) discharges directly to Long Meadow Lake, not through Pond C. While Pond C performs at higher removal rate, the overall system still operates at a lower efficiency than Scenario 1 because Smith/Wrights do not treat to NURP standards and Pond C still does not meet NURP standards.

- Scenario 5 – construction of a pond to treat Watershed W-3 (currently untreated see Figure 3.2) prior to discharge to Pond C – improves system performance (compare to Scenario 1) to meet minimum NURP standards, but it is not substantially better than construction of Pond C expansion alone (Scenario 2).

- Pond C expansion combined with construction of additional ponding west of TH 77 (Scenario 6) provides additional system improvement (77 percent removal), compared to Pond C expansion alone.

- The model does not reflect any appreciable difference instituting the street sweeping/phosphorus ban measures in Scenario 8 (compare to Scenario 2).

6.2.4 Analysis of Scenarios Without Redevelopment Treatment at Select Properties

Comments from MAC staff on the Airport South District Draft AUAR document expressed concern about locating storm water ponding above the bluff, where it might encourage waterfowl thus increasing the risk of bird/aircraft conflicts. To address this concern, and also as a means of evaluating the relative effectiveness of on-site treatment compared to regional treatment, two additional sub-scenarios were analyzed:

A. Assume no ponding within the runway safety zone, i.e., removing ponding from Kelley and Adjoining Lands.

B. Removal of ponding within the Met Center, Thunderbird and Marriott sites in addition to the Kelley and Adjoining Lands parcels.

The model alternatives re-run with these two sub-alternatives included Scenarios 1, 2, and 6. Results are shown in Table 6.5.

TABLE 6.5
DEVICE REMOVAL EFFICIENCIES FOR TOTAL SUSPENDED SOLIDS (TSS)
2020 SCENARIOS WITHOUT NURP PONDING WITHIN SPECIFIC DEVELOPMENT AREAS

Ponding Scenario	Runoff Volume (ac-ft)	TSS input load (lb)	TSS output load (lb)	TSS removed (lb)	Percent (%) Reduction TSS	Percent (%) Reduction TP
2020 SCENARIOS WITHOUT NURP PONDING AT ADJOINING LANDS/KELLEY						
1A 2020 Baseline	296	38,097	13,499	24,598	65	24
2A Pond C Expansion	296	38,097	11,254	26,843	70	28
6A West 77 through W77 Pond w/Pond C expansion	296	38,097	8,448	29,649	78	34
2020 SCENARIOS WITHOUT NURP PONDING AT Adjoining Lands/Kelley/Met Center/Thunderbird/Marriott						
1B 2020 Baseline	296	38,097	13,780	24,317	64	23
2B Pond C Expansion	296	38,097	11,498	26,599	70	28
6B West 77 through W77 Pond w/Pond C expansion	296	38,097	8,651	29,446	77	34

When compared to the original runs of Scenarios 1, 2 and 6 in Table 6.3, these new scenarios indicate that the TSS and phosphorus removal efficiency of the system has very little change when ponding is removed from the specified areas. The ponding within the Adjoining Lands and Kelley parcels, noted as ponding scenario alternative A, resulted in an additional 137 pounds of TSS removed per year for all scenarios. Also removing ponding from the Met Center, Thunderbird and Marriott sites results in an increase in loading of 340 to 420 pounds per year compared to the original scenarios 1, 2 and 6. In each sub-alternative (2A/2B and 6A/6B), the addition of a regional facility (Pond C expansion or W77 pond) provides an overall system that falls within the NURP guidelines, even if the on-site ponds are not constructed at these sites.

6.2.5 Comparison of Ponding Approaches

Using the Design/Effectiveness criteria established in Section 5, each of the 8 model scenarios was evaluated based on effectiveness and cost. The results are shown Table 6.6.

