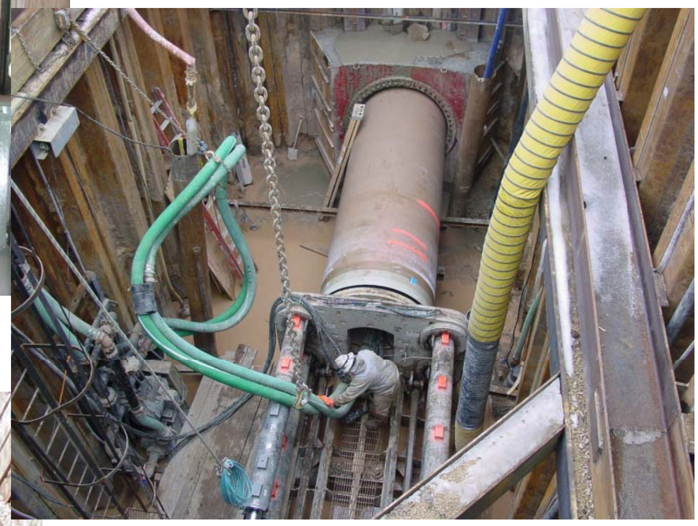

Madison Metropolitan Sewerage District Collection System Facilities Plan Update

Prepared by the Staff of the
Madison Metropolitan Sewerage District

December 2011

MADISON METROPOLITAN
SEWERAGE DISTRICT

COLLECTION SYSTEM FACILITIES PLAN UPDATE

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The Mission of the Madison Metropolitan Sewerage District

To protect public health and the environment, the Madison Metropolitan Sewerage District provides exceptional wastewater collection, treatment, and related services to the metropolitan Madison area and surrounding areas in a wise and cost-effective manner.

PREFACE

Clean water is a precious resource, and the collection and treatment of wastewater hold prominent roles in preserving that resource.

Wastewater collection systems represent a crucial segment of public infrastructure. Collection systems are responsible for continuously conveying huge volumes of household, commercial, and industrial wastewater to treatment facilities where the water can be cleaned and safely returned to the environment. Extensive networks of gravity interceptors, pumping stations, and pressure sewers must operate 24 hours per day and 365 days per year to accomplish this important function.

A robust and reliable collection system is at the heart of the Madison Metropolitan Sewerage District's core services. The essence of the District's Collection System Facilities Plan is to ensure that a high level of reliability continues into the future, supporting MMSD's mission to protect public health and the environment.

This Facility Plan Amendment builds upon the original Collection System Facilities Plan (2002) and ensures that the District's collection system, a huge and dynamic asset of the Madison region, provides sustainable wastewater conveyance by managing, improving, and expanding the system in a wise and cost-effective manner.

Chapter 1
Introduction and Summary

Chapter Outline

This chapter is organized into the following sections:
- Purpose
- Recognition and Dedication
- Background Information
- A Valuable but Aging System
- Methodology & Results
- DNR Facility Planning
- Public Participation

Purpose

The purpose of this *Collection System Facilities Plan Update* is to update and revise the original Collection System Facility Plan conducted in 2002. That Plan reviewed and assessed the adequacy and condition of the Madison Metropolitan Sewerage District's (MMSD's) collection system at that time and identified a set of recommended future collection system projects and an approximate timeline for their completion. MMSD has completed many of the recommended projects over the past nine years since the original Plan was completed, and this update will review those projects remaining on the list while identifying additional projects that will need to be completed in the future to sustain and/or enhance the integrity of MMSD's collection system.

The recommended projects are intended to provide additional soundness to MMSD's overall collection system and to systematically improve or replace individual facilities as needed. In some cases, alternate future scenarios or paths exist and will be dependent on future decisions and study. This document therefore identifies an initial direction and scope of projects that will address MMSD's greatest priorities, while also retaining flexibility for future developments and changes. As with the past facilities plan, the assessments and timetables presented in this facilities plan should be regularly reviewed and updated as significant developments occur and as future information is obtained. In this way, the facilities plan will continue to serve as a functioning planning document well into the future.

This *Collection System Facilities Plan Update* is a reflection of MMSD's continued efforts to provide wastewater services in a wise and cost effective manner. Ensuring that MMSD's collection system remains robust and reliable is the ultimate goal of this planning work.

Recognition and Dedication

The MMSD collection system has been developed over the course of a century, and numerous studies, facility plans, maps, design reports, and evaluations have been prepared over the years. These previous works, many of which were prepared by MMSD staff members, represent a valuable collection of knowledge and insight. Much of this Collection System Facilities Plan has been built upon earlier work, and the writers wish to recognize the many MMSD staff members, consultants, contractors and agencies whose contributions have made this possible. Among the essential building blocks for this facilities plan is the flow and population projection work presented in the "MMSD Collection System Evaluation" (January 2009) prepared by the staff of the Capital Area Regional Planning Commission (CARPC) with significant input and review by MMSD staff. Excerpts from that report are in Appendix A1 and it is referenced throughout this document. Although a separate report, we will use and refer to the Collection System Evaluation as if it were a separate volume of this facilities plan.

The writers would also like to recognize the hard work and dedication of MMSD's staff over the past ten years in completing many of the numerous projects identified in the original Collection System Facilities Plan. A plan is only a plan without the follow-up action to make its recommendations a reality. The improvements to MMSD's system since the original plan have made MMSD's system, a good system at the time, even better and more robust than it was in 2002. We hope and believe that the recommendations within this update will accomplish as much or more than those contained in the original plan.

We would like to dedicate this *Collection System Facilities Plan Update* to MMSD's employees in recognition of and thankfulness for all of their hard work to accomplish the collection system improvements resulting from the first planning effort.

Background Information

The Madison Metropolitan Sewerage District (MMSD) was established in 1930 to consolidate wastewater service for areas surrounding Lake Monona and Lake Mendota. MMSD initially served a 50-square mile area that included Madison, Middleton, Monona, Maple Bluff, Shorewood Hills, and surrounding towns. By the end of 2010, the MMSD service area had grown to approximately 180 square miles.

The MMSD collection system currently conveys wastewater from the Cities of Fitchburg, Madison, Middleton, Monona, and Verona; the Villages of Cottage Grove, Dane, DeForest, Maple Bluff, McFarland, Shorewood Hills, and Waunakee; and from sanitary and utility districts and other areas in the Towns of Blooming Grove, Burke, Dunn, Madison, Middleton, Pleasant Springs, Verona, Vienna, Westport, and Windsor. Additional areas are regularly annexed to the District.

Figure 1.1 is a map of the present-day MMSD collection system. A more detailed map is also available in Figure 9.1 (see enclosed map pocket inside cover), referenced in later chapters of this facilities plan. The MMSD collection system includes approximately 96 miles of gravity interceptor sewers, 17 regional pumping stations, and 29 miles of force mains. These MMSD-owned facilities collect the wastewater from local community-owned collection systems and convey the flow to the Nine Springs Wastewater Treatment Plant. Presently, all wastewater generated in the MMSD service area is treated at this single plant.

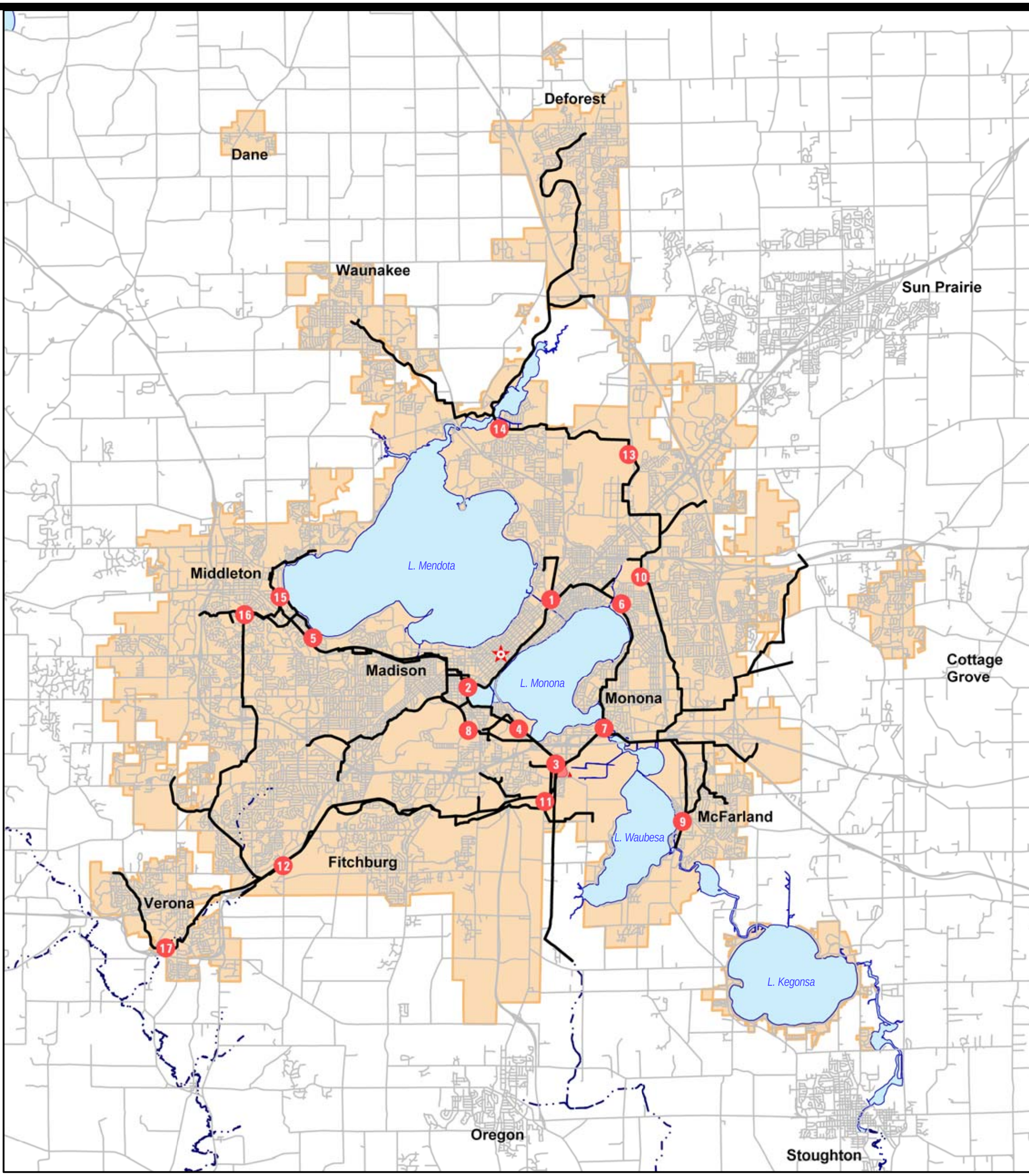
The MMSD system is somewhat unusual in that all flow is *pumped* into the Nine Springs Wastewater Treatment Plant through remote pumping stations and forcemains. The elevation of the treatment plant, constructed in 1926 on a hillside south of the city, is higher than various portions of the metropolitan service area. The geography of the Madison area, including multiple large lakes, a central isthmus, and hilly topography, also contributes to MMSD's special dependence on pumping stations and forcemains for flow conveyance. There are a total of 129 pumping stations (not including 429 small "grinder" pump installations) within MMSD's boundaries. Of these, 17 are owned and maintained by MMSD. The District also maintains 44 of the pumping stations owned by several of the communities it serves.

For the year 2010, MMSD received a total average wastewater flow volume of 43.0 mgd (million gallons per day). With increases in MMSD service area and population, this flow volume has significantly increased over the years and will continue to increase in the future. Figure 1.2 is a plot of MMSD's historical average flows and projected future average flows. As shown, the total average MMSD flow is expected to increase from 43.0 mgd in the Year 2010 to about 50 mgd by the Year 2030. This corresponds to an average increase of about 0.35 mgd per year, or a growth rate of about 0.8 % per year.

A Valuable but Aging System

Table 1.1 summarizes the construction history and replacement value for the MMSD collection system. A brief look at Table 1.1 reveals a long history of construction and indicates that many of MMSD's early collection system facilities are still in use. Although it is difficult to assign an exact useful life for such facilities, the average age of the MMSD collection system is clearly increasing. Figure 1.3 plots the average age and replacement value of the collection system assets. Much of the MMSD collection system was constructed prior to 1970, and Figure 1.3 shows a steady trend upward in average age since that time. The figure also shows that the MMSD collection system represents a very large investment. Based on original construction costs updated per the Engineering News Record Construction Cost Index, the estimated value of MMSD's collection system assets exceeds 250 million dollars.

MMSD actively monitors its collection system facilities and has replaced and rehabilitated numerous components over the years. Table 1.2 is a summary of significant replacement, rehabilitation, relief and major maintenance projects completed by MMSD



Legend

N

Pumping Station	MMSD Service Area	Road	
MMSD Main	Nine Springs WWTP	Receiving Stream	

MMSD	**Madison Metropolitan Sewerage District**	Figure 1.1 Collection System	Prepared by: JP Date: 1/5/11

Figure 1.2
Total MMSD Average Annual Flows: Historical and Projected

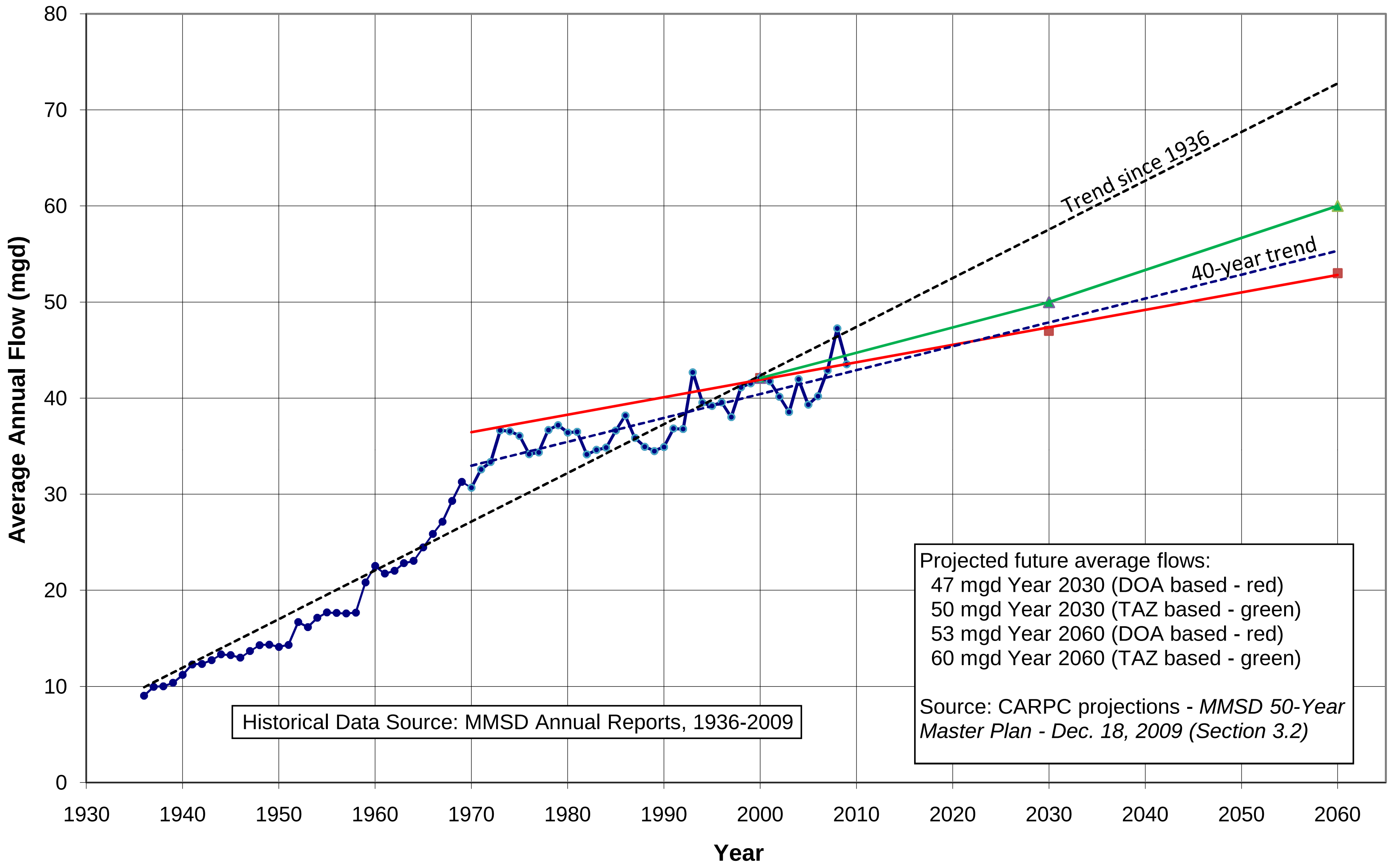


Table 1.1
Construction History of Major Collection System Assets

Collection System Asset / Other Historical Milestones	Placed in Service	Replacement Value Based on Original Cost and ENR Construction Cost Index (2010 $)	Comments
First City of Madison Treatment Plant	1899		Located near Yahara River at East Wash. Avenue Chemical precipitation plant that didn't work Only operated from June 1899 until January 1901
Second City of Madison Treatment Plant	1901		Built next to first - operated until 1914 Consisted of septic tanks with cinder filters
Burke Treatment Plant placed in service	1914		Operated by MMSD until 1936, U.S. Gov. 1942-1946, MMSD 1947-1950, rented by Oscar Meyer 1950-1979, Property sold to Reynolds Transfer & Storage in 1981
Main Pumping Station - (Old PS No. 1)	1916	$ 0	Abandoned in 1950 when new PS No. 1 went into service
Greenbush Pumping Station - (Old PS No. 2)	1916	$ 0	Abandoned in 1964 when new PS No. 2 went into service
Crosstown Force Main	1916	$ 0	Replaced in 2002
Burke Outfall Sewer and Main PS Force Main	1914 & 1916	$ 0	Replaced by North Basin Interceptor in 2002
"Old" Old West Interceptor	1916	$ 690,000	
Wingra Pumping Station - (Old PS No. 3)	1921	$ 0	Abandoned when SW Int. placed in service - 1956
Northend Int. - along Sherman Ave.	1925	$ 120,000	
Fair Oaks East Monona Interceptor	1926	$ 190,000	Replaced downstream of Starkweather Creek in 1997
Northend Int. - along Commercial to Pennsylvania	1927	$ 150,000	
Old Southwest Interceptor - W. Shore Drive	1927	$ 0	Replaced in 2001
(Nine Springs WWTP placed in service)	1928		Prior to this all flow went to the Burke Treatment Plant
Pumping Station No. 2 FM	1928	$ 0	Replaced in 2001
South Interceptor - Baird Street Ext.	1928	$ 120,000	
South Madison Pumping Station - (Old PS No. 4)	1928	$ 0	Abandoned when New PS No. 4 went online in 1967
Creation of the District	1930		By decree of Judge George Kroncke
Old West Interceptor	1931 - 1934	$ 6,090,000	
Old Southwest Int. - Cherokee Dr. to Nakoma Rd.	1932	$ 150,000	
Spring Harbor Pumping Station - (Old PS No. 5)	1932	$ 0	Superstructure and new pumps added in 1959 Abandoned when New PS No. 5 went online in 1996
Northeast Interceptor Relief	1937	$ 170,000	
"Old" Southwest Interceptor Extension	1938	$ 540,000	
Spring St. Relief from Randall Ave. to W. Wash. St.	1941	$ 1,400,000	Original construction paid by City of Madison in 1941.
Commercial Ave. Pumping Station - (Old PS No. 8)	1947	$ 0	Temporary - pumped to Burke Plant - dismantled in 1952
East Interceptor	1950	$ 18,720,000	Parts of East Int. replaced in phases (Phases I-V to date)
Pumping Station No. 1	1950	$ 5,580,000	Remaining value, rehabilitation in 2005
Pumping Station No. 6	1950	$ 6,430,000	
Pumping Station No. 7	1950	$ 4,460,000	Remaining value, rehabilitation in 1992
Southwest Interceptor	1956	$ 4,830,000	
West Interceptor Extension	1957	$ 1,320,000	
West Interceptor Relief	1958, 1960	$ 5,910,000	
Effluent Diverted to Badfish Creek	1958		
Pumping Station No. 3	1959	$ 400,000	Acquired from the village of Monona New pumping units 1980, electrical rehab. 1998
Rimrock Interceptor	1959	$ 370,000	
West Interceptor - Randall Relief	1962	$ 13,220,000	
Southeast Interceptor	1962	$ 7,370,000	
Southeast Interceptor Extension	1962	$ 2,650,000	
Pumping Station No. 9	1961	$ 840,000	Replacement value seems low.
Pumping Station No. 8	1963	$ 3,850,000	Improvements to electrical services by utility in 2000
Pumping Station No. 2	1963	$ 1,910,000	Remaining value, rehabilitation in 2005
Southeast Interceptor - Dutch Mill Extension	1964	$ 720,000	
West Interceptor - PS No. 2 Interceptor Work	1964	$ 390,000	
Second PS No. 7 Force Main	1963	$ 2,680,000	
Northeast Interceptor - SEI to FEI	1964	$ 3,390,000	SEI to FEI
Pumping Station No. 10	1964	$ 2,370,000	Remaining value, rehabilitation in 2005
Nine Springs Valley Interceptor - PS 11 to PS 12 FM	1965	$ 12,200,000	Nine Springs to McKee Rd
Pumping Station No. 11	1964	$ 4,280,000	Remaining value of original construction.
Northeast Interceptor - Burke Extension	1966	$ 0	Replaced by Hwy 30 Ext Replacement in 1996
West Interceptor - Gammon Extension	1966	$ 1,180,000	Replacement value of remaining sewer to Gammon Rd.
West Interceptor - West Point Extension	1966	$ 600,000	Includes Baskerville siphon.
Pumping Station No. 4	1966	$ 1,450,000	
South Interceptor - Lakeside Extension	1966	$ 1,440,000	
Southeast Interceptor - Blooming Grove Extension	1967	$ 2,260,000	
Northeast Interceptor - Truax Extension	1968	$ 7,000,000	Lien Rd to west side of N-S runway at airport
Nine Springs Valley Int. - Mineral Point Extension	1968	$ 7,080,000	McKee Rd to PS 12 to Mineral Point Rd, incluidng PS 12 force main
Pumping Station No. 12	1968	$ 2,110,000	Remaining value of original construction.
Far East Interceptor	1969	$ 3,090,000	NEI to east side of Interstate Highway
Pumping Station No. 13	1969	$ 2,290,000	Remaining value of original construction.
Northeast Int. - Waunakee & DeForest Extensions	1970	$ 24,420,000	Airport to Waunakee and DeForest, including PS 14 force main
Nine Springs Valley Int. - Waubesa Extension	1971	$ 1,430,000	
West Interceptor - Midvale Relief	1971	$ 650,000	

Table 1.1
Construction History of Major Collection System Assets

Collection System Asset / Other Historical Milestones	Placed in Service	Replacement Value Based on Original Cost and ENR Construction Cost Index (2010 $)	Comments
Northeast Int. - Highway 19 Extension	1971	$ 710,000	
Pumping Station No. 14	1972	$ 1,880,000	Remaining value of original construction.
Clean Water Act	1973		
West Interceptor - Spring Harbor Relief (Force Main)	1973	$ 1,420,000	
Pumping Station No. 15	1974	$ 1,500,000	Remaining value of original construction.
Nine Springs Valley Int. - Syene Extension	1974	$ 740,000	
Nine Springs Valley Int. - Hwy. 14 Extension	1977	$ 1,030,000	
West Interceptor - Esser Pond Extension	1978	$ 430,000	Middleton Street to Hwys 12-14, installed in open ground
East Interceptor - Johnson Street Relief	1979	$ 410,000	
Pumping Station No. 16 Force Main	1979	$ 2,100,000	
Pumping Station No. 15 FM Relocation	1981	$ 110,000	
Pumping Station No. 16	1981	$ 3,870,000	
Pumping Station No. 15 FM Diversion	1982	$ 910,000	
Far East Interceptor & Cottage Grove Extensions	1982	$ 970,000	
Pumping Station No. 16 Force Main - Air Vent	1983	$ 9,000	
West Interceptor - Esser Pond Extension	1986	$ 150,000	
Southeast Interceptor - McFarland Relief	1987	$ 940,000	
Pump Station 9 Second Force Main	1987	$ 510,000	
Northeast Int. - Starkweather Ext./Hwy 51 Crossing	1990	$ 30,000	Original casing installed in open-cut.
Pumping Station No. 7 Rehabilitation	1991	$ 3,860,000	
City of Verona annexed to the District	1993		District operates and maintains Verona WWTP
Southeast Interceptor - Siggelkow Extension	1994 & 1996	$ 520,000	
South Interceptor Replacement	1994	$ 910,000	MH 4109 on Lakeside Ext to Wingra Dr, including siphon replacement under Wingra Creek at Beld St
Northeast Interceptor - Lien Interstate Extension	1995	$ 780,000	
Northeast Interceptor - Hwy 30 Ext Replacement	1996	$ 160,000	Replace Burke Ext built in 1966.
Pumping Station No. 5	1995	$ 2,190,000	New pumping station built to replace old PS No. 5
Verona Pumping Station Force Main	1995	$ 1,540,000	
Verona Pumping Station (Pumping Station No. 17)	1995	$ 2,720,000	Verona WWTP abandoned
Effluent returned to Badger Mill Cr./Sugar River	1998		
Far East Interceptor - Door Creek Extension	1998	$ 2,260,000	
Nine Springs Valley Int. - Midtown Extension	1999	$ 1,060,000	
Crosstown Force Main Replacement - Yahara River	1999	$ 680,000	
West Interceptor - Campus Relief Phase 1	1999	$ 1,000,000	
West Interceptor - Campus Relief Phase 2	2000	$ 1,310,000	
P.S. #2 Forcemain Replacement	2000-2001	$ 5,600,000	
NSVI-Nicolet Replacement	2000	$ 210,000	
PS No. 1 North Basin Interceptor	2002	$ 3,410,000	
Crosstown Force Main Replacement	2002	$ 5,880,000	
WI - Gammon Ext - Fortune Drive Replacement Sewer	2002	$ 550,000	
Rehabilitation of Pump Stations 1 - 2 - 10	2003	$ 10,450,000	
West Interceptor - Campus Relief Phase 4	2004	$ 1,690,000	
Lower Badger Mill Creek Int - Cross Country Rd	2004	$ 120,000	
NEI Pflaum Rd Replacement Sewer	2005	$ 3,590,000	
Lower Badger Mill Creek Int - Ph 1	2006	$ 2,140,000	
SWI North and South Legs Relining	2006	$ 0	Sewers transferred to City of Madison in 2010.
PS 13 and 14 Firm Capacity Upgrades	2007	$ 670,000	
WI Ext Replacement	2007	$ 2,240,000	
Lower Badger Mill Creek Int - Ph 2	2008	$ 1,070,000	
NEI - Truax Extension Liner	2008	$ 1,950,000	
FEI - Gaston Road Extension	2008	$ 760,000	
SI - Baird Street Extension Liner	2009	$ 120,000	
FEI - Cottage Grove Extension Liner	2010	$ 340,000	
NEI - PS10 to Lien Road Relief & Replacement Sewer	2010	$ 8,710,000	
Rehabilitation of Pump Station 6 & 8	2010	$ 6,580,000	
Total Costs		$ 261,299,000	

Figure 1.3 - Madison Metropolitan Sewerage District
Collection System Replacement Value and Age

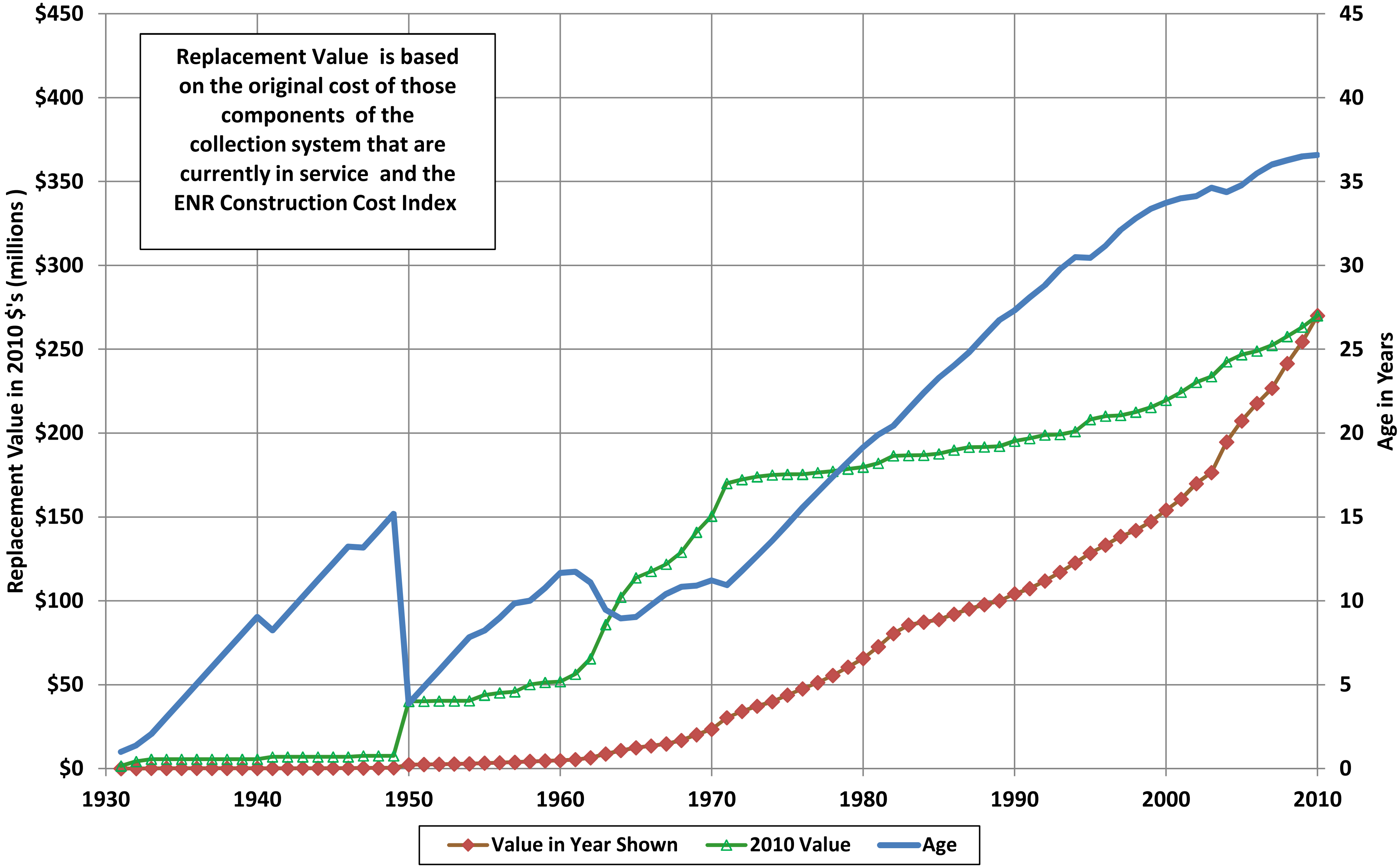


Table 1.2

Major Collection System Maintenance, Renewal, Replacement, and Relief Projects since 2000

Madison Metropolitan Sewerage District

Service Area / Project	Time Period	Approximate Costs (actual $)	Comments
System Wide Projects			
Annual televising projects	2000 - 2010	$ 1,030,000	Portions of MMSD's system are televised each year
Telemetry system improvements	2000	$ 118,000	Included new radios at plant and pumping stations
Pumping Station No. 1			
Rebuilt Pump A	2009	$ 19,000	
Pumping Station No. 1 Force Main Air Release MH	2006	$ 14,000	
Major rehab work on entire pumping station	2003 - 2006	$ 2,534,000	Included new and rebuilt pumps, electrical, and hvac
Crosstown FM Replacement - Phase 2	2002	$ 4,335,000	
Install new hoist and motorize bridge and trolley	2002	$ 20,000	
Burke Outfall Replacement	2002	$ 2,515,000	
Crosstown FM replacement at Yahara River	2000	$ 467,000	1,330 feet near PS No. 1
Pumping Station No. 2			
Rebuilt Pump A	2008	$ 22,000	
WI Repairs at Park Street	2007	$ 40,000	
Major rehab work on entire pumping station	2003 - 2006	$ 2,980,000	Included new and rebuilt pumps, electrical, and hvac
Repair FM leak	2003	$ 44,000	Along Olin Avenue
PS No. 2 FM Replacement	2000 - 2001	$ 3,966,000	17,000 feet of new 36" ductile iron
Southwest Int. Replacement on Shore Drive	2001	$ 437,000	1,700 feet of new 36" PVC interceptor
PS No. 2 Roof Replacement	2001	$ 18,000	
Pumping Station No. 3			
Install flowmeter	2005	$ 13,000	Part of PS 1, 2, & 10 Rehab Project
Pumping Station No. 4			
SI - Baird Street Extension Liner	2009	$ 113,000	
Second feed and transfer switch	2004	$ 60,000	Second power feed and transfer switch by MG&E
PS 4 painting	2003	$ 11,000	Contractor painted pumps, piping, motors, and railings.
Replace telmetry system and modify controls	2001 - 2002	$ 23,000	
Pumping Station No. 5			
PS 5 painting	2006	$ 13,000	
Replace Pump A Adjustable Frequency Drive	2005	$ 13,000	
Pumping Station No. 6			
Major rehab work on entire pumping station	2008 - 2010	$ 3,300,000	Work in progress - New pumps, electrical, hvac, etc.
Repair force main break after contractor damage	2009	$ 133,000	$125,000 was reimbursed as part of the settlement

Table 1.2
Major Collection System Maintenance, Renewal, Replacement, and Relief Projects since 2000
Madison Metropolitan Sewerage District

Service Area / Project	Time Period	Approximate Costs (actual $)	Comments
Pumping Station No. 6 continued			
Repair motor for Pump D	2006	$ 19,000	
Remove bar screen	2006	$ 7,000	In conjunction with new Plant headworks - 10th Addition
Install new hoist and motorize bridge and trolley	2002	$ 22,000	
Pumping Station No. 7			
FEI Cottage Grove Extension - Lining Project	2010	$ 343,000	Lined 5500 feet of 18-inch sewer
Replace bubbler system for level controls	2009 - 2010	$ 22,000	Work in progress - costs as of 12/3/2010
New sluice gate acutators	2009 - 2010	$ 96,000	Work done as part of PS 6 & PS 8 Rehab project
Roof replacement	2009	$ 22,000	
Third power feed to pumping station	2009	$ 87,000	MG&E installed 3rd power feed to pumping station site
Installed portable generator connection point	2009	$ 26,000	Connection for portable generator
Rebuilt Pump A	2009	$ 14,000	
Rebuilt Pump D	2009	$ 14,000	
Rebuilt Pump B	2009	$ 15,000	
FEI - Gaston Road Extension	2008	$ 714,000	
Rebuilt Pump B	2007	$ 12,000	
Remove bar screen	2006	$ 7,000	In conjunction with new Plant headworks - 10th Addition
Northeast Int. - Pflaum Road	2005 - 2006	$ 3,012,000	Relief for 5000 feet of sewer
Peak Capacity Modifications	2002	$ 26,000	
PS 7 FM	2001	$ 18,000	Added Air Release Manhole on FM near WPS
Pumping Station No. 8			
Major rehab work on entire pumping station	2008 - 2010	$ 3,300,000	Work in progress - New pumps, electrical, hvac, etc.
West Interceptor - Walnut Street Siphon Cleaning	2008	$ 102,000	
Southwest Interceptor - Line North & South Legs	2007	$ 519,000	Lined north and south legs of SW Interceptor
Replace suction valve on Pump D	2005	$ 17,000	
West Interceptor Campus Relief - Phase IV	2004	$ 1,354,000	Relief of WI to Walnut Street
Install actuator on Pump C discharge valve	2004	$ 17,000	
Replace suction valve with actuator on Pump A	2004	$ 44,000	
Southwest Interceptor - Chippewa Drive Rehab	2001	$ 49,000	
West Interceptor Campus Relief - Phase III	2000	$ 525,000	1,100 ft of 36" pipe behind stock pavilion
West Interceptor Campus Relief - Phase II	2000	$ 918,000	700 feet of new 48 inch pipe crossing Campus Drive
Power System Modifications	1999 - 2001	$ 60,000	Included new underground services from MG&E
Roof replacement	2000	$ 17,000	
Rebuilt Pump B	2000	$ 16,000	

Table 1.2

Major Collection System Maintenance, Renewal, Replacement, and Relief Projects since 2000

Madison Metropolitan Sewerage District

Service Area / Project	Time Period	Approximate Costs (actual $)	Comments
Pumping Station No. 8 continued			
Install channel grinder at PS 8	1999 - 2000	$ 98,000	Grinder removed from service in 2004.
Pumping Station No. 9			
New Pump B with motor	2007	$ 27,000	In-house pump installation
Install new electrical services and equipment	2004	$ 191,000	Included all new electrical and two power services
New Pump A with motor	2004	$ 21,000	In-house pump installation
New Pump C with motor	2002	$ 26,000	In-house pump installation
Pumping Station No. 10			
NEI - PS 10 to Lien Road	2009 - 2010	$ 8,710,000	Work in progress - provide relief for 9200 feet of sewer
Rebuilt Pump B	2009	$ 11,000	Sent to Cornell for warranty repair
Rebuilt Pump A	2008	$ 14,000	Sent to Cornell for warranty repair
Major rehab work on entire pumping station	2003 - 2006	$ 2,619,000	Included new and rebuilt pumps, electrical, and hvac
Rebuilt Pump C	2002	$ 11,000	
Pumping Station No. 11			
Rebuilt Pump B	2009	$ 16,000	
Rebuilt Pump B	2009	$ 18,000	
Rebuilt Pump B	2007	$ 14,000	
Install dehumidifier	2006	$ 17,000	
Remove bar screen	2006	$ 7,000	In conjunction with new Plant headworks - 10th Addition
PS 11 painting	2004	$ 27,000	
Control system improvements	2001 - 2002	$ 30,000	Replace relay panels with PLC controls
NSVI Nicolet Replacement	2000	$ 150,000	Replaced 1,170 feet of corroded pipe with new 30" PVC
Pumping Station No. 12			
Rebuilt Pump A	2010	$ 28,000	
Install dehumidifier	2005	$ 15,000	
PS 12 painting	2001	$ 23,000	
Control system improvements	2000 - 2001	$ 28,000	Replace relay panels with PLC controls
Pumping Station No. 13			
Replace well level controls	2009	$ 22,000	Adjust new float levels to new levels in SCC
NEI - Truax Area Liner	2008	$ 1,832,000	
Replace Pump B suction valve	2007	$ 25,000	
Upgrade pumping station firm capacity	2006 - 2007	$ 291,000	New and rehabbed pumps and control modifications

Table 1.2
Major Collection System Maintenance, Renewal, Replacement, and Relief Projects since 2000
Madison Metropolitan Sewerage District

Service Area / Project	Time Period	Approximate Costs (actual $)	Comments
Pumping Station No. 13 continued			
NEI - Airport Reconstruction	2005	-	Relocated with airport work - reimbursed by Dane Co.
Install dehumidifier	2004	$ 12,000	
Replace Pump A motor starter	2002 - 2003	$ 8,000	New soft starter and controls installed
PS 13 painting	2002	$ 23,000	
Control system improvements	2002	$ 31,000	Replace relay panels with PLC controls
Pumping Station No. 14			
Install monitoring manhole, MH14-156A	2008	$ 85,000	
Upgrade pumping station firm capacity	2006 - 2007	$ 314,000	New and rehabbed pumps and control modifications
Replace Pump A motor starter	2005	$ 8,000	New soft starter and controls installed
Install dehumidifier	2004	$ 12,000	
PS 14 painting	2003	$ 17,000	
Control system improvements	2003	$ 39,000	Replace relay panels with PLC controls
Pumping Station No. 15			
Rebuild Pump B	2008	$ 13,000	
West Int. Extension Replacement	2007	$ 2,014,000	
Installed new station control center (SCC)	2003	$ 39,000	Replace relay panels with PLC controls & HMI
PS 15 force main casting replacements - Allen Blvd	2000	$ 11,000	
Pumping Station No. 16			
Replace shingled roof	2008	$ 9,000	
Major control system renovations/replacement	2007 - 2010	$ 200,000	In-house design & installation of control system changes
Install dehumidifier	2005	$ 17,000	
PS 16 painting	2005	$ 17,000	
West Interceptor Fortune Drive Relief Sewer	2002	$ 406,000	
Odor Control System	2000	$ 26,000	
Pumping Station No. 17			
Rebuilt Pump A	2010	$ 21,000	
Rebuilt Pump C	2009	$ 15,000	
Rebuilt Pump B	2008	$ 23,000	
Lower Badger Mill Creek Int - Ph 2	2008	$ 1,000,000	
Control system modifications to allow dual pumping	2007	$ 6,000	Primarily staff time for re-programming & testing
New transformer installed - allows dual pumping	2007	$ -	New 300 kVA transformer installed by Alliant Energy
Lower Badger Mill Creek Int - Ph 1	2006	$ 1,869,000	

Table 1.2

Major Collection System Maintenance, Renewal, Replacement, and Relief Projects since 2000

Madison Metropolitan Sewerage District

Service Area / Project	Time Period	Approximate Costs (actual $)	Comments
Pumping Station No. 17 continued			
Rebuilt Pump B	2006	$ 19,000	
Lower Badger Mill Creek Int - Cross Country Rd	2004	$ 99,000	
Rebuilt Pump C	2004	$ 16,000	
PS 17 painting	2004	$ 13,000	
Rebuilt Pump B	2002	$ 18,000	
Replace main circuit breaker	2002	$ 16,000	

during the last decade. Although this list of projects is substantial, compromising over $54 million worth of improvements and repairs, it represents a relatively small portion of MMSD's extensive collection system. As the overall age of the system continues to increase, it is likely that the rate of such replacement and improvement projects will need to accelerate to ensure the continued high reliability that MMSD requires.