TABLE 6.6
MODEL SCENARIO COMPARISON

Ponding Scenario	Description	Additional TSS removed (lb) [1]	Approximate Costs [2]	Cost Per Pound of TSS Removed [3]	Percent Reduction TSS (%)
	2000 Existing conditions	-	-	-	62
2020 SCENARIOS WITH NURP PONDING AT ALL DEVELOPMENT SITES					
1	2020 Baseline	817	$14.7 million	$1,200	65
2	Pond C Expansion	3,061	$19.8 million	$431	71
3	Ravine Pond	1,016	$16 million	$1,050	65
4	Wrights Lake /Smith Pond bypass	-192	$14.7 million	N/A	62
5	West 77 pond	3,232	$29 million	$598	71
6	West 77 area through NURP pond w/ Pond C expansion	5,867	$35 million	$398	78
7	Smith/Wrights Lake NURP pond w/Pond C expansion	5,343	$35 million	$437	77
8	2020 Combo- w/Pond C Expansion	2,985	$19.8 million	$442	71
2020 SCENARIOS WITHOUT NURP PONDING AT ADJOINING LANDS/KELLEY					
1A	2020 Baseline	678	$10.4 million	$1,023	65
2A	Pond C Expansion	2,923	$15.4 million	$351	70
6A	West 77 through W77 Pond w/ Pond C expansion	5,729	29.5 million	$343	78
2020 SCENARIOS WITHOUT NURP PONDING AT Adjoining Lands/Kelley/Met Center/Thunderbird/Marriott					
1B	2020 Baseline	397	$4.8 million	$806	64
2B	Pond C Expansion	2,679	$9.8 million	$244	70
6B	West 77 through W77 Pond w/ Pond C expansion	5,526	$24 million	$290	77

[1] Additional TSS removal is computed by normalizing the existing 2000 drainage area to the 2020 condition, then subtracting the TSS removed from each model scenario.

[2] Costs include applicable construction and land costs. The 2020 Baseline costs included in the scenario costs. The Ravine Pond and Pond C expansion are both within existing public right-of-way, and therefore do not include land cost. Land costs for the development ponds assumed at $25/sf.

[3] Per pound costs are computed based on a 15-year design life.

A number of observations can be drawn from the data in Table 6.6.

- Regional ponding provides the most cost effective removal of TSS.

- The expansion of the Pond C is a common link between all of the scenarios that provide the lowest cost per pound of TSS removed (scenarios 2,6,7, and 8).

- While providing treatment for the untreated watershed W-3 (scenario 5) accomplishes similar levels of treatment as expanding pond C, it does so at almost 1.5 times the cost.

- Adding a pond west of TH 77 is cost effective in terms of treatment, when the pond provides additional treatment for the Wrights Lake outfall in addition to watershed W-3. Scenario 6 (Pond expansion plus a pond west of TH 77) has the lowest cost per pound of TSS removed, however, it costs nearly twice as much as the next closest alternative (Scenario 2 – Pond C expansion), which also meets the minimum treatment criteria (70% TSS removal).

- The 'B' alternative provides the least cost per pound of removal for all of the scenarios studied, due to elimination of the need to pay a high cost for land needed to construct ponding in Scenarios 2 and 6.

7.0 Conclusions

A number of general conclusions can be drawn from reviewing the results of the modeling analyses and the other implementation considerations.

- <u>The majority of untreated runoff from above the bluff comes from the drainage area west of TH 77.</u> 71 percent of the total drainage area within the study area (1,796 acres) originates west of TH 77. Of this area, 606 acres discharge untreated to Pond C, while the Smith-Wrights Ponding system reduces the TSS loading by only 40 percent prior to discharging to Pond C.

- <u>Pond Scenario 2, the expansion of Pond C, provides a level of treatment within the range of the NURP guidelines for total loading from the area.</u> The expansion of Pond C, in conjunction with the NURP level of treatment within the 2020 redevelopment areas, results in TSS reduction of 71 percent. This level of treatment is within the range found in typical NURP basin design.