Methodology and Results

As detailed in the following chapters, this facilities plan recognizes the need for improvements based upon several factors. Each major component of the collection system is evaluated for hydraulic capacity and physical condition. The interaction between the major system components is also examined to help identify where and how specific projects can be combined and prioritized.

The result of this approach is a list of recommended projects and initiatives with an approximate timeline for completion. These results are presented in Chapter 9 and are intended to serve as a future guide for MMSD collection system planning, budgeting, and construction. Since the recommended timetable covers a long period (20-years), it is likely that the scope and priority of some projects may change as more detailed studies are performed and as future developments occur. It is recommended that the project timetable be annually reviewed and updated and that the results be incorporated into MMSD's capital budgeting process.

DNR Facility Planning

Collection system projects are funded by MMSD through two main sources: (1). Connection charges paid by new users that connect to existing infrastructure; and (2). Clean Water Fund (CWF) loans administered by the Wisconsin Department of Natural Resources (WDNR). The DNR requires that projects funded through CWF loans include a "facility planning" step. In general the facility plan report should include, at a minimum, the following elements:

- Description of proposed project and the need for the project.
- Preliminary cost estimate and expected user charge impacts to a typical customer.
- Environmental impacts of project, especially those related to floodplain, wetlands, or other environmentally sensitive areas.
- Letter from the Capital Area Regional Planning Commission stating conformance of the project with approved urban sewer service areas.

Pre-Design and Design Reports for each project are intended to be developed after facility planning in conjunction with the preparation of detailed plans and specifications and would address issues related to alternatives analysis and cost-effectiveness.

This Facilities Plan describes each proposed project and the driving forces for its construction. Preliminary cost estimates and a generalized user charge impact are provided in Chapter 9. However, detailed user charge impacts and environmental impacts are not developed in this document due to the unique nature of each project. MMSD believes that these issues are best addressed as part of Pre-Design or Design Reports that will be submitted to the WDNR for each project. As such, this Facilities Plan is not meant to satisfy all of the facility planning requirements set forth by the WDNR in order to secure CWF funding for a particular project.

Public Participation

The District held a public hearing on Wednesday, February 22, 2012, to present the methodology and recommendations of the facilities plan update and to solicit questions and comments from local officials and the general public. The public hearing was noticed in the local newspaper and notifications letters were also sent to each of the District's customers. A 12-day comment period was provided prior to the hearing for submission of written comments regarding the facilities plan update, which was made available for viewing at the District's administrative office and on its website.

Documents related to the public hearing are included in Appendix 11. No written comments were received from the public and no local officials or members of the general public attended the public hearing.

Approval letters for the update to the Collection System Facilities Plan were received from the Capital Area Regional Planning Commission and Wisconsin Department of Natural Resources on June 20, 2012 and July 20, 2012, respectively (see Appendix 12).

Chapter 2
Asset Management and CMOM

Chapter Outline

This chapter is organized into the following sections:

- Introduction
- Asset Management
- Capacity, Management, Operation and Maintenance (CMOM)

Introduction

This chapter will discuss the topics of asset management and CMOM (capacity, management, operation, and maintenance). The chapter is organized into three sections: this brief introduction, Asset Management, and CMOM. The sections on asset management and CMOM each contain specific conclusions and recommendations and are not reiterated here. However, a general summary of the conclusions and recommendations is provided.

The topics of asset management and CMOM have received a lot of attention at both the State and national levels. The definition of what constitutes a good asset management plan or CMOM program is fuzzy at best. However, general guidance is available as are many examples of best practices. It is up to each utility to determine what approaches and practices will be most beneficial in providing the best service to its customers.

The District's collection system facilities plan includes advanced asset management concepts and meets many of the criteria required in a CMOM program. Although the facilities plan may not include all aspects of either, it is certainly a part of the whole.

In this chapter, advanced asset management and CMOM concepts are reviewed and compared with the current practices used at the District. In general, the District takes a practical approach to managing its assets and in meeting regulations. Although the District's present approach appears to meet most of its needs, improvements and better approaches are always possible. Those improvements have been included within the recommendations and may be summed up as follows: (1). Provide better documentation, (2). Migrate towards the use of more advanced asset management and CMOM concepts, and (3). Develop and implement systems that are monitored and continually improved.

Asset Management

The District's *Collection System Facilities Plan* is an asset management plan. In conjunction with the District's maintenance programs, it is used to manage the District's collection system assets. The District has been progressively adopting additional asset management concepts (advanced asset management concepts) since its first collection system facilities plan was implemented in 2002. Eventually all of those advanced concepts that fit the District's approach will be incorporated into its asset management program. However, rather than a wholesale change, a migration toward using these more advanced asset management concepts is taking place.

The topic of asset management has received significant attention over the last five to ten years. As an engineering concept rather than an accounting concept, asset management considers how the assets of a utility can be optimized to provide the appropriate levels of service with an acceptable level of risk and at minimum life-cycle costs. Thus, asset management is defined as an integrated set of processes to minimize the life-cycle costs of infrastructure assets, at an acceptable level of risk, while continuously delivering established levels of service (definition from *Implementing Asset Management: A Practical Guide*). As stated in the EPA's advanced asset management training materials, Asset Management is the systematic integration of advanced and sustainable management techniques into a management paradigm or *way of thinking*, with primary focus on the *long-term life cycle* of the asset and its sustained performance, rather than on short-term, day-to-day aspects of the asset.

The District's *Collection System Facilities Plan* anticipates the timing of needs related to both condition and capacity. Each of the District's pumping station's physical condition is assessed by analyzing six categories on a scale of 1 to 5, with 1 being very good and 5 being very poor. The District's sewers are assessed for condition via its sewer maintenance televising program. Maintaining the District's sewers through cleaning, televising, and rehabilitation or replacement plays a major role in meeting expected levels of service. In addition, providing an appropriate level of maintenance is a part of minimizing life-cycle costs. The level of service is also established by determining the capacity adequate to meet peak events. The District has used a benchmark called the Madison Design Curve for many years to set the required capacity for its pumping and sewer systems. This benchmark sets a peaking factor of between 2.5 and 4.0 for all of the District's facilities based upon the average flow of the system's component. (This factor is described in more detail elsewhere in this facilities plan.)

The amount of information on advanced asset management concepts is significant although somewhat nebulous and non-standardized. This makes it difficult to compare what your organization is doing with a single standard or even with best practices. In the remainder of this section, comparisons have been made with what the EPA considers to be the Fundamentals of Asset Management and their ten step approach to developing an asset management plan. The District's *Collection System Facilities Plan* is fundamentally an asset management plan and as such provides a framework for improving the District's collection system and its assets (the system of pumps, pipes, manholes, structures, etc.). In addition, the *Collection System Facilities Plan* also provides a framework to continually improve the planning process itself, i.e., this "asset management" planning process and how it interfaces with the capital improvement plan.

The Five Core Questions

Per the US EPA's Fundamentals of Asset Management (retrievable from the EPA's website at http://www.epa.gov/OWM/assetmanage/assets_training.htm), there are five core questions to answer when developing an asset management framework. Those questions are as follows:

1. What is the current state of my assets?
    - What do I own?
    - Where is it?
    - What condition is it in?
    - What is its remaining useful life?
    - What is its remaining economic value?

2. What is my required level of service (LOS)?
    - What is the demand for my services by my stakeholders?
    - What do regulators require?
    - What is my actual performance?

3. Which assets are critical to sustained performance?
    - How does it fail? How can it fail?
    - What is the likelihood of failure?
    - What does it cost to repair?
    - What are the consequences of failure?

4. What are my best O&M and CIP investment strategies?
    - What alternative management options exist?
    - Which are the most feasible for my organization?

5. What is my best long-term funding strategy?

Answering most or all of these questions should lead to a well-developed and advanced asset management program.

The following comments should be made about several of the questions and more discussion will occur later. One of the questions under what is the state of my assets asks what the remaining useful life is. Age and better yet, the actual condition, can be good indicators of the remaining life of a piece of equipment from an operational standpoint, but may not be good indicators from the standpoint of capacity or the ability to meet actual system requirements or level of service. Therefore, from an asset management perspective, failures to meet capacity or other service level requirements are considered failure modes and can also limit the remaining useful life of an asset.

Note that another question asks about remaining economic value. This is sometimes difficult to assess. The original cost of a piece of equipment or system depreciated over time may be significantly different from its actual economic value. Perhaps a better indicator of economic value is replacement cost and the timing of the replacement. If a replacement can be delayed by repair or rehab, what is the value to the ratepayers of extending its life?

Another fundamental key of asset management is determining the desired level of service to provide and measuring actual performance. The desired level of service sets the bar for the utility's performance. Measuring actual performance determines where improvements need to be made.

Ten Steps to an Asset Management Program

One method of implementing the five core questions is a ten-step process also included in the EPA's Fundamentals of Asset Management and obtainable at the same website location as the five core questions. The ten steps are listed below:

1. Develop asset registry
2. Assess condition, failure modes
3. Determine residual life
4. Determine life cycle & replacement costs
5. Set target levels of service (LOS)
6. Determine business risk ("criticality")
7. Optimize O&M investment
8. Optimize capital investment
9. Determine funding strategy
10. Build asset management plan

Integrating the five core questions with the ten-step process answers the five core questions and helps develop a comprehensive asset management plan. Steps 1 to 4 relate to question 1. Step 5, and to some extent step 6, address question 2 regarding level of service. Step 6 relates primarily to question 3. Steps 7 and 8 address question 4. Step 9, and to some degree step 10, address question 5. Lastly, step 10 packages everything together. The list below adds a little more information related to each of the 10-steps without going into depth.

1. Develop asset registry
    - System layout
    - Data hierarchy, standards, and inventory

2. Assess condition, failure modes
    - Condition assessment protocol
    - Rating methodologies

3. Determine residual life
    - Expected life tables
    - Decay curves

4. Determine life cycle & replacement costs
    - Valuation
    - Life cycle costing

5. Set target levels of service (LOS)
    - Demand analysis
    - Balanced scorecard
    - Performance metrics

6. Determine business risk ("criticality")
    - FMECA (failure mode effects and criticality analysis)
    - Business risk (probability of failure times consequence of failure)
    - Delphi techniques

7. Optimize O&M investment
    - Root cause
    - RCM (reliability centered maintenance)
    - PdM (predictive maintenance)
    - ORDM (optimized renewal decision making)

8. Optimize capital investment
    - Confidence level rating
    - Strategic validation
    - ORDM

9. Determine funding strategy
    - Renewal
    - Annuity

10. Build asset management plan
    - Asset management plan
    - Policies and strategy
    - Annual budget

Existing Assessment of District Asset Management Practices

The District's Collection System Facility Plan addresses many of the above steps and other steps are addressed by the District's CMMS (computerized maintenance management system) and CIP (capital improvement plan). Without addressing all of the details of the District's approach, the following includes brief summaries of how the District meets or does not meet certain aspects of advanced asset management.

Step 1 – Asset Registry

Numerous drawings show the layout of the District's collection system and the assets and components that make up that system.

The District's computerized maintenance management system has a well-developed asset management registry with well-developed standards and a systematic parent-child hierarchy. The system is used for maintenance purposes, but is not used more globally for overall asset management. The District also has a financial asset management system (FAMS), which is used to track the book value of its assets. In general, this system is mainly used for accounting purposes, for not engineering or O&M purposes. Perhaps the future of asset management at the District will link these two systems together and the FAMS information will be based upon actual asset condition rather than the value of depreciated assets based solely on age, thus providing information that may be beneficial to engineering and O&M.

Step 2 – Assessing Condition and Failure Modes

A somewhat anecdotal and generalized system exists for assessing the condition of the District's pumping station assets. The adequacy of the firm and maximum capacity are determined by Capital Area Regional Planning Commission (CARPC) projections and the adequacy of the pumping station to meet capacity now and for the next twenty years. Power system redundancy (emergency measures), electrical system condition, mechanical system condition, and structural condition are less specific in their determination and there is therefore quite a bit more subjectivity built into the related assessments. Therefore, the assessments do not roll up from specific assessments of all equipment at the facility (e.g., assessments that would be made by maintenance staff during preventive or predictive maintenance work). However, assessments are based upon professional judgment by knowledgeable staff. Still, a more direct link between predictive maintenance findings and the condition ratings used in the facilities plan may be desirable and could enhance the results of future facilities (asset management) plans.

The adequacy of the capacity of the District's sewers is assessed using the same CARPC projections as were used to rate the pumping station capacities. The projections are used to determine the timeframe when the sewers will reach capacity and may need relief. Condition of the sewers is determined from findings of the sewer televising inspections that are completed on an annual basis. Deficiencies and problems are recorded in a database and this database is used to determine sewers that are most likely to require repair, rehab, or replacement. Although some problems exist with the present system, the system appears to be working reasonably well. With ongoing improvements to the system, it should be easily modifiable to meet the District's overall needs for asset management.

Step 3 – Determine Residual Life

The District assumes an asset's life based upon its age and type for purposes of depreciation in its Financial Asset Management System (FAMS). The actual useful life of equipment is determined by the asset's actual condition and anticipated remaining life. The two different approaches generally result in significantly different numbers. In addition, the life of an asset is often determined more by its ability to meet the conditions that are presently required and this can change over time, e.g., capacity requirements change or new regulations mean different equipment such as may occur with ventilation equipment.

Determining residual life and the use of decay curves does not presently receive much time or attention. In general, the condition of critical equipment is known and repair, rehab, or replacement of equipment and/or systems is anticipated and taken into account using the District's budget and planning processes. Repairs are generally treated as O&M expenses and addressed as maintenance while major rehab or replacement projects are treated as capital expenses.

Doing a more thorough job of determining and recording the residual life of equipment could potentially benefit overall planning and financial management by providing better information related to equipment needs and scheduling of repairs, rehabs, or replacements. However, the benefits of attempting to be more precise need to be balanced with the time commitment involved.

Step 4 – Determine Life Cycle and Replacement Costs

The District's present approach separates accounting requirements from actual long-term planning for needs. In addition, determining life cycle and replacement costs is only completed, if at all, at a very high level. Actual life, based upon asset condition, is oftentimes difficult to assess, and typically, capacity is the normal failure mode of the District's collection system assets. Replacement costs, for inclusion in the District's capital improvement plans, are also typically completed at a relatively high level until the design process begins and then, costs are refined as the project progresses.

Depreciated value says little about the actual value of a piece of equipment. In fact, even when equipment is depreciated using decay curve methods or by depreciating based upon the equipment's remaining life, the number tells little about the equipment's actual value. Depreciating based upon condition (the modified approach) may, however, help tell outside organizations more information than straight-line depreciation. If an organization is keeping its equipment well maintained and/or renewed, the modified approach will reflect some of the organization's good practices in its financial numbers.

Perhaps a better indicator of actual equipment value is its life cycle and replacement costs and the timing of those costs. This may be where the District could work at improving its present approach to long-term planning. Replacement costs should include an estimate of the life cycle costs for the best options. Even a modest approach to determining overall long-term replacement costs could prove helpful in identifying periods where the District might experience relatively high financial burdens due to renewal or replacement of existing infrastructure.

Step 5 – Set Target Levels of Service (LOS)

Regardless of whether or not an organization uses advanced asset management concepts or not, organizations have to determine appropriate levels of service. Knowing the appropriate level of service for each service provided is fundamental to any business. It provides the business with knowledge of the proper balance between service cost and the service performance.

The District has operated with, for the most part, an informal set of rules regarding how its collection system is operated and maintained. A stable, well-trained and well-managed workforce, known regulations from governing bodies, reasonable reserve capacity, certain guiding principles, and proper levels of automation have all contributed to a collection system that has worked well and provided good quality service to its customers.

One key target level of service is the District's capacity curve for sizing the capacity of its collection system assets. This curve, called the "Madison Design Curve" (MDC), is used to determine whether or not an existing sewer or pumping station is adequate as well as to help determine how large to size a new asset. The average flow is multiplied by a peaking factor from the Madison Design Curve to determine the peak flow capacity requirements for either the existing asset or a new one. Per EPA regulations, sanitary sewer overflows are strictly forbidden even in the event of a flood. Therefore, the MDC has received some attention regarding whether or not it is a conservative enough approach to designing facilities in the Madison area. This will be investigated further and is but one area where the target levels of service may need further review.

The District's informal set of rules has served it well; however, making these rules more formal and defining key performance indicators (KPIs) may be appropriate as workforce turnover increases with the departure of many long-term employees. Although an area in which the District's commission has not typically become involved, formalizing and communicating current levels of service to the District's governing body may provide helpful direction to the staff. Increased levels of service mean increased costs; there are trade-offs that need to be made and risks that need to be taken.

Step 6 – Determine Business Risk ("Criticality")

Risk and criticality are concepts that are used within asset management to help prioritize repair, renewal, or replacement of existing assets and/or installation of new assets. Not all projects can be constructed at the same time; there are financial, physical, and other resource constraints that hinder this. The level of risk or the critical nature of a specific asset can help determine how long the organization can wait to repair, renew, or replace it versus doing something with another asset in the same condition.

Although all of the District's collection system assets were built to serve the fundamental purpose of conveying wastewater to the District's Nine Springs Wastewater Treatment Plant, and all are therefore fundamentally important, some assets are more important than others and some involve higher levels of risk. A method to factor in criticality and risk for the pumping stations was included in the first collection system facilities planning effort and the same system was used for the second effort. Presently, a method to include risk or criticality for the District's sewers is also being developed.

Risk is defined as probability of failure times consequences of failure. The District's present approach to risk has barely scratched the surface of this concept. However, how much could be gained by going into much more detail in this area remains to be seen. Including risk level in decision-making has always been part of the District's approach and a general inclusion and understanding of risk while prioritizing maintenance and projects may provide the appropriate level of emphasis. As with many of the other advanced asset management concepts, this one may require further analysis to determine the appropriate level to meet the District's needs.

Step 7 – Optimize O&M Investment

Most collection system assets are long-lived assets. Therefore, most of them will need some form of maintenance, repair, and/or renewal, and ultimately, they will need replacement. How much maintenance and repair are required and when to renew or replace are not simple questions to answer. Neither is optimizing investments in maintenance, repair, and renewal to provide the lowest life cycle costs while meeting appropriate levels of service. However, that is one of the goals of a good asset management program.

The District's approach to maintenance has changed over the years and it has used a computer-based maintenance system for over ten years. The District continually modifies its approaches to maintenance based upon industry trends and specific pieces of equipment. Further analysis and improvements of the District's maintenance practices will and should continue to optimize the investment in its assets and in its maintenance resources and practices.

Step 8 – Optimize Capital Investment

All utilities should optimize their capital investments. To optimize its capital investments, a utility must make sure that its capital investment decisions include the right solutions at just the right time. Capital investments in the wastewater industry are generally significant long-term infrastructure investments with significant long-term consequences. Therefore, the decisions cannot be approached lightly. Much thought and evaluation need to go into the decision-making process to make wise and cost-effective decisions.

The District, like other utilities, must use all of its assets wisely and optimize its capital investments. Its collection system facilities plan is a prime example of how it approaches investments in its collection system capital wisely and with cost-effectiveness in mind. Projects are prioritized based upon need and follow-up planning and pre-design further investigate the need and best approach to meeting the intended purpose. The following techniques are best management practices for optimizing capital investment (taken from EPA's fundamentals of Asset Management):

1. Build a strategic CIP "Business Plan"
    - Includes project identification, validation, prioritization, and financing
    - Asks the following questions:
        1. What are we going to do and why?
        2. What will it cost?
        3. How will it be funded?
        4. Life cycle impact on level of service, rates, and financial condition
    - Essentially – Are these the right projects at the right time and at the right cost?