- <u>Pond C gains additional benefits from a separate outfall to Long Meadow Lake for the area west of TH 77.</u> Model Scenario #6 provides treatment of all of the storm water west of TH 77, including area W-3, and provides a separate outfall to Long Meadow Lake. With an expansion to Pond C, the normal pool surface area is approximately 1.5 percent of the contributing drainage area, which is within the guidelines for effective pond treatment for the mix of residential and commercial area.

- The greatest reduction in overall loading to Long Meadow Lake occurs with additional treatment of storm water discharging from Wrights Lake/Smith Pond to pond W77 with the expansion of Pond C. Pond W77, in addition to Smith Pond and Wrights Pond, provides pond surface area equaling roughly 1.7 percent of the upstream drainage area. The modeling results, 77-78 percent overall removal efficiency, are within the range of treatment reported in the various research documents available on pond effectiveness.

- The addition of on-site NURP level treatment (70-80 percent reduction in TSS) at redevelopment sites provides minor reductions in overall system TSS removal. The study scenarios include a comparison of redevelopment with and without NURP level treatment within each site (Sub-alternatives A and B). The 70-80 percent reduction in TSS at the Adjoining Lands and Kelley sites result in a removal of 1,015 pounds of TSS for the individual sites. However, the resulting reduction in loading to the Long Meadow Lake is 139 pounds, or about 14 percent of that removed at the sites.

- Regional ponding appears to provide the most cost effective approach to overall TSS and TP removal. A comparison of estimated cost per pound of TSS removal for the various treatment scenarios studied indicates that Sub-alternatives A and B (that assume fewer on-site ponding areas, compared to Scenarios 1, 2 and 6) result in lower cost per pound removal of TSS with only minor changes in efficiency. For example, the cost of TSS removal for Scenario 2 (Pond C expansion) drops 18 percent from Scenario 2 to Scenario 2A, with a less than 1 percent drop in removal efficiency. The cost decrease between Scenario 2B vs. Scenario 2 is 43 percent, also with only a 1 percent decrease in efficiency of TSS removal.

8.0 Recommendations

8.1 *Recommendations*

The following recommendations are based on the conclusions stated above.

- Pursue design and permitting of the expansion of Pond C.

  The Pond C expansion can provide a substantial benefit for the water quality of storm water entering Long Meadow Lake from the study area, as it provides treatment of 92 percent of the total ASD study area. It also brings overall removal for the ASD and the area west of TH 77 within the NURP TSS removal gridlines (70 percent removal).

  Construction access, right-of-way, and existing soils data will be critical to the design and construction. Soil borings for the skimmer structure installed in 2000 indicated the excavated soils to be granular in nature, which may reduce the construction costs.

- Pursue ponding locations for the drainage area west of TH 77, and/or the expansion of Wrights Lake, if redevelopment occurs in this area in the future.

  The cost of purchasing right-of-way for a regional ponding facility west of TH 77 makes a proposed regional pond cost prohibitive. While this area is not expected to redevelop in the near term, a pond providing additional treatment of this area would provide a substantial increase in sediment and phosphorus removal.

- If no regional ponding facilities are available for a subwatershed (i.e., 80th Street and Ceridian outfall areas), then on-site treatment ponds (or equivalent treatment facilities) should be incorporated into all new development/redevelopment projects within the subwatershed.

- Incorporate rate control and primary treatment measures as a minimum treatment at all redevelopment areas within subwatersheds served by regional ponds.

  While the analysis reveals that on-site ponding within the proposed re-development areas has a relatively small impact on overall water quality (compared to the regional ponding improvements identified above), long term maintenance costs of the regional facilities would be lessened by removing sediment nearer the source through the use of primary treatment measures such as grit chambers and 'floatables' removal devices. While the sediment that is easily removed does not carry the bulk of the pollutant loading, it does create the largest volume of material. Removing a large percent of this material at the source may double the expected maintenance schedule for the ponding areas, as well as allow them to operate at higher efficiencies for a longer period of time.

- Encourage low impact development (LID) management practices be incorporated for treatment in redevelopment areas where appropriate.

  LID practices include a variety of methods to provide on-site infiltration. Appropriate LID practices may include infiltration trenches, rain gardens, green roofs, treatment trains, and many other practices that promote infiltration such as wetland restoration. While it is difficult to establish these practices in the larger commercial and industrial settings found within the ASD, it may be possible to incorporate them into an office campus setting, or the redevelopment of small sites.

- To reduce the potential for pollutant overloading from accidental spills from commercial and industrial properties within ASD, City staff should continue to work with commercial/industrial property owners within ASD and the remainder of the City in implementation of spill prevention plans and containment procedures.

8.2 *Agency Coordination/Partnerships*

We anticipate the following coordination will be required to implement the recommendations.

- Expansion of Pond C takes place within existing Mn/DOT right-of-way, and will likely require coordination with them for the proposed expansion.
- USFWS for potential construction and maintenance access to expand Pond C.
- Business owner's cooperation with City staff will be required for spill prevention implementation.

Appendix A

Pond Feasibility Matrices

Table A.1
Pond Feasibility Matrices

LOCATION	CRITERIA					
	DESIGN PARAMETERS/EFFECTIVENESS					
	Contributing Drainage Area	**Proposed Normal Pool Depth/Surface Area**	**Potential Dead Pool/Active Pool Storage**	**Allowable Flood Elevation**	**Pollutant Removal (lbs./year)[1]**	**Estimated Construction Cost (incl. r/w costs)**
Pond C Watershed						
Pond C Expansion	Total drainage area approx. 2,320 acres. Airport South Area approx. 524 acres.	8' average depth/ 12.7 ac. (total area).	58 ac.-ft. (total w/existing pond is 93 ac.-ft.)/ 100 ac.-ft.	722 ft. (with ~ 1.5 ft. of freeboard) at existing maintenance road elevation.	2,244	$5.0 million [3]
West of TH 77	Approx. 605 acres.	4 ft./21 ac.	84 ac.-ft./ 76 ac.	Depends on site chosen.	2,415	$14.1 million [4]
Hogback Pond Watershed						
Ravine Pond	Approx. 37 acres (considers low flow pipe routing only).	5-15 ft./3.0 ac.	13.8 ac.-ft./ 56 ac.-ft.	Approx. 788 ft.	199	$1.4 million [5]
Muir Watershed						
80th Street Outfall Pond	Approx. 75 acres.		9.5 ac.-ft. reqd.		n/a	
Olnick/Ceridian/VTC Watershed						
Long Meadow Pond - Upper	Approx. 77 acres.	4 ft./1 ac.		Depends on site chosen.	481	$1.6 million [4]
Long Meadow Pond - Lower	Approx. 77 acres.	6 ft./0.5 ac	2.0 to 2.25 ac.-ft./ 1.2 ac.-ft.	712 ft. (provides one foot of freeboard).	366	$1.2 million [4]
Typical Development On-Site NURP Basin						
Health Partners	45 acres/70 percent impervious	4 ft average depth/1.5 ac.	4.9 ac-ft	-	815	$2.2 million [4]

[1] Pollutant removal percentages based on overall system improvements using the 2020 land uses and 2020 baseline conditions (Model Scenario 1).
[3] Costs include excavation/grading; removal/relocation of existing pipes and trash removal structure: restoration.
[4] Costs include excavation and right-of-way.
[5] Costs include dam; lining (seepage control); slope stability measures; trash structure relocation; construction access; restoration.

Table A.1
Pond Feasibility Matrices (cont.)