2. Deliver the project on time and on budget
    - Includes execution and control
    - Addresses the following areas:
        1. Managing costs
        2. Managing schedules
        3. Managing contracts and changes

3. Integration into the portfolio of assets
    - Includes handover
    - Addresses the following areas:
        1. Registry
        2. Start-up, shake-down, burn-in, commissioning
        3. Manuals, spares, and service
        4. Initiating the maintenance regimen

In general, the District's approach to capital investment covers all of these areas relatively well. That does not mean that the approach to any one technique could not be improved; however, all of the areas are covered and the District continually strives to improve how it performs them.

The District's collection system facilities plan begins the process of building the strategic business plan and initial justification for the project. The pre-design and design phases further analyze alternatives and evaluate whether or not the project is the right project at the right time. The bidding process sets the initial costs and provides a last go or no go decision. During the construction process, proper project management helps keep the project on time and on budget helping determine the final construction cost. Lastly, the turnover to the District's maintenance group integrates the new assets into the District's group of existing assets. Proper O&M throughout the life of the asset ensures that assets operate effectively to control life-cycle costs appropriately.

Step 9 – Determine Funding Strategy

Determining a funding strategy is step 9 in the process of building an asset management plan and the District has a well-developed funding strategy for funding its capital improvements plan. Rather than collect funds that are allocated for replacement, the District borrows money to pay for capital improvements including renewal projects. The District's philosophy behind this approach is that by borrowing, generally at below market rates, the District's customers who continue to use the District's system pay for the improvements while they are using them rather than having those who may or may not benefit from the improvements pay for them ahead of time.

In general, the District derives funds from three separate areas to help pay for capital improvements including borrowing, connection charges, and interest received on the capital account balance. The District takes advantage of State Revolving Fund loans to the extent possible to help fund rehabilitation projects. The District also funds new projects via connection charges for new connections to its collection system. Two separate connection charges are assessed, an interceptor connection charge and a treatment plant connection charge. In the past, connection charges have helped fund collection system expansion as well as fund a certain level of the renewal projects.

Additional funding also arrives in the way of interest derived from the balance in the capital fund accounts; the balance of these accounts should never go below a minimum of three million dollars. Recently, growth has slowed significantly as has the interest received on the capital account balance. Therefore, the District has had to borrow for a greater percentage of its overall capital expenditures. If this trend continues well into the future, the District might have to rethink some of its funding strategy.

Step 10 - Build Asset Management Plan

Step 10 integrates the previous nine steps into an asset management framework and continues to build upon and improve the plan going forward. As stated previously, the District does not have a formal advanced asset management plan; however, the District does use many of the concepts contained in the ten-step process to achieve an asset management plan and utilizes some steps more than others. The intended purpose of the District's collection system facilities plan is the same as an asset management plan: to meet expected levels of service within the District's collection system by managing those assets properly and/or by constructing new assets where necessary. The collection system facilities planning process, like any process, is subject to analysis and improvement. The components, and even the framework of this process, should be reviewed and improved periodically.

Conclusions and Recommendations

The District's Collection System Facilities Plan is an asset management plan. It utilizes some advanced asset management concepts, but has certainly only touched some of them on the surface. Further analysis of advanced asset management concepts may be warranted; however, ultimately, the usefulness of the original collection system facilities plan proves that even without major changes that would include more of these concepts, it provided a useful pathway for the District's engineering staff and capital improvement planning. Each new advanced asset management concept must add a reasonable level of additional value to the plan or it's not worth the additional effort to complete.

Although all advanced asset management concepts are not worth pursuing as part of the District's approach to asset management, the District should at least consider reviewing some of them to a greater extent. The following are recommendations based upon a cursory review of asset management concepts and the District's present practices. Further investigation and analysis is required in most instances.

- In general, become more knowledgeable in advanced asset management concepts and determine which, if any, to integrate into the District's present system of managing its assets.

- Continue to improve asset registry for the District's collection system and the condition assessment of those assets. A good systematic and consistent approach is preferred over one that is overly detailed and cannot be consistently followed.

- Improve methods to estimate remaining asset life, life cycle costs, and replacement costs.

- Review and/or establish written levels of service based upon stakeholder (customers, regulators, and other stakeholders) expectations. Consider presenting these to the District's governing body for review and approval.

- Continue to use methods that include risk and criticality in decision-making to help prioritize maintenance, repair, renewal, and/or replacement. Determine appropriate level of risk analysis to meet the District's needs.

- Optimize and continuously improve the District's maintenance program, repair and renewal methods, and capital improvement planning methods. Integrate these programs and methods to optimize overall asset and process costs.

- Continuously monitor funding strategies for the District's asset management program.

- Continue to monitor and improve the District's approach to managing its assets by building upon and improving existing practices and adding advanced asset management concepts as appropriate.

An asset management plan does not need to include all advanced asset management concepts to be a successful asset management approach. Those concepts that add value to the program should be incorporated into the District's asset management approach; those that do not should not be included. As with any change, it will take time to incorporate these practices into the District's present practices and these should occur over a reasonable timeframe. Advanced asset management and District practices are also likely to continue changing over time and therefore, review of both should continue. The ultimate goal is that the District fully optimizes how it uses its assets and continually searches for and incorporates methods to improve its practices.

<u>Capacity, Management, Operation, and Maintenance (CMOM)</u>

A CMOM program addresses the capacity, management, operation, and maintenance activities of a collection system. It contains many of the same elements that comprise an Asset Management Plan, with greater detail given to certain components. In general terms, a CMOM program consists of best management practices that have been developed by the wastewater industry with consideration given to the entire life cycle of the collection system components. The program helps the owner of a collection system provide a high level of service to its customers while at the same time working to improve regulatory compliance regarding sanitary sewer overflows.

Currently there are no formal requirements by state or federal governments for establishing or implementing CMOM programs. A guidance document for CMOM programs was published by EPA in 2005 to assist owners and operators in management of their collection systems (*Guide for Evaluating Capacity, Management, Operation, and Maintenance (CMOM) Programs at Sanitary Sewer Collection Systems, EPA, 2005).* In August of 2007 the EPA released a document entitled "Model NPDES Permit Language for Sanitary Sewer Overflows" that essentially requires collection system owners and operators to develop and maintain a CMOM program as outlined in its guidance document.

While the proposed revisions to NPDES permits for SSO overflows have yet to be adopted, it is clear that development and adherence to such a program is likely to occur in the near future, and possibly within the planning horizon of this Facility Plan. As a result, this section is provided to: (1). Discuss the major requirements of a CMOM program (as defined by the EPA's guidance document); (2). Summarize how the District's facilities and operations are currently positioned to address each of these requirements; and (3). Provide recommendations for areas that may require further improvement.

Each major component of the EPA's proposed CMOM program will be discussed in turn in the remainder of this section as shown below:

1. Capacity Assurance
2. Management
3. Operation
4. Maintenance
5. Sewer Rehabilitation
6. Conclusions and Recommendations

1) <u>Capacity Assurance</u>

a) General

Capacity of the collection system should be evaluated periodically to evaluate the effects of both dry and wet weather flows on system conveyance. The first step in this evaluation involves an inventory of existing facilities and system features, including service population, total system size, and a characterization of pipe sizes, lengths, materials, and ages. The District's Collection System Database currently stores this information and integrates it with its Geographic Information System (GIS).

The second step in evaluating the capacity of the collection system is a general inspection of the system. This is discussed in more detail later in this subsection. The final step in the capacity evaluation involves identifying those areas of the collection system that are prone to capacity limitations in the form of wet weather related SSO's, surcharging, or basement backups. Those areas that are identified should be investigated more fully using techniques such as flow and rainfall monitoring and hydraulic modeling.

The District's *Collection System Evaluation (2009)*, as prepared by CARPC, will be a useful tool in identifying areas with capacity limitations by comparing system capacities against projected peak flowrates for each section of the collection system. The District has also used its recently acquired hydraulic model to further analyze areas of the collection system where capacity limitations have been identified by CARPC's analysis. The results of this investigation are discussed in more detail in Chapter 4 and Appendix 5.

b) Inspection Techniques

i) Flow Monitoring

Flow monitoring is used to collect fundamental information about the collection system, including dry weather flowrates and estimates of inflow and infiltration. The District employs a full-time crew to monitor flowrates from its satellite communities for billing purposes. Most in-line monitoring is done through weir measurements in manholes. This information is occasionally useful for establishing dry weather flowrates in District facilities, but it has limited applicability due to the short duration of the monitoring and due to the location in the system in which the monitoring is conducted. The District does not own any area-velocity or ultrasonic meters that are better suited for measuring flows in larger sewers. The District typically contracts with a consultant for flow monitoring in larger sewers for extended periods of time as part of flow and/or I/I studies. The District may want to consider investing in one or more meters if future I/I studies provide beneficial results and prove cost effective.

ii) Sewer System Testing

Leaks in the collection system are commonly identified through smoke testing or dyed water testing. Both of these techniques are used on a periodic basis when excessive I/I or a storm water cross connection is suspected in a portion of the collection system. Smoke testing is done by plugging each end of the test section, introducing smoke into the section via a blower, and recording those locations in the test section where smoke escapes. In a properly operating system the smoke should escape from the plumbing vents of adjacent buildings. In a leaky system the smoke will also escape from the ground at points along the sewer main or sewer laterals.

Dyed water testing is used to confirm the connection of a fixture or appurtenance to the sanitary sewer system. It is often used in conjunction with smoke testing to validate the results. The District has employed occasional use of both sewer system testing techniques, but would not likely have a routine need for either that would warrant additional investment.

iii) Sewer System Inspection

Visual inspection of manholes and pipelines is used to identify existing or potential problem areas that may limit capacity. Various defects in the pipeline can be identified and recorded such as root intrusion, corrosion, grease accumulation, and joint offsets. A variety of techniques for sewer inspection are available. They include lamping, camera inspection, sonar, sewer scanner, and closed circuit television (CCTV). The District aims to televise each segment of the collection system no less than once every ten years by contracting with a sewer cleaning and televising contractor. The use of CCTV by this process has served the District well in the past and should continue to do so in the future.

2) <u>Management</u>

Proper management of a collection system is crucial to the operation and management activities. The EPA's guidance document cites six important goals of a management program:

- Protection of public health and property
- Minimization of I/I and capacity assurance
- Prompt response to service interruptions
- Efficient use of funds
- Identification and correction of system deficiencies
- Safety

In order to achieve these six goals, a good management program should contain a strong focus on the following elements: organizational structure, training, internal communication, customer service, management information systems, a SSO notification program, and a clearly defined legal authority. Each of these areas will be discussed briefly in turn.

a) Organizational Structure

A well-defined organizational structure helps to delineate responsibilities and authority for each position in the collection system. This typically includes the use of an organizational chart, position descriptions for each employee, or both. The EPA recommends that vacant positions and work that is contracted out also be accounted for in the organizational chart. It is also recommended that one supervisor have overall responsibility for the collection system.

The District has a well-defined, overall organizational chart that is kept current and includes positions related to work in the collection system. Position descriptions for each employee have also been added to the organizational structure recently to help clarify job responsibilities and expectations. The District may want to consider developing an organizational chart specific to the collection system that shows contracted work responsibilities such as sewer cleaning and televising.

b) Training

While employee training is not explicitly required under current regulations, it is an important element of a collection system with regard to safety and regulatory compliance. The EPA recommends that training be provided in the following areas for collection system personnel:

- Routine line maintenance
- Confined space entry
- Traffic control
- Record keeping
- Pump Station O&M
- Electrical and instrumentation
- Public relations and customer service
- SSO/Emergency response

The District has a Training and Safety Manager on staff and a well-established safety program that addresses most of the areas identified above. Confined space entry policies are recorded in a written handbook and training is conducted for all affected personnel on an annual basis. A permit program for all entries is also in place. Operational and maintenance training for mechanics and electricians on all new or rehabilitated equipment at pump stations is routinely conducted by District staff or equipment suppliers.

While the District has prepared and periodically updates an emergency response manual that addresses SSO’s, no formal training for employees is currently conducted. Written procedures for identification, clean-up, and notification of SSO's should be considered by the District in addition to employee training on these items. In addition, the District should consider offering formal training related to proper traffic control procedures for conformance to local road and state highway requirements.

c) Internal Communication

Effective communication requires the exchange of ideas and information amongst staff. The EPA's guidance document references the use of bulletin boards, regular staff meetings, e-mail, and employee incentive programs to promote effective communication. The District currently employs each of these communication tools as a way to exchange ideas between staff members.

d) Customer Service

Work in this area involves addressing all comments, questions, requests for information, and complaints from the public in a timely manner. This area also extends to the development of a public relations program that educates the general public, public officials and local utilities about the collection and treatment of wastewater.

The District provides wastewater conveyance and treatment to satellite communities of varying size. Thus, most of the District's customer service involves municipal officials at the town, village, or city level. In general the District's customer relations with these entities are very good. In the last year the District has worked to strengthen its public relations program. It recently contracted with a media relations company for radio advertisements promoting water conservation and I/I reduction efforts. In addition, the District recently completed a 50-year Master Planning effort and held extensive public meetings throughout the planning process to educate stakeholders about the Plan.

e) Management Information Systems

The collection, maintenance, and retrieval of data for collection system operations are important tools for system management. A good management information system improves preventive maintenance on equipment, allows work orders to be tracked more efficiently, and aids in preparing and justifying capital budget expenditures. The trend in the industry has been to use computer-based systems to manage data. For several years the District has used a computer-based Maintenance Management System (CMMS) to track the performance of assets in the collection system. Among other things, it is used to document problems and generate work orders, schedule routine maintenance activities, maintain equipment inventories, track costs, and create purchase orders.

The District has also developed a computerized database of its collection system for all pertinent physical characteristics. This database is used in conjunction with a Geographic Information System for locating and mapping of its facilities.

f) SSO Notification Program

A written procedure should be developed for all entities that could be affected in the event of an SSO. This includes the public, public health officials, and any regulatory authorities. The procedure should indicate the different agencies to be notified as well as contact information and responsibilities for all personnel involved.
The District currently notifies the Wisconsin Department of Natural Resources for each sewer bypass or sewer overflow event. It also works directly with public health officials to notify the public in the rare cases where overflows occur to surface waters. Contact information for these agencies is currently in the District's *Emergency Response Manual*, although specific written procedures are not included. The District should revise the manual to clarify the procedures to be used for SSO notification.

g) Legal Authority

This section deals with the regulation of flow that enters the collection system from residential, commercial, and industrial sources. The legal authority for this regulation can be in the form of a sewer use ordinance, contracts, service agreements, or some other legally binding document. Included in this authority is a pretreatment program to prevent the discharge of materials into the collection system that would interfere with the conveyance or treatment operations. This legal authority should also extend to include general prohibitions, grease control requirements, restrictions on stormwater inflow and infiltration from laterals, and new construction standards.

The District's Sewer Use Ordinance, along with its pretreatment program, provides the legal authority to regulate most of the items described above. Among other regulations, it provides standards for new connections, restricts clear water and storm water flows, and prohibits grease discharges. The District's pretreatment procedures are prepared in a written document and approved by WDNR on a periodic basis. These procedures specify sampling requirements and procedures and sets limits on constituents in wastewaters discharged from non-domestic sources.

With regard to the issue of excess flows from satellite communities, the District continually evaluates the effects of large rainfall events on the collection system and works with its customers to identify and correct problem areas. This approach has worked with success in the past. As such, the District currently does not employ the use of contracts, agreements, or allocations to regulate excess flows from its satellite communities. However, large storm events have increased in intensity and frequency over the last ten years and may cause the District to consider executing agreements for excess flow allocations in the future. Significant expense would be incurred by the District to enforce the monitoring requirements for such a program given the large number of customers served by the District as well as their geographic layout. The costs for this monitoring effort would need to be balanced against the costs needed to reinforce the District's conveyance facilities to accommodate larger wet weather flows. Non-economic factors should also be considered in this evaluation.

3) Operation

Collection systems have limited operability options relative to wastewater treatment plants as there usually is only one route for the wastewater to travel from the source to the plant. There are many factors to consider, though, with regard to operational activities of the collection system.

a) Budgeting

Budgeting is one of the most important components of a CMOM program. Inadequate funding makes achieving operational goals difficult. One way to avoid inadequate funding is to develop a consistent annual baseline for operating costs and to track expenditures closely. Costs of preventive and corrective maintenance and major repairs for the collection system are key components of the annual operating budget. An owner may develop a separate Capital Improvement Plan (CIP) to complete small projects (one to two year cycles) or larger projects (three to five year cycles).

The District prepares and adopts annual budgets for operational expenses and capital projects. The primary source of revenue to cover these expenses comes from service charges collected from the District's satellite communities. As mentioned previously, the District uses a CMMS system to track its annual operating expenses and also projects a 10-year Capital Improvement Plan. As a result, increases in service charge rates are generally consistent and average approximately 5% per year. No further changes in budgetary practices are anticipated to meet CMOM program requirements.

b) Monitoring

Monitoring of wastewater discharges in the collection system may be done by the owner for a variety of reasons. These include monitoring of industrial users for permit compliance, monitoring of satellite communities for billing purposes, monitoring receiving waters to assess SSO effects, and monitoring required for NPDES permit compliance. The EPA guidance document recommends that written procedures be developed to ensure that sampling is done in a safe, effective, and consistent manner. This document should include key items such as instructions for sampling and field monitoring and laboratory procedures for analysis.

The District employs one full-time crew for monitoring and sampling of wastewater throughout the collection system. The majority of the crew's time is devoted to quarterly sampling and monitoring of flows from satellite communities for determination of service charges. The crew also performs monitoring on a limited number of industrial users, although many of these users do their own monitoring. While the District's pretreatment program does contain some written procedures related to sampling and monitoring requirements (i.e. sample volumes and frequencies), the District may want to consider developing a more detailed procedure that contains all of the elements referenced in the EPA's guidance document.

c) Hydrogen Sulfide Monitoring and Control

Hydrogen sulfide gas can collect in various parts of collection systems and react with bacteria to form sulfuric acid, which can corrode metal and concrete surfaces. The EPA's guidance document recommends that a program be developed to monitor areas of the collection system which may be adversely affected by the presence of hydrogen sulfide.

The District performs routine manhole and sewer line inspections as part of its televising program. The condition of the manholes and sewers due to corrosion is recorded on inspection forms, although pH readings in the system are not generally taken. Acquiring pH readings in manholes in vulnerable parts of the collection system may be something the District wants to consider in its inspection program going forward.

The District has also addressed the issue of hydrogen sulfide control in specific parts of its collection system due to odor complaints or observations from operations staff. The addition of chemicals to reduce the level of sulfides has been studied but not implemented as a long-term solution. Other chemicals have been used to "mask" the odors caused by sulfides. The best operational strategy to eliminate problems due to hydrogen sulfides is to select materials of construction that are resistant to corrosion (i.e. PVC, fiberglass). The District has elected to use these pipe materials as its standard on new or rehabilitation projects over other materials such as concrete and steel.

d) Safety

Safety programs define the standards under which the work is to be accomplished and to make employees aware of safe working procedures and specific regulations. The safety program should be established in writing with respect to specific procedures and policies.

The EPA's guidance document recommends that safety programs be enacted for the following areas related to collection systems:

- Confined spaces
- Chemical handling
- Trenching and excavations
- Material Safety Data Sheets
- Biological hazards in wastewater
- Traffic control and work site safety

- Lockout/Tagout
- Electrical and mechanical safety
- Pneumatic or hydraulic systems safety

The District holds weekly safety meetings for all employees that deal with most of the items listed above. Material Safety Data Sheets are readily available to all employees for materials which are routinely used in District operations. While clearly defined procedures and policies have been developed for some of the items such as confined spaces, more written documentation could be provided for some of the other areas.

e) Emergency Preparedness and Response

Comprehensive plans should be in place for handling both routine and catastrophic emergencies. Examples of routine emergencies include overflowing manholes, sewer main breaks, localized electrical failures, and power outages at pumping stations. Catastrophic emergencies include extreme events such as floods, tornados, earthquakes, chemical spills, and widespread electrical outages.

The District has prepared, and updates on an annual basis, its *Emergency Response Manual* to address emergency situations. Among other information, it provides procedures to be followed during pump station outages, information related to repair of force mains, and contact information related to sewer overflows and other types of spills. This manual is made available to each employee in written form and on the District's internal website.

In addition, the District is in the preliminary stages of preparing a risk-based condition assessment for its collection system. This assessment will account for risk factors such as facility age, material, depth, location, and criticality in order to assess the risk of failure of each component and aid in prioritizing future rehabilitation projects.

f) Modeling

Sewer system modeling is done to help simulate non-uniform and unsteady flows throughout the collection system in response to different operating conditions and rainfall events. It can be a valuable tool in new designs and in evaluating different operating scenarios.