LOCATION	CRITERIA							
	PHYSICAL FEASIBILITY							
	Physical Accessibility	**Land Ownership**	**Proximity of Existing Storm Sewer**	**Soil Conditions**	**Constructability**	**Wetland Impacts**	**Permitting Requirements**	**Cultural Resources**
Pond C Watershed								
Pond C	Access via USFWS service roads	Mn/DOT right-of-way	2 existing inlets to pond (Mn/DOT & municipal pipe); relocate outlets to fit expanded pond	Some organics/sand and sandy loams	Significant excavation/access onto TH 77 or through USFWS roads	Minimal if any	Mn/DOT permit	unknown - likely reviewed with original construction of Pond C by Mn/DOT
West of TH 77	Along existing residential streets	Approx 75 private residential parcels	Will require evaluation once a site is identified	-	-	-	-	-
Hogback Pond Watershed								
Ravine Pond	Existing, steep side slopes and wooded areas in ravine limit accessibility to pond for construction	Need to identify property owners and difficulty of purchasing land	Existing 72" storm sewer flows through the pond area	Sandy soils in area of ravine will affect seepage rates and slope stability of side slopes of proposed pond	Lack of existing access routes and steep side slopes contribute to constructability concerns			Bluffs adjacent to ravine are "high probability" sites. Ravine itself is not currently thought to contain these sites due to natural erosion
Muir Watershed								
80th Street Outfall Pond	-	-	-	-	-	-	-	-
Olnick/Ceridian/VTC Watershed								
Long Meadow Pond – Upper	Along existing city streets	Private residential parcels	Existing storm sewer will need to be re-built to proposed location	Soils are most likely very sandy with seeps along the bluff	Construction likely within steep ravines	May be minor areas along side hill seeps	Permitting requirements have not been identified	Moderate to high
Long Meadow Pond - Lower	Difficult access through existing USFWS property	Either Kelley family or USFWS	Existing storm sewer to be re-built to proposed location	Soils are most likely very sandy with seeps along the bluff	Difficult access/storm sewer construction down bluff	Pond would likely impact a natural creek	Likely require Wetland permits	Moderate to high
Typical Development On-Site NURP Basin								
Health Partners	Through Development	Developer	Constructed with redevelopment	-	-	-	-	-

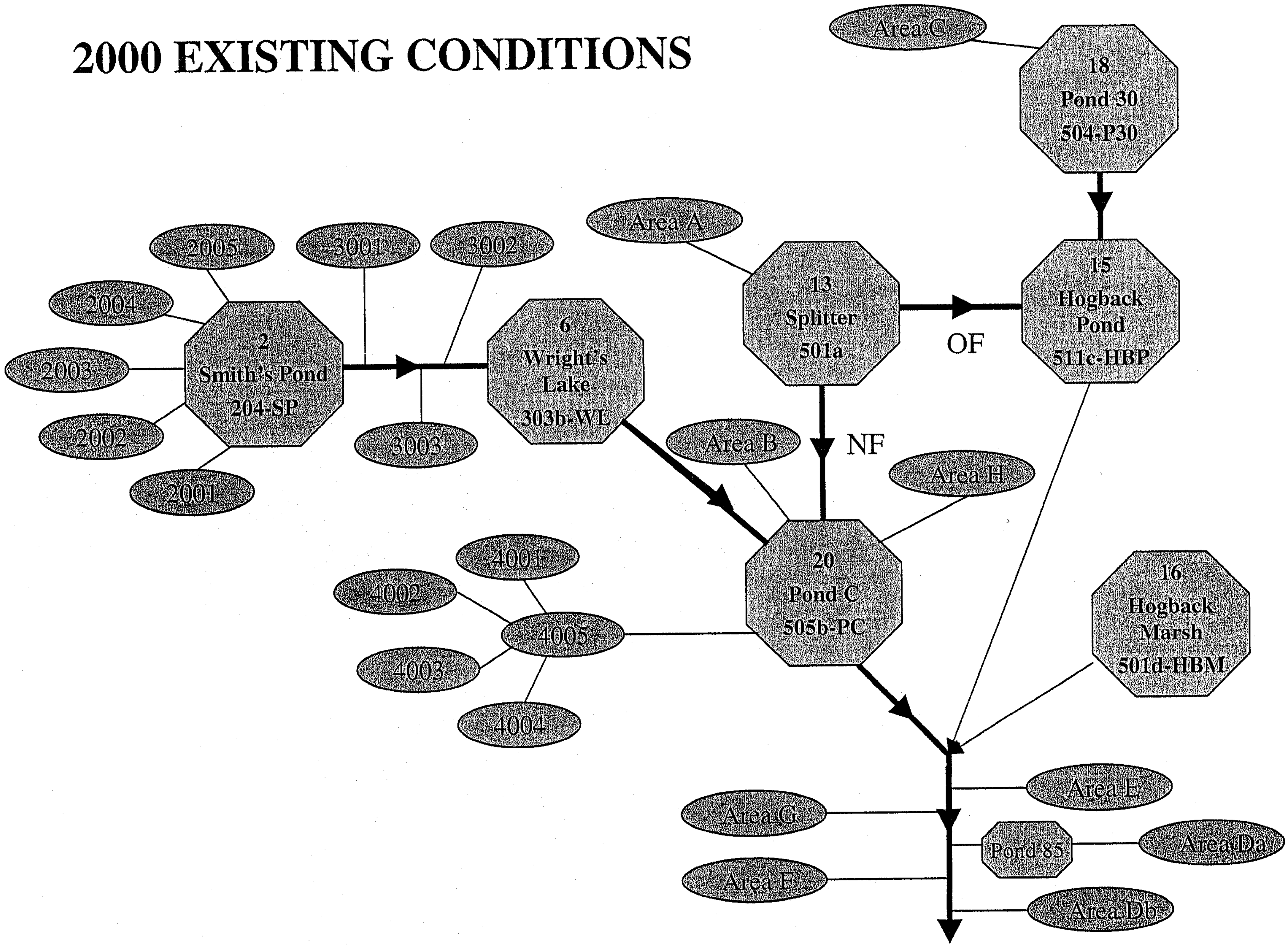
Table A.1
Pond Feasibility Matrices (cont.)