The District developed a hydraulic modeling tool for its collection system in 2005. It has been used primarily for evaluating capacity based on existing flows and future flow projections. The hydraulic model is described in more detail in Chapter 3 of this Plan.

g) Mapping

The creation and maintenance of good mapping records is crucial to the effectiveness of a collection system. The EPA's guidance document specifies that the following information should be included at a minimum: sewer mains, laterals, manholes, cleanouts, force mains, pump stations, service area boundaries, and other landmarks. The District maintains all the physical characteristics described above in its collection system database and maps these features using its Geographic Information System. Aerial photography is included in the mapping to aid in the location of facilities. Map books are updated on a regular basis to incorporate system modifications and mapping improvements.

h) New Construction

This section calls for the strict control and regulation of flows into the collection system from new construction. This includes both public and private sewers. The owner should adopt standards for new construction and procedures for the review of proposed extensions.

The District specifies standards for plan review, construction, inspection, and testing of new connections through its Sewer Use Ordinance. Proposed sewer extensions are reviewed by District staff, a county regulatory agency for conformance to area water quality plans, and the Wisconsin Department of Natural Resources. The District's review ensures that the collection system has adequate capacity to serve the proposed extension and that the proposed construction materials are adequate.

i) Pump Stations

Pump stations vary in their type, size, and complexity and require differing levels of specialized mechanical, electrical, and hydraulic knowledge. Failures can lead to equipment and environmental damage, or even endanger public health. The District owns and operates 17 regional pumping stations and employs its own electrical and mechanical maintenance staff to maintain and repair equipment.

4) <u>Maintenance</u>

Collection system owners should develop well-planned, systematic, and comprehensive maintenance programs which incorporate the following goals:

- Prevention of overflows
- Maximization of service and system reliability at minimum cost
- Assurance of infrastructure sustainability

Maintenance activities can be broadly classified as planned or unplanned. Planned maintenance includes both predictive and preventive measures, which aim to treat operational problems prior to equipment failure. Unplanned maintenance consists of corrective or emergency measures which are used to repair equipment once it has failed.

Proper maintenance programs should incorporate the various elements discussed further in this subsection.

a) Maintenance Budgeting

Maintenance costs can be a significant part of the annual operating budget. As such, these costs should be closely tracked throughout the year to ensure that future budgets have appropriate funding.

The District's maintenance costs are included in its annual operational budget. As mentioned earlier in the discussion of operational budgets, the District employs the use of a CMMS system to track operational and maintenance costs. This system has served the District well and no changes to this system are recommended at this time.

b) Planned and Unplanned Maintenance

i) Predictive Maintenance

Planned maintenance involves a systematic approach to maintenance activities such that equipment failure is avoided. As mentioned previously, this includes both predictive and preventive maintenance. Examples of predictive maintenance include equipment inspection and monitoring equipment for early warning signs of failure such as vibration, heat, dirty oil and leakage. Recording and storing the data obtained from inspection and monitoring activities is a key component of predictive maintenance.

The *2002 Collection System Facilities Plan* recommended development of a predictive maintenance program for pumping equipment in the collection system. The District has implemented this recommendation in its rehabilitation of Pump Stations 1, 2, 6, 8 and 10 through the installation of pump vibration sensors. In addition, the District recently purchased a thermal imaging scanner to detect unusual heat patterns or temperature changes in electrical equipment (i.e. motor control centers and control panels) as an indicator tool for impending electrical failure of the equipment. The goal is to scan each piece of equipment to develop a baseline for future comparison so that any problems can be corrected before equipment failure. One challenge of this thermal imaging program is to develop an efficient way to store all of the information that is obtained from the scans and link it to the District's asset management software. This is an area that will require further study and work.

ii) Preventive Maintenance

Preventive maintenance aims to reduce equipment breakdowns, improve system reliability by minimizing equipment outages, lengthen equipment life, and avoid potential noncompliance situations. An effective preventive maintenance program should contain the following elements:

- Trained personnel
- Scheduling based on system specific knowledge and manufacturer’s recommendations
- Detailed instructions related to the maintenance of various pieces of equipment
- System for recordkeeping
- System knowledge in the form of maps, historical knowledge and records

A maintenance record for each piece of equipment should be maintained which contains information related to maintenance recommendations, schedule, instructions, and past maintenance history.

The District’s CMMS is used to store and track information on all District assets at the treatment plant and at pumping stations in the collection system. This includes equipment specifications, bill of materials, maintenance schedules, and other related maintenance materials. The District typically requires and receives an Operating and Maintenance Manual from the manufacturer for each new or rehabilitated piece of equipment in the collection system. This information is used to generate schedules and instructions for preventive maintenance items. An asset identifier for each gravity sewer segment in the collection system has recently been added to the District’s CMMS.

Other examples of predictive maintenance activities performed by the District include biweekly inspections of its 17 regional pumping stations, periodic inspection and cleaning of air release valves on force mains, and lubrication of equipment at pumping stations. Air release valves have historically been inspected and cleaned as necessary. Due to repeated problems with plugging of these valves, the District recently began a program to inspect and clean these valves no less than twice a year.

Pump station inspections include starting and stopping each pumping unit to check for vibration or plugged vent lines and documentation of other items that may require corrective maintenance. In addition, the District employs one full-time lubrication mechanic to ensure that all pumping equipment is greased according to the manufacturer’s schedules.

On an annual basis inspections of all the District’s pumping stations are made by the Director of Operations and Maintenance to identify and document large repair items that may be outside the scope of routine work orders. These items are prioritized and inserted into the Capital or Operational budgets as appropriate.

iii) Corrective Maintenance

Maintenance of this type can occur as a result of predictive or preventive maintenance activities which identify a problem. In these instances a work order is generally issued to the proper personnel for repair as soon as a problem is identified. Maintenance of this type usually results in the equipment being taken out of service for a period of time and reduces redundancy in the system.

The District's CMMS is used to generate, store, and track all work orders that pertain to corrective maintenance. Lengthy service disruptions are minimized through use of the CMMS by the ability to easily review open work orders.

iv) Emergency Maintenance

Emergency maintenance requires immediate attention and repair of a problem to avoid equipment failure or threats to public health or the environment. In large collection systems this may require emergency crews to be available at all times throughout the year, while smaller systems may utilize an "on-call" system. Written procedures should be in place to outline actions to be taken and the equipment needed for emergency situations.

The District has prepared, and updates on an annual basis, its *Emergency Response Manual* for responses to emergency events. This document deals with situations such as repairs to force mains, outages at pumping stations, emergency spills (including SSO's), and contact information for contractors, satellite communities, and regulators. For emergency events such as force main breaks, the District usually hires a contractor to excavate and make repairs.

c) Sewer Cleaning

Sewer cleaning removes accumulated material from the sewer and helps to prevent blockages and prepare the sewer line for televising. The key to an effective sewer cleaning program is recordkeeping. Not all areas of the collection system need to be cleaned at the same frequency. For example, those parts of the system with a high density of restaurants may need to be cleaned every six months, while a residential area with new pipe may not require cleaning for several years. An owner should be able to identify problem areas in the system and show how the preventive maintenance schedule addresses these areas. In addition, an owner should be able to document the number of stoppages experienced per mile of sewer.

The District does not clean sewers with its own forces. All sewer cleaning is contracted out on an annual basis under one contract. In general all sewers are cleaned no less than once every ten years, with any problem areas receiving more frequent attention. Due to the larger pipe sizes and magnitude of flows in the District's sewers compared to local sewers, this frequency of cleaning has found to be adequate based on past experience. The District links its sewer cleaning and televising operations and manages them through a computerized database. While the database has proved to be a useful tool, challenges have been noted with regards to development of a scoring and rating system for sewer condition and with the reporting of these scores for use in scheduling cleaning operations and repair or rehabilitation projects. The District should continue to develop the database to refine these areas.

d) Parts and Equipment Inventory

Spare parts, equipment, and supplies should be kept in inventory to keep equipment from being placed out of service for long periods of time after breakdown or malfunction. Inventory should be based on the equipment manufacturer's recommendations as well as the owner's past experience.

The District's Purchasing Manager is responsible for overall management of inventory for equipment used in the collection system, with assistance from the mechanical and electrical maintenance departments. Information regarding inventory is stored and tracked via the District's CMMS. Sign-out procedures for parts are in place for replenishing inventory. No changes to the District's inventory practices are recommended at this time.

5) Sewer Rehabilitation

The owner should develop a sewer rehabilitation program to incorporate the results of the capacity assurance, management, operation, and maintenance activities. Sewer rehabilitation helps to ensure that the collection system remains viable by: (1). Maintaining structural integrity; (2). Limiting the loss of conveyance; and (3). Controlling the rate of exfiltration from the pipe network to protect groundwater. The sewer rehabilitation program should clearly indicate how projects are prioritized and how rehabilitation methods are selected (i.e. open cut vs. trenchless construction).

The District currently does not have a formal sewer rehabilitation program. Projects are currently identified as a result of periodic capacity analyses or condition reports. The decision on the type of repair method to be used is generally made based on facility planning or pre-design reports. The District has completed a number of sewer lining projects in the last 3-4 years and has found them to be a cost-effective tool to prolong the service life of sewers in certain applications. As this technology evolves and improves and the District's collection system ages and grows, the District may want to consider a more formalized approach for identifying rehabilitation projects and construction methods.

As mentioned in the Emergency Preparedness and Response section for Operational activities, the District recently began development of a risk-based condition assessment tool to help identify and rank the most critical portions of the collection system. Continued development, refinement, and use of this tool with other data regarding the collection system are recommended to help prioritize future rehabilitation projects.

6) Conclusions and Recommendations

After reviewing the key elements and requirements for a CMOM program as found in the EPA's guidance document, it appears that the District is well positioned in the event that the program gets enacted. The District currently implements many facets of the program in its current operation of its collection system. Recommendations for improvements to the collection system have been discussed in the preceding sections. These recommendations are summarized by section below:

- Capacity Assurance
  - Consider purchase of flow metering equipment for I/I studies.

- Management
  - Develop an organizational chart specific to the collection system. Indentify all contracted work in the structure.
  - Develop a written procedure for SSO events. This should include procedures for identification and clean-up of overflows and notification requirements.
  - Offer or conduct training program for traffic control procedures.
  - Consider use of service agreements or contracts with satellite communities to regulate wet weather flows and I/I into District's collection system.

- Operation
  - Develop written rules and procedures for monitoring of wastewater.
  - Acquire pH readings in manholes as part of hydrogen sulfide monitoring program.
  - Develop and assemble a written safety program relating to collection system work areas.

- Maintenance
  - Develop a system to link thermal imaging scans for predictive maintenance to equipment asset information in CMMS.
  - Refine District televising database to improve scoring and ranking system. Incorporate the scheduling of cleaning and televising operations into database.

- Sewer Rehabilitation
  - Develop a risk-based condition assessment model to aid in prioritizing sewer rehabilitation and replacement projects.

The District will also need to consider the format of its CMOM document. At present the required information can be found in several separate locations (i.e. Geographic Information System, Collection System Facilities Plan, Emergency Response Plan, etc.). The District will need to consider the advantages and disadvantages of compiling all of this information in one central location and/or document.

Chapter 3
Progress since Original CSFP was Developed

Chapter Outline

This chapter is organized into the following sections:

- Status of recommended projects in *2002 Collection System Facilities Plan*
- Screenings and Solids Handling Update
- Hydraulic model

Status of recommended projects in *2002 Collection System Facilities Plan*

The *2002 Collection System Facilities Plan* has served as a useful guide for the District in identifying, prioritizing, and implementing improvements to the collection system over the past ten years. A list of recommended projects was included in Chapter 7 of the original facilities plan spanning four different periods of time (Table 7.1). A condensed listing of these projects is shown in Table 3.1 to show the current status of the recommendations up to the year 2010. Of the 52 projects recommended for completion between 2000 and 2010, 48 have been completed or the project was under construction as of 2010. Table 3.2 is a brief summary of the recommended projects that have yet to be completed and their current status.

Table 3.2 – Status of Uncompleted Projects from *2002 Collection System Facilities Plan*

Project	Status	Projected Completion
New PS 18	Facility planning starting in 2011	2015
PS 18 – New forcemain	Facility planning starting in 2011	2015
PS 10 – I/I study	Pending	-
PS 14 – I/I Study	Recommended per CSFP Update	2011-2012

Based on Tables 3.1 and 3.2, all collection system projects recommended in the 2002 Plan, with the exception of the inflow and infiltration study for the PS10 service area, will be completed by 2015. The need and scope for an I/I study in the PS10 service area requires further study and prioritization relative to other areas in the collection system.

Table 3.3 provides a summary of the major improvement projects that have been completed in the collection system from the year 2000 to 2010. At least one project was

Table 3.1 - MMSD Collection System Projects Completed From 2000 to 2010

Table 7.1 - MMSD Collection System Facilities Plan (2002)
MMSD Collection System Projects
Approximate Timetable and Costs

Project	Project Completed?	Cost Estimate (Year 2000 dollars) Period A 2000 - 2005	Period B 2006 - 2010	Primary Need: Hydraulic Capacity	Primary Need: Physical Condition	Comments
System Wide Projects						
Telemetry System Modifications	x	$ 100,000				Majority of work completed in 2000
Predictive maintenance program for pumps	x					To minimize risk of equipment outages
Collection System Dynamic Modeling	x	$ 250,000				A tool for analysis of high flows vs. time
Pumping Station No. 1 Service Area						
Crosstown FM Repl. at Monona Terrace	x	$ -			x	1,050-ft. project, completed in 1995.
Crosstown FM Replacement at Yahara River	x	$ 500,000		x	x	1,330-ft. project, completed in 2000.
Crosstown FM Replacement to PS2	x	$ 5,000,000		x	x	14,400-ft. E. Wash. Ave. to PS2, 2002.
Burke Outfall Replacement	x	$ 2,500,000			x	5,000-ft. Commercial Ave. to First St.
PS1 Major Rehab.	x	$ 3,000,000		x	x	PS1 (1950) will be approx. 55 years old
North End Interceptor Replacement	x	$ 300,000		x	x	1,700-ft. along Commercial Ave.
Pumping Station No. 2 Service Area						
PS2 Force Main Replacement-Phase I	x	$ 2,000,000		x	x	NSWTP to Van Duesen St., 2000
PS2 Force Main Replacement-Phase II	x	$ 2,500,000		x	x	Van Duesen St. to PS2, 2001
PS2 Major Rehab. incl. capacity revisions	x	$ 3,000,000		x	x	PS2 (1963) will be approx. 50 years old
SWI W. Shore Drive Replacement	x	$ 400,000		x	x	1,700-ft included with PS2FM Phase II
Pumping Station No. 3 Service Area						
Pumping Station No. 4 Service Area						
South Int. Baird Street Rehabilitation	x		$ 300,000		x	1,500-ft. VCP from 1928 (lined in 2009)
Install Second Power Feed	x	$ 100,000			x	
Pumping Station No. 5 Service Area						
Pumping Station No. 6 Service Area						
Short term electrical improvements	x	$ 50,000			x	
PS6 Major Rehab	x		$ 3,000,000		x	PS6 (1950) will be approx. 60 years olc
Pumping Station No. 7 Service Area						
NEI Replacement at Buckeye Road	x	$ 150,000		x	x	FM, MH & 186-ft of 48", with City road project.
NEI Relief from PS10FM to FEI junction	x	$ 2,500,000		x	x	7,400-ft. of 30"-42" needs relief
New PS18			$ 5,000,000	x		For future growth and reliability
New PS18 Forcemain			$ 7,000,000	x	x	From new PS18 to NSWTP
Door Creek - Gaston Road Extension	x		$ 200,000			Extension to cross Interstate 94 at Gaston Road
Pumping Station No. 8 Service Area						
West Interceptor Campus Relief - Phase 1	x	$ 600,000		x	x	1,147-ft. Randall Ave to Matls.Science Bldg.
West Interceptor Campus Relief - Phase 2	x	$ 900,000		x		700-ft. University Ave. Tunnel
West Interceptor Campus Relief - Phase 3	x	$ 50,000		x	x	1,101-ft. behind Babcock Hall & Stock Pav.
West Interceptor Campus Relief - Phase 4	x	$ 1,300,000		x	x	2,600-ft. UW Ag. Campus to Walnut Street
SWI Relocation for Home Depot	x	$ -				Completed in 2000 for new buildings.
SWI South Leg Relief	x		$ 800,000	x		4,322-ft from Home Depot to SWI junction
SWI North Leg Relief	x		$ 1,100,000	x		5,639-ft of 15"-18" may need relief
SWI Rehab at Chippewa Drive	x	$ 100,000			x	1,387-ft of 12" VCP rehab for I/I
Power System Modifications	x	$ 50,000				For added power supply redundancy
PS8 Major Rehab	x		$ 3,000,000	x	x	PS8 (1964) will be approx. 45 years old
Pumping Station No. 9 Service Area						
I/I Study	x			x		Monitoring study completed 1998
Second power feed	x	$ 100,000				
Pumping Station No. 10 Service Area						
I/I Study		$ 50,000		x		
NEI Relief d/s of Lien Interceptor junction	x		$ 2,000,000	x		6,600-ft of 48" d/s of Lien Int. will need relief
NEI Relief u/s of Lien Interceptor junction	x		$ 800,000	x		2,600-ft. of 36"-42" u/s of Lien Int. may need relief
PS10 Major Rehab.	x	$ 3,000,000		x	x	PS10 (1964) will be 40 years old. Operating new peak capacity must wait for d/s NEI gravity relief.
Pumping Station No. 11 Service Area						
NSVI Nicolet Replacement	x	$ 300,000			x	Completed in 2000
PS11 Firm Capacity Improvements	x	$ 200,000		x		

Project	Project Completed?	Cost Estimate (Year 2000 dollars) Period A 2000 - 2005	Period B 2006 - 2010	Primary Need: Hydraulic Capacity	Physical Condition	Comments
Pumping Station No. 12 Service Area						
I/I Study	**x**			x		Monitoring study completed 1998
Control system modifications	**x**	$ 50,000				
PS12 Firm Capacity Improvements	**x**	$ 200,000		x		
Pumping Station No. 13 Service Area						
I/I Study	**x**	$ -		x		Will be performed by City of Madison
NEI Rehab west of Airport	**x**		$ 300,000		x	1,500-ft. of 48" RCP rehab
PS13 Firm Capacity Improvements	**x**	$ 200,000		x		
Pumping Station No. 14 Service Area						
I/I Study		$ 50,000		x		
PS14 Firm Capacity Improvements	**x**	$ 200,000		x		
Pumping Station No. 15 Service Area						
West Int. Extension Replacement	**x**		$ 750,000	x	x	3715-ft. 24" & 18", Century Blvd. to Allen Blvd.
Pumping Station No. 16 Service Area						
West Int. Gammon Ext. Relief	**x**	$ 750,000		x		2,000-ft. on Voss Pkwy. and Fortune Dr. was built. ~1,200 ft on Middleton St. not built.
PS16 Control Improvements	**x**	$ 50,000			x	
Pumping Station No. 17 Service Area						
Lower Badger Mill Creek Interceptor to PS17	**x**		$ 3,000,000	x		Phases I & II built (PS17 to Cross Country Road). Remainder of route to Midtown Road not built.
Total Projects		$30,500,000	$27,250,000			

Table 3.3
IMPROVEMENTS TO MMSD COLLECTION SYSTEM: 2000-2010

	PROJECT NAME	Project Description	Approx. Year Completed
System-wide Improvements			
	Telemetry system upgrade		2000
	Dynamic Model	Computer model of MMSD collection system for continuous flow simulation.	2005
Pump Station No. 1 Service Area			
	Crosstown FM: PS#1 to East Wash.	New 24" FM from PS#1 to East Wash Ave	2000
	Crosstown FM: PS#1 East Wash to PS#2	New 30" FM from East Wash Ave to PS#2	2002
	North Basin Interceptor	New 42" & 36" from 1st St. to Commercial; New 18" & 20" from Sherman to Commercial	2002
	PS#1 Rehab	X-Town pumps removed. A&B removed and new X-Town pumps installed. C&D remain. All VFD.	2005
Pump Station No. 2 Service Area			
	PS#2 FM Replacement	New 36" FM replaced existing 30"	2001
	PS#2 FM Changes at NSWTP	Extension of PS#2 FM to new Headworks Building during the 10th Addition	2005
	PS#2 Rehab	All 4 pumps replaced (all same size: 2 constant speed & 2 VFD)	2005
	SWI-Shore Drive Replacement	Approx 1,700 LF of 24" replaced with 36"	2001
	WI-Spring St. Relief Replacement @ Park St.	Approx. 155 LF of 24" replaced with 24" at Park Street crossing	2006
Pump Station No. 3 Service Area			
	Impeller Mods (due to PS#2 changes)	Larger impellers (both pumps) installed by the O&M Department	2005
Pump Station No. 4 Service Area			
	Impeller Mods (due to PS#2 changes)	Larger impellers (pumps B & C only, not A) installed by the O&M Department	2005
	South Interceptor - Baird Street Extension Lining	Cured-in-place liner installed in approximately 1,400 feet of 15" pipe	2009
	Power Feed Redundancy	Install second power feed.	2006
Pump Station No. 5 Service Area			
	None		
Pump Station No. 6 Service Area			
	Pump #6C Retired	Motor on pump 6C failed and not replaced. Impacts firm capacity	2006
	Pump Station Rehabilitation	Four new pumps, related electrical and control work, and new HVAC system.	2010
Pump Station No. 7 Service Area			
	NEI-Pflaum Road Replacement	Approx. 7,400 LF of new 36"-54" from Buckeye Road to FEI Junction	2005
	PS#7 FM Changes at NSWTP	Extension of PS#7 FM to new Headworks Building during the 10th Addition	2005
Pump Station No. 8 Service Area			
	WI Campus Relief-Phase 1	Approx. 1,150 LF of 36" Relief Sewer-Randall St. to Metallurgy Bldg.	2000
	WI Campus Relief-Phase 2	Approx. 700 LF of 36" Tunnel from Metallurgy Bldg. to Babcock Hall	2001
	WI Campus Relief-Phase 3	Approx. 1,100 LF of 36" Relief Sewer-Babcock Hall to Stock Pavilion	1999
	WI Campus Relief-Phase 4	Approx. 2,700 LF of 36" Relief Sewer-Stock Pavilion to Walnut Street	2004
	SWI-MH02-163 to MH02-167 Liner	Approx. 1,390 LF of 12" VP lined with CIPP	2001
	SWI-North & South Legs Liner	Entire length of North & South Legs lined (with CIPP)	2007
	PS#8 FM Changes at NSWTP	Extension of PS#8 FM to new Headworks Building during the 10th Addition	2005
	Pump Station Rehabilitation	Four rebuilt pumps, related electrical and control work, and new HVAC system.	2010

Table 3.3
IMPROVEMENTS TO MMSD COLLECTION SYSTEM: 2000-2010

	PROJECT NAME	Project Description	Approx. Year Completed
Pump Station No. 9 Service Area			
	Pump Replacements (by O&M Dept.)	All 3 pumps replaced (same size) by O&M Department	2006
	Electrical Improvements	Replaced electrical system and added second power feed.	2005
Pump Station No. 10 Service Area			
	PS#10 Rehab	All 4 Pumps replaced (3 new pumps; all same size; 1 constant speed & 2 VFD)	2005
Pump Station No. 11 Service Area			
	PS#11 FM Changes at NSWTP	Extension of PS#11 FM to new Headworks Building during the 10th Addition	2005
	NSVI-Nicolet Replacement	Approx. 1,150 LF of 24" replaced with 30" near Nicolet Instruments	2001
	PS#11 Firm Capacity Improvements	11B to pump in parallel with 11C or 11D to improve firm capacity	2008
Pump Station No. 12 Service Area			
	PS#12 Firm Capacity Improvements	12B to pump in parallel with 12C or 12D to improve firm capacity	2008
	Control System Modifications	Installed new system control center	2000
Pump Station No. 13 Service Area			
	PS#13 Firm Capacity Improvements	13A replaced. 13B re-built. 13A&13B pump in parallel for firm capacity. 13C unchanged.	2008
	NEI-Airport Relocation	Approx 1,990 LF of 48" FRP replaced 2,480 LF of RCP on west side of Airport	2007
Pump Station No. 14 Service Area			
	PS#14 Firm Capacity Improvements	14A replaced. 14B re-built. 14A&14B pump in parallel for firm capacity. 14C re-built.	2008
Pump Station No. 15 Service Area			
	WI Extension Replacement	Approx. 2,800 LF of 24" replaced with 3,200 LF 42"&36" from Mendota Ave to Mid. Sprgs Dr.	2007
Pump Station No. 16 Service Area			
	Fortune Drive Replacement	Approx. 2,000 LF of 24" replaced with 36" from Gammon Road to Middleton Street	2002
	Control System Upgrade	Replaced original control system with PLC-based controls	2009
Pump Station No. 17 Service Area			
	LBMC Interceptor-Phase 1	Approx 8,000 LF of new 27"-36" interceptor on west side of Verona	2006
	LBMC Interceptor - Phase 2	Approx 5,000 LF of new 27"-30" Interceptor on west side of Verona	2008
	Dual Pumping Modifications	Electrical/control modifications to allow parallel pumping	2007
Pump Station No. 18 Service Area			
	New Forcemain at NSWWTP	Installed ~650 feet of 42" forcemain piping at plant as part of Tenth Addition to NSWWTP project	2005

completed in each pump station area except for Pump Station 5, with a strong emphasis on improvements in the Pump Station 8 service area.

Screenings and Solids Handling Update

Chapter 5 of the *2002 Collection System Facilities Plan* provided a discussion of screening and solids handling at MMSD's pumping stations. The goals for screening and solids handling as presented in the facility plan are summarized as follows:

- Remove screening materials from the wastewater at some point in the treatment or conveyance process.
- Minimize the number of sites where screening materials are collected in order to mitigate operation and maintenance costs and odor complaints.
- Reduce the volume and weight of any screened material by washing, dewatering, and compacting.
- Contract with a local waste removal company to handle and dispose of screening materials.
- Protect pumps from harmful objects and materials that are present in the wastewater.

Fine screening equipment was installed in the new Headworks Facility that was constructed as part of the 10th Addition to NSWWTP improvements in 2005. The use of fine screens accomplished the District's primary goal of removing solids in one central location. Three alternatives were presented in the *2002 Collection System Facilities Plan* to address the remaining solids handling issues at District pumping stations:

1. Alternative 1: Remove all existing solids handling equipment at pumping stations.
2. Alternative 2: Install channel grinders in the incoming flow stream.
3. Alternative 3: Install and/or retrofit mechanical bar screening operations and provide an automated method for cleaning, dewatering, and compacting the screened materials.

Alternative 1 was chosen as the preferred alternative upon installation of fine screening equipment at the treatment plant in 2005. This alternative addressed all of the aforementioned goals except for the protection of pumping equipment. It allowed operation and maintenance efforts to be concentrated in one central location, thereby lowering costs. Screening, dewatering, and compaction of screenings would not have to be performed at each pumping station. This alternative also eliminated the need for District personnel to clean and maintain solids handling equipment and manually dispose of screenings at several pumping stations. As a result, working conditions for District employees would be improved by less exposure to confined spaces and less handling of wet, heavy, odorous material.

Solids handling equipment was present in eight of the District's 17 pumping stations at the time that the fine screens were installed at the treatment plant in 2005. The bar screens at PS1, PS2, PS6, PS7, PS10, and PS11 were removed upon start-up of the fine screening equipment. The bar screen at PS8 was decommissioned in 1999 when a channel grinder was installed. The grinder was subsequently removed as part of the station rehabilitation in 2008 due to its maintenance requirements and inability to pass certain materials. A channel grinder at PS17 is the only form of solids handling equipment that is still present in the District's collection system. A brief history of solids handling equipment employed at each station is shown in Table 3.4.

The only solids handling goal of the *2002 Collection System Facilities Plan* that was not addressed under Alternative 1 was protection of wastewater pumping equipment. Based on past experience, District staff felt that the pumping units in the collection system could adequately pass any large solids that might be present in the wastewater without endangering the performance of the equipment.

The result of removing bar screens at the larger pumping stations in 2005 and 2006 has generally been positive. Pumps at PS1, PS2, PS8, PS10, and PS11 have generally required more frequent attention due to plugging with rags and other solid material over the last five years, as would be expected, although the increase in required maintenance is not excessive. Table 3-5 provides an estimate of the time spent by District mechanics removing rags from pumps in 2010. Overall, it is estimated that District mechanics spend approximately 6.9% of their working time addressing the issue of rags at both District and non-District pumping stations. The amount of time spent at PS7 dealing with rags is significant (approximately 26% of total).

There does not appear to be a clear reason for the higher frequency of pump plugging at PS7 relative to the other pumping stations. Pumping stations immediately upstream of PS7 (PS6, PS9, and PS10) do not exhibit similar problems. District staff has analyzed various control strategies to address pump plugging problems at PS7. One strategy that has been employed thus far uses automated gates in the inlet channel to the wet well to isolate each half of the well to increase flushing velocities and eliminate possible dead zones in the wet well near pump inlet piping. The effect of this change and other possible changes will be evaluated on an ongoing basis.

Given the problems observed with pump plugging at PS7 since removal of the bar screens in 2006, it is expected that some form of screening will be implemented at PS18. The screened material will be cleaned, dewatered, and compacted to mitigate volume, weight, and odors, and will be collected for disposal by a private waste hauler. It is not expected that MMSD personnel will be involved in the collection and disposal of the screenings. Based on preliminary flow splitting alternatives for PS7 and PS18, it is estimated that approximately 75% of the flow that is currently conveyed to PS7 will be screened at future PS18.

Table 3.4 – History of Solids Handling Equipment at District Pumping Stations

Pump Station	Solids Handling Equipment	Year Installed	Year Removed	Status of Solids Handling Equipment (2010)
1	Bar screen	1975	2005	None
2	Bar screen	1964	2005	None
3	None	-	-	None
4	Comminutor	1967	1994	None
5	None	-	-	None
6	Bar screen	1975	2006	None
7	Bar screen	1992	2006	None
8	Bar screen	1963	1999	None
	Channel grinder	1999	2008	
9	None	-	-	None
10	Bar screen	1965	2005	None
11	Bar screen	1965	2006	None
12	Comminutor	1969	1994	None
13	Comminutor	1970	1993	None
14	Comminutor	1971	1994	None
15	Barminutor	1974	1989	None
16	Barminutor	1982	1985	None
17	Channel grinder	1996	N/A	In Service

Table 3.5 – Rag Removal at District Pumping Stations (2010)

Pump Station	Time Spent by District Mechanics Removing Rags from Pumps (hours)	Time Spent by District Mechanics Removing Rags from Pump Vents (hours)
Pump Station 7	228	-
Pump Station 11	142	6
Other District Pump Stations(1)	214	37
Non-District Pump Stations(2)	194	45
TOTAL	**778**	**88**

Notes/Calculations:

1) Includes all District owned pumping stations except for PS7 and PS11.

2) Includes pump stations maintained, but not owned, by MMSD.

3) Estimate of time spent by District mechanics in 2010 on rag removal at pumping stations:
   a) Seven District mechanics
   b) Total annual work hours (gross) = 7 x 2080 hr/year = 14,560 hours
   c) Each mechanic averages 282 hours away from work each year (paid leave, sick leave, etc.)
   d) Total annual work hours (net) = 14,560 – (282)(7) = 12,586 hours
   e) Percent time spent on rag removal = (778+88)/12,586 = 6.9%

Hydraulic Model

The development of a hydraulic modeling tool was recommended as a special project in Chapter 4 of the *2002 Collection System Facilities Plan*. Given the District's vast and interconnected collection system, a means of analyzing non-uniform and unsteady flows over time was desired. As mentioned in the 2002 Plan, the primary uses for such a model were twofold: (1). The model would provide a tool to test the effect of various assumed storms and recurrence intervals on trial designs, and (2). By incorporating previous study data and new calibration data, the model would characterize the estimated degree of infiltration and inflow susceptibility for each of the individual basins making up the model and would illustrate the potential effects on the system of reducing the I/I within any basin.

In 2003 the District hired a consultant to build and develop a hydraulic model of the collection system. All physical characteristics of the collection system were input into the model database. This included pipe characteristics such as size, diameter, and material type as well as all pump information for each station. Every pipe, manhole, and pumping station was modeled in the network as well as some of the larger City of Madison facilities.

Drainage catchments were added using the GIS features of the model and population estimates for each catchment were made using U.S. Census data. From these population estimates dry weather flows for each catchment were developed by comparison to historical average daily flow records at pumping stations.

One of the primary benefits of the chosen hydraulic model is its ability to generate and route wet weather flows throughout the collection system. Long-term rainfall records can be input into the model and routed into the collection system as overland flow or infiltration using a complex groundwater module.

One of the primary considerations given to the model development was the need for a rigorous and detailed calibration. A significant effort was undertaken to ensure that the model could simulate and route flows for both dry and wet weather periods. The model was calibrated for large wet weather events by comparing predicted flows to actual flows observed in the system for the large rain event in May of 2004. Validation of the model was subsequently performed for a previous large rain event in August of 2001. Ongoing maintenance and calibration of the model will be important considerations as the District's service area grows and improvements to the collection system are made.

The District received the final model in 2005 and has used it primarily as a tool for checking peak flow conveyance in certain parts of the collection system and for assessing the effects of station outages due to construction-related projects. In time it is hoped that the model can be further developed to take advantage of its groundwater modeling capabilities so that projects to identify and remove inflow and infiltration can be addressed. The issue of I/I is discussed in greater depth in Chapter 8.

Chapter 4
System Capacities and Projected Flows

Chapter Outline

This chapter is organized into the following sections:

- Introduction
- Projected Flowrates
- Benchmark Design Capacities
- Limitations of Flow Measurements
- Pumping Station Capacity Analysis
- Pumping Station No. 15 Flow Diversion
- Forcemain Capacity Analysis
- Gravity Interceptor Capacity Analysis
- Discussion

Introduction

This chapter will examine the available capacities and the projected flows for each major component of the MMSD collection system.

As shown schematically in Figure 4.1, the MMSD collection system includes a network of gravity interceptors feeding into 17 regional pumping stations. Each pumping station conveys its flow through a forcemain into the next gravity drainage basin or (for the downstream-most stations) to the Nine Springs Wastewater Treatment Plant. The entire system ultimately converges into six stations (PS No. 2, 3, 4, 7, 8, 11) that convey the flow directly into the treatment plant through four forcemains. A common forcemain conveys the combined flow from Pumping Stations No. 2, 3 and 4.

Figure 4.1 shows measured average daily flows in 2010 and projected average daily flows in 2030 for each pumping station. The 2010 average daily flows are based on analysis of MMSD's venturi meter and pump run-time records. Flows in 2010 were selected as the baseline year for analysis and comparison to pumping station capacities and projected flowrates in the collection system.

The 2030 average daily flows are as projected by the Capital Area Regional Planning Commission (CARPC) in their report entitled *MMSD Collection System Evaluation* (Appendix A1). According to this evaluation, over the 20-year study period (2010-2030) MMSD's total flow is expected to increase from 44.1 mgd to 49.7 mgd, a 13% overall increase or approximately 0.65% per year. Using estimates for both 2010 and 2030, flows in several pump station service areas are projected to increase very slightly or actually decrease over the 20-year period, including PS1, PS3, PS4, PS5, and PS8. Conversely, flows in the PS7 and PS17 service areas are projected to increase 37% and

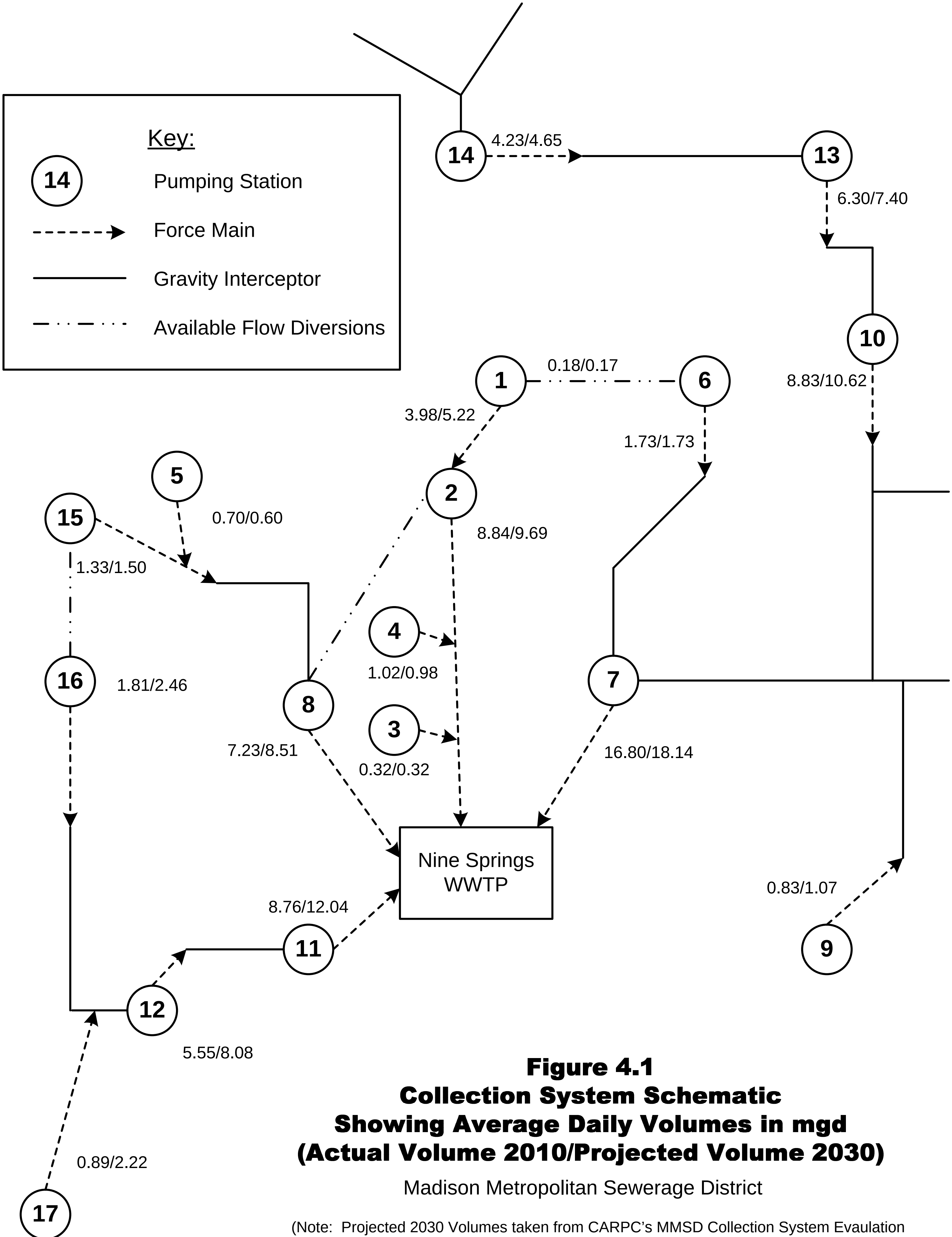


Figure 4.1
Collection System Schematic
Showing Average Daily Volumes in mgd
(Actual Volume 2010/Projected Volume 2030)

Madison Metropolitan Sewerage District

(Note: Projected 2030 Volumes taken from CARPC's MMSD Collection System Evaulation (2009). 2030 Volumes derived from Traffic Analysis Zone (TAZ) data).

87%, respectively, over this same time period due primarily to population growth. Most of these general trends are also observed when comparing actual 2010 flows to projected 2030 flows (Figure 4.1).

Projected Flowrates

Estimation of population, employment, and land use changes in the District's service area are important considerations for projecting future average daily and peak hourly flowrates. Accurate and reliable projections are needed so that the capacity of existing conveyance facilities can be analyzed properly and additional facilities can be planned for if needed. Table 4.1 summarizes the historic trends in population as well as forecasts for future years for the MMSD service area.

Table 4.1: Population Trends and Forecasts for the MMSD(1)

	1980	1990	2000	2030	*2060*
Central USA	218,344	245,390	268,850	339,222	*404,204*
Cottage Grove USA	901	1,131	4,059	9,372	*11,798*
Dane USA			799	1,351	*1,594*
Fox Bluff LSA			240	240	*240*
Kegonsa LSA			2,228	2,252	*2,252*
Morrisonville USA			352	428	*464*
Northern USA	5,393	7,160	9,901	16,883	*23,825*
Verona USA			7,306	15,685	*20,178*
Waubesa LSA			2,027	2,027	*2,027*
Waunakee USA	3,890	5,899	9,000	17,458	*23,367*
Windsor Prairie LSA			509	509	*509*
Westport LSA			377	377	*377*
MMSD	228,528	259,580	305,648	405,804	*490,835*

Note: (1). Data from *MMSD Collection System Evaluation, CARPC (January 2009).*

The population forecasts for the MMSD service area were developed by CARPC based on countywide projections prepared by the Wisconsin Department of Administration (DOA). The latest DOA projections were prepared in 2004 based on 2000 U.S. Census data. CARPC allocated these countywide forecasts to urban service areas within the MMSD service area.

Additionally, smaller planning units called traffic analysis zones (TAZ) were used to develop and refine population and employment projections. These zones were developed by the Madison Area Transportation Planning Board (MATPB) and contain socioeconomic data that includes population, number of households, and total employment for the year 2000 and forecasts for the year 2030. The MATPB developed the TAZ 2030 population and household data by allocating the forecasts prepared by DOA/CARPC to the various traffic analysis zones based on community development plans.