LOCATION	CRITERIA — LONG-TERM OPERATION ISSUES: Access (Right-of-way for maintenance)	Aesthetics	Maintenance Issues	Other Issues/Comments	Decisions
Pond C Watershed					
Pond C	Maintenance access will likely be maintained through the USFWS Refuge	Two-cell option may be more aesthetically-pleasing	See "Access (Right-of-Way for Maintenance)."	〈 Horizontal/vertical alignment/location of existing 54", 60" and 66" storm sewer outfalls to this pond need to be accurately located	
West of TH 77	Access via city streets	Pond would be within existing residential areas		〈 Provides treatment for areas flowing to Pond C untreated by Smith Pond/Wrights Lake. 〈 Significant right-of-way needs	
Hogback Pond Watershed					
Ravine Pond	ROW for maintenance vehicle access needs to be identified			〈 May require relocating the existing trash removal structure. 〈 Soil conditions may be unfavorable and/or cost prohibitive if structural stability measures required 〈 Area is adjacent to known areas of cultural resources sites (i.e. surrounding bluffs) 〈 Seeps from the existing ravines may cause instability to liners	
Muir Watershed					
80th Street Outfall Pond				〈 The potential pond area is much smaller than that for the Lower Long Meadow Pond with about the same drainage area	No appropriate site available due to physical restraints
Olnick/Ceridian/VTC Watershed					
Long Meadow Pond - Upper	Access via city streets	Pond would be required as part of the site redevelopment		〈 Ponding would be a cooperative effort with redevelopment 〈 Requires reconstruction of storm sewer system within Old Shakopee Road.	Removed from the regional pond concepts. Identified in the models as 'LMA' in the NURP basins
Long Meadow Pond - Lower	Access would likely occur via USFWS service roads	Would remove a small area of upland within the wetland complex		〈 The pond will intercept an existing creek channel. Assuming normal flow in the creek of 1 cfs, the available dead pool volume would be exchanged about once per day 〈 Requires reconstruction of storm sewer within Old Shakopee Road 〈 Would likely require construction of a new outfall pipe down the bluff. Location will affect cost	Site removed from the regional list due to physical constraints
Typical Development On-Site NURP Basin					
Health Partners	Through development		Developer		

Appendix B

P-8 Water Quality Model Diagrams

2000 EXISTING CONDITIONS



2020 DEVELOPMENT WITH NURP PONDS