Since the TAZ data was prepared prior to the preparation of many municipal comprehensive and neighborhood development plans, there is some uncertainty with regards to the accuracy of these projections. To account for this uncertainty, CARPC developed an additional forecast method employing an uncertainty factor (UF). The UF method works with both the TAZ data and the most recent community development plans to allocate increases in population and employment based on available land area throughout the MMSD service area.

In general the UF forecasts project higher development rates, and thus higher wastewater flows, than the TAZ forecasts. Unless specifically noted, MMSD has elected to utilize the UF forecasts for purposes of analyzing capacity in its collection system as part of this Facilities Plan. It is understood that UF data will most often result in identifying a need to replace or reinforce a facility before it may actually be necessary. The TAZ data and other considerations such as pumping records and in-line flow measurement should be used to further define the need and timing for system improvements as each individual project is identified and moves forward.

<u>Benchmark Design Capacities</u>

Sanitary sewers in principle are intended to convey point source sanitary sewage, not stormwater. The actual design of sanitary sewer systems, however, is largely controlled by an estimate of the system's susceptibility to stormwater inflow. Average wastewater flow is sometimes used as a convenient base parameter that can be useful to help estimate the degree of susceptibility. Other parameters, such as the tributary land area, population, or miles of sanitary sewers, can also be used as base parameters.

MMSD has historically used the "Madison Design Curve" (MDC) as a benchmark tool for determining the peak design capacity needed for its wastewater conveyance facilities. This curve was prepared for MMSD by consultants Greeley and Hansen in their "Report on Sewerage and Sewage Treatment" (1961), and is also known to MMSD as the "Greeley and Hansen Formula".

The Madison Design Curve can be represented by the following formula:

$$\textit{Peaking Factor} = 4 \Big/ \left(Q_{avg}\right)^{0.158} \quad \text{, for Q in mgd}$$

$$\text{or} \qquad Q_{peak} = 4 \left(Q_{avg}\right)^{0.842} \quad \text{, for Q in mgd.}$$

The MDC is similar in concept to other wastewater conveyance design curves that provide design capacity guidelines as a function of population or of average daily flow. As a general trend of such curves, the peak to average ratio (or "peaking factor") tends to decrease as the size and population of the service area increases. Significant variation of rainfall intensities and flow travel times within a large service area tend to decrease the peaking factors used for large areas.

Typical peaking factors for the Madison Design Curve range from 4.0 (for average flows less than 1 mgd) to 2.5 (for average flows greater than 20 mgd). This is a similar range as the default design capacities referenced in the Wisconsin Administrative Code's NR 110.13. The code calls for peak design capacities to be based on existing records. Where records are not available, the code references design capacities of 400% x average design flow for sub-main and branch sewers, and 250% x average design flow for interceptors, main (trunk) sewers, and sewage outfall pipes.

Peaking factor magnitudes can vary greatly from city to city and from region to region. They are largely dependent on the rainfall and climate of the particular region and the "leakiness" of the particular collection system. MMSD's collection system is relatively tight compared to many systems. Peaking factors experienced in some collection systems are many times higher than the values that would be derived using the Madison Design Curve.

It is important to recognize that peaking factor curves and design guides, including the MDC, cannot guarantee protection against all possible storm events or flood situations. The size and cost of constructing facilities large enough to handle any possible flood is generally not feasible. Actual peak flow rates in Madison during major storms have sometimes exceeded the MDC. Figure 4.2, for example, superimposes the MDC over peaking factor data for each District pumping station from a major rainstorm on June 7-8 of 2008. This rain event delivered 6.3 inches of rain in Madison over a two day period, with extremely high rainfall intensities in the northern portion of the collection system. As shown in Figure 4.2, roughly half of the peaking factors at District pumping stations for this major event exceeded the MDC, and roughly half were less than the MDC. Figure 4.3, which plots the service area peaking factor for each pumping station, shows similar results as Figure 4.2 relative to the MDC.

The MDC provides a useful overall benchmark or reference for comparison of design flows. In general, it is considered by MMSD to be a reasonable design curve for a reasonably tight collection system. For detailed design of individual projects, the analysis of actual flow measurements during major storm events and the consideration of known backup and bypass occurrences within the particular basin provide valuable additional information to help determine an appropriate design. For some projects, the MDC may provide a very conservative level of protection against even very large storm events. For others, the MDC may be less conservative. Further, even if sewer capacities *are* exceeded by an extreme wet weather event, individual drainage basins vary in their ability to withstand surcharged sewers. Due to the variation of topography and basement elevations, some basins may quickly experience bypasses or basement flooding after a sewer is surcharged, while other basins can accept significant surcharges with little adverse result. Further discussion of peaking factors and the handling of wet weather flows can be found in Chapter 8.

Figure 4.2
Peaking Factors vs Flow at MMSD Pump Stations (June 2008)

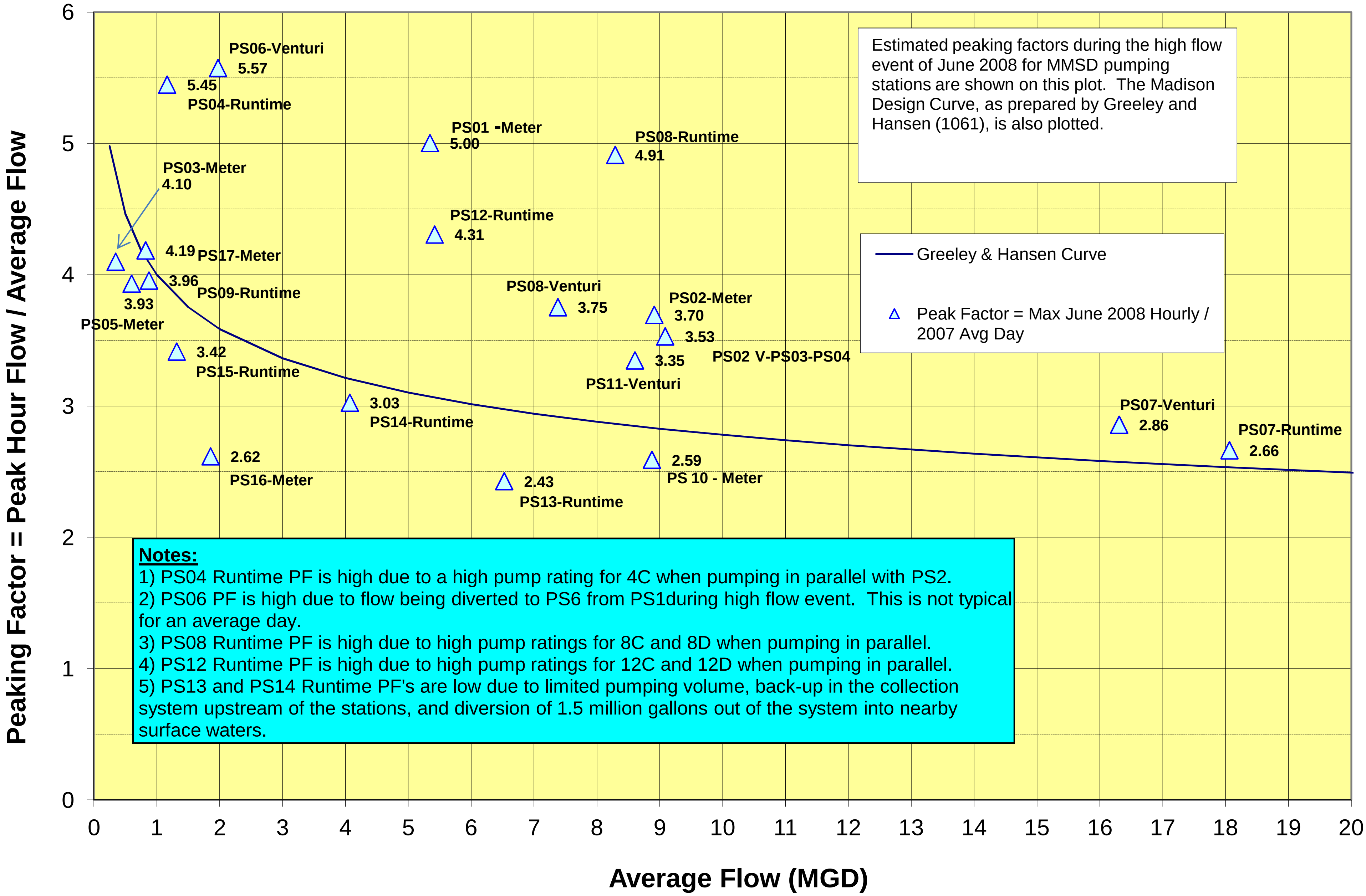
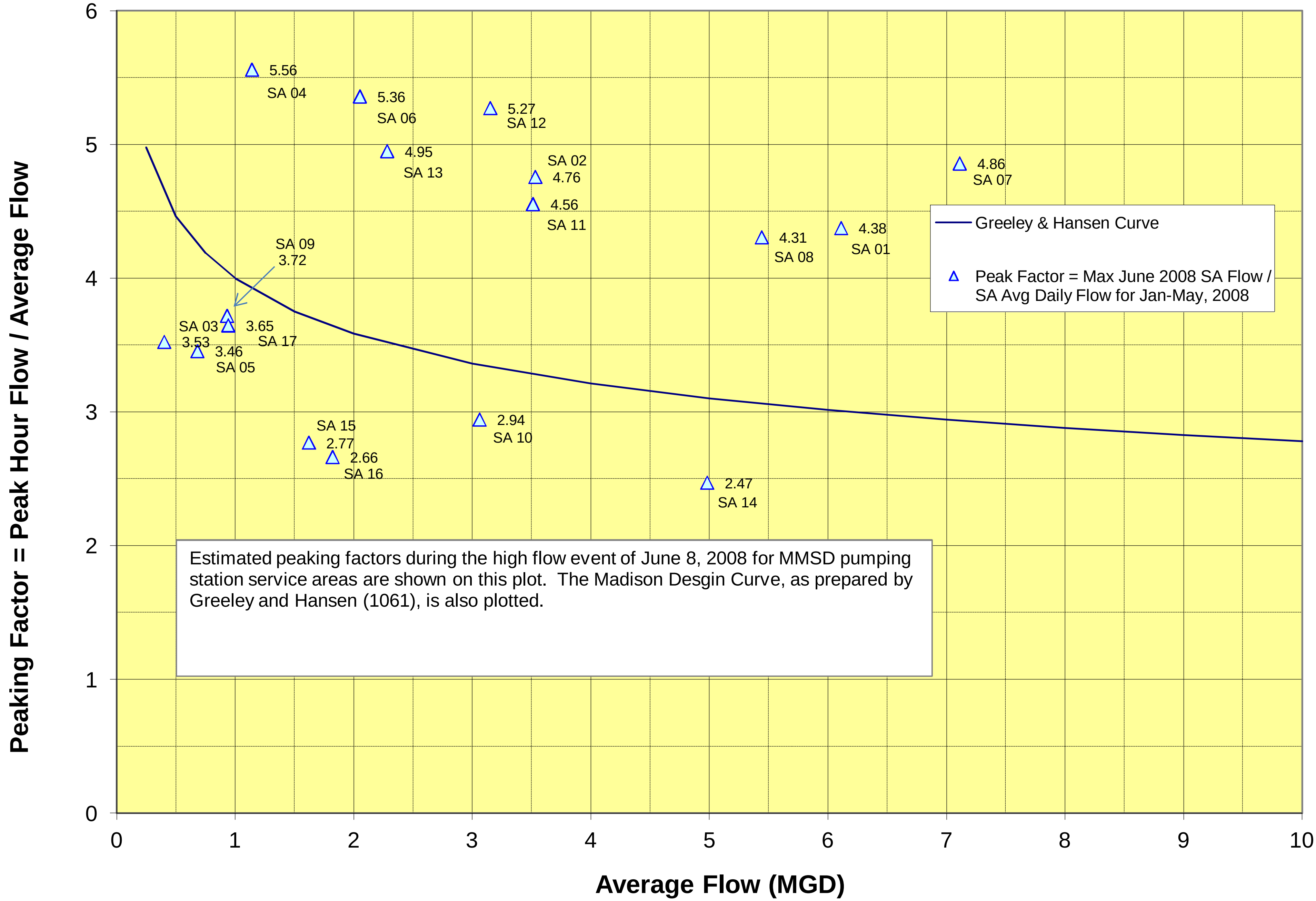


Figure 4.3
Peaking Factors vs Flow for MMSD Service Areas (June 8, 2008)



Limitations of Flow Measurements

Flow volumes and discharge rate data are used extensively in the preparation of wastewater designs and studies, including this facilities plan. It is important, however, to also recognize the limitations of most measured wastewater flow quantities.

A direct measurement of the elapsed time to fill a container of known volume is the most precise way to measure a flow rate. With large volumes of moving water, however, this method is seldom feasible. Venturi meters, magnetic flow meters, or flumes provide the next best source of information. Such meters exist at eleven of MMSD's 17 stations (PS1, 2, 3, 5, 6, 7, 8, 10, 11, 16, 17), but do not exist at the other six stations. Meters can be prone to errors or limits in accuracy, however. Sources of these errors include: (1). Calibration error; (2). Occasional malfunctions of transducers and piping assemblies; and (3). Non-submerged venturi meters during low flow periods.

Flow quantities based on pump run-time data are available at all 17 MMSD stations. However, these flows can be subject to significant errors, since they depend on assumed pump capacity ratings. The assumed pump capacity rating might be based on the original project specifications, the pump manufacturer's catalog, or factory test curves. This rating may be different from the actual in-station pump capacity due to differences in expected friction or wetwell levels, differences in actual motor rpm, or differences in actual impeller diameter. Even if the original pump rating was well documented in the station at the time of installation, pump wear over years of operation, particularly on the impeller and wearing rings, can significantly reduce the original pump discharge rate. Major repairs or impeller substitutions over the years could also impact the pump discharge.

In general, flow rates can be quoted with the most confidence when they can be verified by independent information. Given a significant increase in pump run time, for example, it would not be clear whether the change was caused by an actual incoming flow increase or by the deterioration of a pump. If a flow meter exists at the site, however, the true situation could be verified. In some cases, other information may be available to help provide a "reality check" on suspected faulty flow data. Balancing of flows to agree with trusted measurements from other stations, for example, is sometimes possible.

In many cases, a flow measurement will still depend on some significant assumptions. As a general rule of thumb, it is probably wise to assume that most measured wastewater flow rates are generally within ten percent of the "true" values, but should not be assumed to be much more certain than this. This should not be viewed as a catastrophic limitation, however. In many cases, the increasing or decreasing *trend* of a quantity is more important than the absolute value of the quantity itself.

Pumping Station Capacity Analysis

Table 4.2 summarizes key pump performance data for each of MMSD's 17 pumping stations. As shown, the number of pumps within a station varies from two to four. The maximum overall pumping capacity is also shown for each station. Some stations are designed to achieve their maximum pumping capacity with multiple units operating in parallel, while other stations achieve their maximum capacity with an individual large pump operating alone. The "firm" pumping capacity for a station (sometimes called the reliable capacity) is the overall capacity that can be achieved assuming the largest single pump is out of service.

It might be argued that the firm station capacity can afford to be somewhat less than the maximum station capacity, since the firm capacity becomes important only when a pump outage occurs at the very same time as an extreme flow event. However, it could also be argued that the likelihood of a pump failure increases somewhat during an extreme flow event. For the purpose of this analysis, the more conservative approach is used, and the MDC is used as the benchmark for both maximum and firm capacities.

Table 4.3 is a comparison of recent (2010) and future (2030) flows, the benchmark peak design flows based on the Madison Design Curve, and the present actual pumping capacities at each station. Table 4.3 uses the concept of an "adequacy ratio" for each station. This ratio relates the actual pumping capacity of a station to its benchmark capacity, or estimated influent peak flows. This provides a relative indicator of how well each station is presently equipped to handle present and future peak flows. For example, for PS13, the Year 2010 ratio of 1.06 for maximum capacity means that this station's present pumping capacity was able to provide 106% of its benchmark capacity for the Year 2010. The Year 2030 ratio of 0.78 means that this station's present pumping capacity, if not changed, would be able to provide only 78% of its projected benchmark peak flow in 2030.

Review of Table 4.3 shows that the maximum capacities at five stations (PS7, PS11, PS12, PS13, PS17) are anticipated to become more than 10% short of their benchmarks by 2030. No pumping stations were short of their benchmark maximum capacity in 2010. With regard to *firm* capacities, six stations are anticipated to become more than 10% short of their benchmarks by 2030, although only two of these stations (PS7, PS12) were short of benchmark capacity in 2010. In each case the shortage was less than 10%. The adequacy ratios of Table 4.3 are also presented in Figure 4.4 using a bar chart format. This information will be used in the following chapters to help prioritize future improvement projects.

Pumping Station No. 15 Flow Diversion

MMSD's Pumping Station No. 15 serves the far northwest side of the MMSD service area, including much of the City of Middleton. This station can pump its flow in two

Table 4.2

Pump Performance Data for MMSD Pumping Stations

Madison Metropolitan Sewerage District

Pumping Station No.	Station Location and Year Placed On-Line	Station Pumping Capacity		Individual Pump No.	Estimated Pump Performance at Turn-On Elevation		Nominal speed (rpm)	Nominal Motor Size (HP)	Year Pump On-line	Comments
		Maximum	Firm		Q (gpm)	H (ft.)				
1	104 N. First St. Madison 1950	1A (or 1B) + 1D 26,600 gpm 38.3 mgd	1A (or 1B) + 1C 24,475 gpm 35.3 mgd	1A	14,100	134	890	600	2005	1A & 1B are the new Crosstown pumps and pump to PS#2. 1C & 1D are the old pumps (with re-wound motors) and pump to PS#6. 1A or 1B can pump with 1C or 1D. Pump 1D rating per 6/96 venturi analysis.
				1B	14,100	134	890	600	2005	
				1C	10,375	31	580	150	1950	
				1D	12,500	41	585	150	1950	
2	833 W. Washington Brittingham Park Madison 1964	Any 3 pumps 9,500 gpm (ea) 28,500 gpm total 41.0 mgd total	Any 3 pumps 9,500 gpm (ea) 28,500 gpm total 41.0 mgd total	2A	16,500	108	890	600	2005	All pumps were replaced during station rehab in 2005. All 4 pumps are equal size. 2A & 2B are VFD and 2C & 2D are constant speed. Data reflects new 36" FM online in 2001.
				2B	16,500	108	890	600	2005	
				2C	16,500	108	890	600	2005	
				2D	16,500	108	890	600	2005	
3	Nine Springs WWTP 1959	3A or 3B 1050 gpm 1.51 mgd	3A or 3B 1050 gpm 1.51 mgd	3A	1,050	60	1175	30	1980	New 36" FM (Aug. 2001) has no significant impact on capacities. New Headworks (Aug. 2005) adds ~4' static. New impellers (13.0" vs 12.2") installed in 2004.
				3B	1,050	60	1175	30	1980	
4	620 John Nolen Drive, Madison 1967	4B or 4C 2,900 gpm 4.2 mgd	4B or 4C 2,900 gpm 4.2 mgd	4A	2,000	47	860	40	1967	Peak capacities include new 36" FM (8/2001), new Headworks (8/2005), WSEL=32, wetwell @ -7, PS3 @1,000gpm, PS2 @ 28,500 gpm. New impellers (17.0" vs 16.25") in 4B&4C-2004.
				4B	2,900	95	1160	100	1967	
				4C	2,900	95	1160	100	1967	
5	Spring Harbor Park Madison 1996	Any two pumps 2,480 gpm 3.6 mgd	Any two pumps 2,480 gpm 3.6 mgd	5A	1,800	75	1256	50	1996	Variable speed units. Ratings per 1996 startup testing at 106% speed.
				5B	1,800	75	1256	50	1996	
				5C	1,800	75	1256	50	1996	
6	402 Walter Street Madison 1950	Any 3 pumps 5,600 gpm (ea) 16,800 gpm total 24.2 mgd total	Any 3 pumps 5,600 gpm (ea) 16,800 gpm total 24.2 mgd total	6A	7,700	45	890	125	2009	All ratings shown reflect station rehabilitation project in 2009. All 4 pumps are equal size. 6A is variable speed and 6B-6D are constant speed.
				6B	7,700	45	890	125	2009	
				6C	7,700	45	890	125	2009	
				6D	7,700	45	890	125	2009	
7	6300 Metropolitan Lane, Monona 1950	7C + 7D 31,250 gpm 45.0 mgd	7B + 7C 27,100gpm 39.0 mgd	7A	11,500	47	695	250	1950	Dual pump ratings per 1996 high flow data. No major pump changes since station was rehabbed in 1992.
				7B	15,200	53	705	250	1992	
				7C	19,400	59	705	350	1992	
				7D	19,400	59	705	350	1992	
8	901 Plaenart Dr. Madison 1964	8C+8D+8A(or 8B) 7,900 gpm (ea) 23,700 gpm total 34.1 mgd total	8A+8B+8C(or 8D) 7,850 gpm (ea) 23,600 gpm total 34.0 mgd total	8A	12,800	58	585	250	2009	All ratings shown are **after** station rehabilitation in 2009. 8A&8B (formerly 8C&8D)are variable speed and equal size. 8C&8D (formerly 6C&6D) are constant speed and equal size.
				8B	12,800	58	585	250	2009	
				8C	13,900	60	705	300	2009	
				8D	13,900	60	705	300	2009	

Pumping Station No.	Station Location and Year Placed On-Line	Station Pumping Capacity		Individual Pump No.	Estimated Pump Performance at Turn-On Elevation		Nominal speed (rpm)	Nominal Motor Size (HP)	Year Pump On-line	Comments
		Maximum	Firm		Q (gpm)	H (ft.)				
9	4612 Larsen Beach Road, McFarland 1962	Any two pumps 3,150 gpm 4.5 mgd	Any two pumps 3,150 gpm 4.5 mgd	9A	2,300	51	1185	40	2003	All American Well Works pumps were replaced with Fairbanks Morse Built-Togethers (5434S) between 2002 & 2007. New pumps are same capacity as old.
				9B	2,300	51	1185	40	2007	
				9C	2,300	51	1185	40	2002	
10	192 Regas Road Madison 1965	Any 2 pumps 14,700 gpm (ea) 29,400 gpm total 42.2 mgd total	Any 2 pumps 14,700 gpm (ea) 29,400 gpm total 42.2 mgd total	10A	18,900	94	890	600	2005	All pumps were replaced during station rehab in 2005. All 3 pumps are equal size. 10A & 10B are VFD and 10C is constant speed. Pumps are currently not allowed to operate in parallel.
				10B	18,900	94	890	600	2005	
				10C	18,900	94	890	600	2005	
11	4760 E. Clayton Rd. Town of Dunn 1966	11C + 11D 21,700 gpm 31.2 mgd	11C or 11D + 11B 17,700gpm 25.5 mgd	11A	6,400	43	860	125	1950	11A relocated to PS11 from PS7. 11C & 11D individual capacities per testing in 2/2008. Firm capacity (11C or 11D in parallel with 11B) per testing in 2/2008.
				11B	9,100	49	880	150	1982	
				11C	13,300	57	705	250	1982	
				11D	13,300	57	705	250	1982	
12	2739 Fitchrona Rd. Town of Verona 1969	12C + 12D 16,300 gpm 23.5 mgd	12C or 12D + 12B 11,500 gpm 16.6 mgd	12A	3,400	44	700	50	1969	Firm capacity (12C or 12D in parallel with 12B) per estimate in 2/2008.
				12B	7,200	48	885	100	1969	
				12C	9,000	48	880	150	1982	
				12D	9,000	48	880	150	1982	
13	3634 Amelia Earhart Drive, Madison 1970	13C 14,000 gpm 20.2 mgd	13A + 13B 13,900 gpm 20.0 mgd	13A	8,200	16	585	50	2008	Pump 13A replaced in 2008. 13A matches 13B. Pump 13B re-built, including new impeller (same size). Pump 13C unchanged.
				13B	8,200	16	585	50	1970	
				13C	14,000	20	505	100	1970	
14	5000 School Rd. Madison 1971	14C 10,800 gpm 15.6 mgd	14A + 14B 10,400 gpm 15.0 mgd	14A	7,200	24	705	60	2008	Pump 14A replaced in 2008. 14A matches 14B. Pump 14B re-built, including larger impeller (17.375" vs. 16.5"). Pump 14C re-built with larger impeller (22.0" vs. 20.5").
				14B	7,200	24	695	60	1971	
				14C	10,800	29	585	100	1971	
15	2115 Allen Blvd. Madison 1975	15C 6,100 gpm 8.8 mgd	15A 4,000 gpm 5.8 mgd	15B	3,000	68	885	100	1975	Pump ratings shown are for pumping to the West Int. and PS8. See note (ii).
				15A	4,000	76	885	100	1975	
				15C	6,100	100	885	200	1982	
16	1303 Gammon Rd. Middleton 1982	Any two pumps 13,000 gpm 18.7 mgd	Any two pumps 13,000 gpm 18.7 mgd	16A	7,000	182	1185	500	1982	
				16B	7,000	182	1185	500	1982	
				16C	7,000	182	1185	500	1982	
17	405 Bruce Street Verona 1996	Any two pumps at 118% speed 3,250 gpm 4.6 mgd	Any two pumps at 118% speed 3,250 gpm 4.6 mgd	17A	2,300	115	1290	100	1996	Variable speed pumps. Nominal 100% speed=1190 rpm. Ratings shown are for 118% max speed=1404 rpm. Incorporated 118% dual pumping in 6/2008.
				17B	2,300	115	1290	100	1996	
				17C	2,300	115	1290	100	1996	

Notes:

i) Pump ratings are based on analysis of pump performance curves and system curves, and where available, flow meter data.

ii) For PS15 diversion to PS16, pump ratings are as follows: 15B) 1500 gpm @ 84' 15A) 3000 gpm @ 87' 15C) 6500 gpm @ 96'.

iii) Pump ratings are per pump turn-on level (high wetwell) and C=130.

Table 4.3
Pumping Station Capacities and Projected Flows

Madison Metropolitan Sewerage District

Pumping Station No.	Diversion Status	Station Maximum Pumping Capacity (mgd)	Station Firm Pumping Capacity (mgd)	Average Flows (mgd)			Benchmark Peak Flows (mgd) per Madison Design Curve[4]			Ratio Firm Capacity / Benchmark		Ratio Max. Capacity / Benchmark	
				2000	2010[1]	2030[3]	2000	2010	2030	2010	2030	2010	2030
1	Normal Scenario: PS15 pumps tp PS8.	38.3	35.3	6.87	4.16	5.54	20.27	13.28	16.91	2.66	2.09	2.88	2.27
2		41.0	41.0	4.48	8.84	10.74	21.34	25.06	29.52	1.64	1.39	1.64	1.39
3		1.5	1.5	0.30	0.32	0.35	1.20	1.28	1.40	1.18	1.08	1.18	1.08
4		4.2	4.2	0.91	1.02	1.03	3.69	4.07	4.10	1.03	1.02	1.03	1.02
5		3.6	3.6	0.70	0.70	0.63	2.80	2.80	2.52	1.29	1.43	1.29	1.43
6		24.2	24.2	7.75	1.73	1.74	15.23	6.35	6.38	3.81	3.80	3.81	3.80
7		45.0	39.0	20.15	16.80	23.94	42.95	43.03	59.85	**0.91**	**0.65**	1.05	**0.75**
8		34.1	34.0	8.77	7.23	9.31	24.89	21.16	26.18	1.61	1.30	1.61	1.30
9		4.5	4.5	0.81	0.83	1.28	3.24	3.32	4.92	1.36	**0.91**	1.36	**0.91**
10		42.2	42.2	8.79	8.83	13.26	24.94	25.04	35.26	1.69	1.20	1.69	1.20
11		31.2	25.5	7.50	8.76	15.03	21.82	24.87	39.18	1.03	**0.65**	1.25	**0.80**
12		23.5	16.6	4.32	5.55	10.48	13.71	16.93	28.92	**0.98**	**0.57**	1.39	**0.81**
13		20.2	20.0	5.60	6.30	9.14	17.06	18.84	25.77	1.06	**0.78**	1.07	**0.78**
14		15.6	15.0	3.34	4.23	5.26	11.04	13.47	16.19	1.11	**0.93**	1.16	**0.96**
15		8.8	5.8	1.30	1.33	1.83	4.99	5.09	6.65	1.14	**0.87**	1.73	1.32
16		18.7	18.7	1.37	1.81	3.05	5.48	6.59	10.23	2.84	1.83	2.84	1.83
17		4.6	4.6	0.67	0.89	3.41	2.68	3.56	11.24	1.29	**0.41**	1.29	**0.41**

Pumping Station No.	Diversion Status	Station Maximum Pumping Capacity (mgd)	Station Firm Pumping Capacity (mgd)	Average Flows (mgd)			Benchmark Peak Flows (mgd) per Madison Design Curve(4)			Ratio Firm Capacity / Benchmark		Ratio Max. Capacity / Benchmark	
				2000	2010(1)	2030(3)	2000	2010	2030	2010	2030	2010	2030
8	Alternate Scenario: PS15 pumps to PS16	34.1	34.0	7.47	5.90	7.48	21.75	17.83	21.77	1.91	1.56	1.91	1.57
11		31.2	25.5	8.80	10.09	16.86	24.96	28.01	43.16	**0.91**	**0.59**	1.11	**0.72**
12		23.5	16.6	5.62	6.88	12.31	17.11	20.29	33.12	**0.82**	**0.50**	1.16	**0.71**
15		9.4	4.3	1.30	1.33	1.83	4.99	5.09	6.65	**0.85**	**0.65**	1.85	1.41
16		18.7	18.7	2.67	3.14	4.88	9.14	10.48	15.20	1.78	1.23	1.78	1.23

Notes:

1). Year 2010 actual average flows are based on MMSD metered data for PS1, 2, 3, 5, 7, 8, 10, 11, 16 and 17. Pump run-time records are used at all other stations.

2). Year 2010 was selected as the baseline year for recent average annual flows. Year 2010 is believed to be a representative year for purposes of analysis and comparison.

3). Projected Year 2030 average flows are per CARPC's January 2009 report. These flows are generated from population forecasts utilizing traffic analysis zones and application of an uncertainity factor (UF).

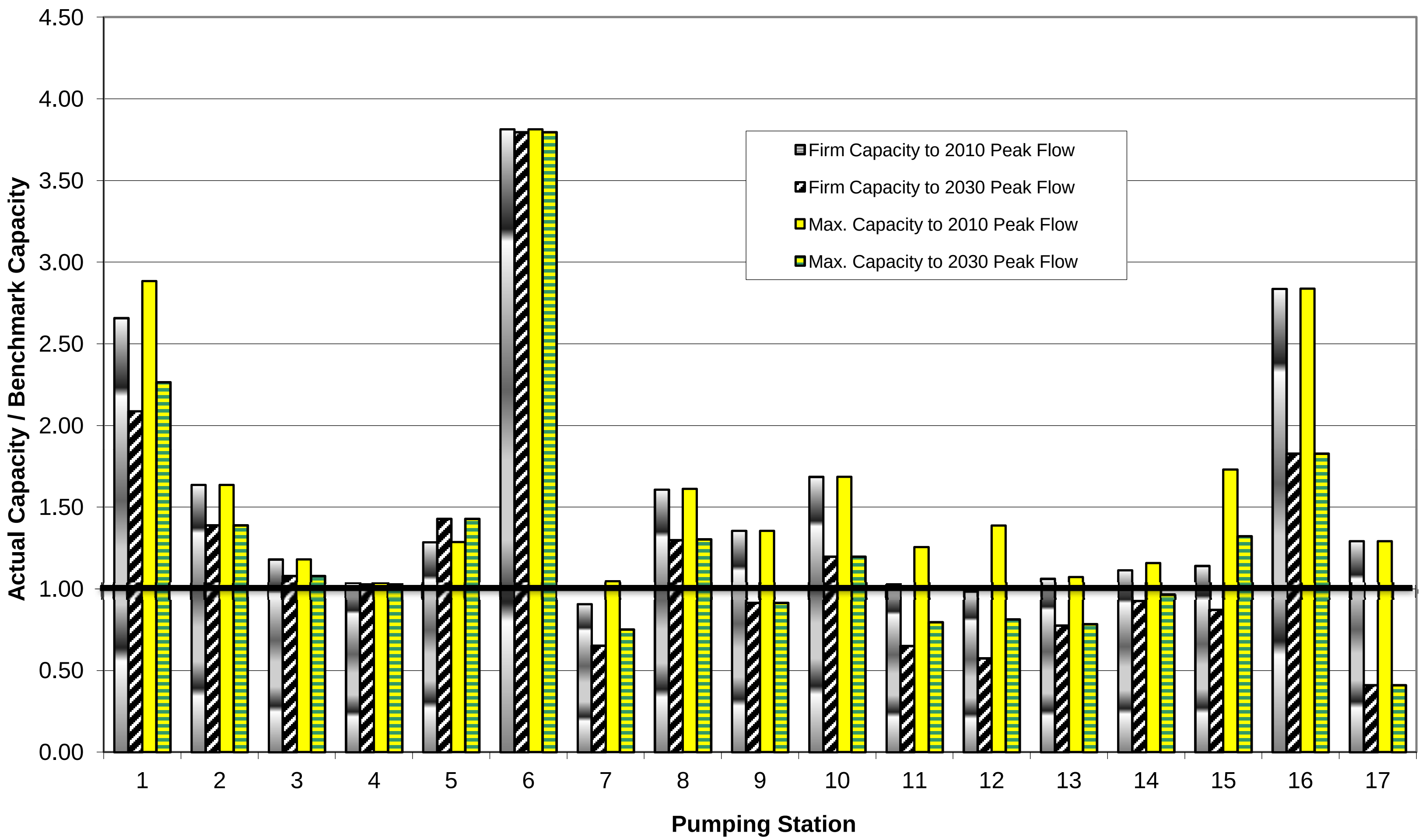
4). Benchmark peak flow requirements are computed per Madison Design Curve. Peaking factor of 4.0 applied for all average flowrates less than 1 MGD. Peaking factor of 2.5 applied for all average flowrates greater than 20 MGD. All other peaking factors equal to 4/(ADF)^0.158).

5). Year 2010 flows from PS 1 were apportioned to downstream pumping stations as follows: (a). 3.98 MGD to PS 2; and (b). 0.18 MGD to PS 6. Benchmark peak flows were based on these average flowrates.

6). All flows from PS 15 in Year 2010 were directed to PS 8. No flow was diverted to PS 16.

7). PS15 pump capacities pumping to PS16, as shown, are different than those pumping to PS8.

Figure 4.4
Pumping Station Capacity Adequacy Ratios
Madison Metropolitan Sewerage District



directions. When originally constructed in 1974, PS15 and its forcemain conveyed its flow to the West Interceptor system, which ultimately leads to PS8. In 1983, a diversion forcemain was constructed to allow the PS15 flow to be diverted to PS16, and then on to the Nine Springs Valley Interceptor system. This diversion was the main operating configuration from 1983 until 1996. Starting in September 1996, the PS15 flow was directed back to the West Interceptor and PS8. This operating change was made in an attempt to reduce odor complaints occurring in the PS16 area, and also to reduce energy costs. No change to the flow direction has been made since 1996, and none is anticipated for operational requirements.

The direction of the discharge from PS15 has significant implications as capacity needs exist in the PS8, PS11 and PS12 service areas in the near term. With PS15 discharging to the PS8 service area as it currently does, it is anticipated that approximately 10,100 feet of sewer in the West Interceptor Relief system will need relief by the year 2020. Approximately 32,000 feet of the Nine Springs Valley Interceptor (NSVI) in the PS11 and PS12 service areas is expected to reach benchmark capacity by 2030 *without* any discharge from PS15.

Diverting flow from PS15 to PS16 would alleviate the need to provide additional capacity for the West Interceptor Relief system prior to 2060, but would accelerate the required timing and scope of improvements needed for the NSVI system. A 50-year present worth analysis was conducted to compare the two alternatives for PS15 pumping. The results are shown in Table 4.4.

Table 4.4: Present Worth Analysis for Pumping Alternatives at PS15

Cost Items	Replacement Costs	Lining Costs	Total Costs
Alternative No. 1: PS15 to PS8			
NSVI	$23,265,000	$9,487,000	$32,752,000
West Interceptor Relief	$10,288,000	$1,602,000	$11,890,000
Pumping Energy	-	-	$5,656,000
TOTAL	$33,553,000	$11,089,000	**$50,298,000**
Alternative No. 2: PS15 to PS16			
NSVI	$33,424,000	$10,663,000	$44,087,000
West Interceptor Relief	$0	$1,258,000	$1,258,000
Pumping Energy	-	-	$15,155,000
TOTAL	$33,424,000	$11,921,000	**$60,500,000**

Note: All costs in 2010 dollars.

The present worth analysis considers new construction and rehabilitation of interceptors for both PS15 pumping alternatives. The analysis assumes that all sewers requiring capacity relief will have a new sewer of the same size built parallel to the existing sewer and that the existing sewer will be rehabilitated with a cured-in-place liner at that time. For those segments not in need of capacity relief prior to 2060, rehabilitation with a new liner was assumed to take place at the end of the sewer's useful service life. The worksheet at the end of this chapter (Appendix 4-1) contains other assumptions used in the analysis as well as detailed information for each individual sewer segment. It should be noted that unit costs for replacement of the West Interceptor are assumed to be twice those for the NSVI system owing to the difficult construction expected along the West Intercepting system route (traffic control, adjacent utilities, etc.).

Energy costs related to pumping were also considered in the present worth analysis (see Appendix 4-2). The overall costs to pump from PS15 to PS16 are approximately three times greater than the costs to pump from PS15 to PS8. This has a significant impact on the cost comparison. Another factor that needs to be considered in the analysis but is not included quantitatively is the issue of odor control. Significant odors were documented at PS16 from 1983-1996 when the PS15 flow was directed to PS16. Odor concerns still exist at PS16 at this time. While it may be possible to construct an odor treatment system to address this issue, the cost of implementing and maintaining such a system would be costly and would likely be prohibitive.

Given the present worth costs outlined in Table 4.4 and the issue of odors at PS16, it is recommended that the District continue its current practice of pumping both average daily and peak flows from PS15 to PS8. As a result, all capacity evaluations in this Facility Plan for all pumping stations, force mains, and interceptors assume that PS15 flow will continue to be directed toward PS8 instead of PS16 and the Nine Springs Valley Interceptor (NSVI) System.

Forcemain Capacity Analysis

The capacity of any pumping station is influenced by the characteristics of its pumping equipment together with the characteristics of its forcemain system. The diameter, length and roughness of the forcemain, and the elevation difference between station wetwell and forcemain discharge, will significantly affect the performance of the pumping unit. All pumping capacities reported in the previous sections of this chapter therefore reflect the characteristics of the station's pumping equipment together with the characteristics of its particular forcemain system.

It is also important, however, to consider the limiting capacity of the forcemain facility itself. Table 4.5 summarizes the characteristics and nominal capacities for each of MMSD's raw wastewater forcemains, without regard to pumping equipment. The nominal capacities shown are based on a common industry practice to limit forcemain velocities to a maximum of 8 feet/second. Using the 8 fps criterion, Table 4.5 shows that the nominal limiting capacity of the forcemain is less than the Year 2030 benchmark

Table 4.5
Forcemain Capacities and Characteristics
Madison Metropolitan Sewerage District

Pumping Station Forcemain No.	Forcemain Characteristics					Nominal FM Capacity (mgd) based on 8 fps velocity	2030 Benchmark Peak Flows (mgd)	
	Segment Length (feet)	Dia. (inches)	Mat'l	Year Installed	Comments		If PS15 pumps to PS8	If PS15 pumps to PS16
1 (to PS 6)	2,638	30	RCCP	1948		25.4	0.00	
1 (to PS 2)	1,340	24	DI	2000	Segment from PS1 to E. Washington Ave.	16.2	16.91	
	998	20	PVC	1995	Segment under Monona Terrace	11.3		
	14,205	30	DI	2002	Balance of FM from E. Wash. Ave. to PS2	25.4		
2	17,064	36	DI	2001	From PS2 to near old meter vault @ NSWTP	36.5	29.52	
	364	36	DI	2005	Installed during the 10th Addition			
3	5	8	CI	1959	Original forcemain remaining	1.8	1.40	
	21	8	DI	2000	Installed dring PS2FM replacement			
4	100	16	CI	1959	Original forcemain remaining	7.2	4.10	
	60	16	DI	2000	Installed dring PS2FM replacement			
5	28	16	DI	1996	Segment from new PS5 to 1959 junction	7.2	2.52	
	504	16	RCCP	1959	Segment to PS15 FM junction	7.2		
	1,746	24	RCCP	1959	Segment from PS5/15 junction to Whitney	16.2		
6	7,208	36	RCCP	1948		36.5	6.38	
7	13,992	2 x 36	RCCP	1948, 63	Dual forcemains from PS7 to plant grounds	65 (based on 8 fps) 55-60 (based on transients) see note 3 below	59.85	
	1,332	48	RCCP	1963	Through plant grounds to 10th Add connection			
	323	48	DI	2005	Installed during the 10th Addition			
8	13,174	42	RCCP	1964	78' of 42" abandoned during 10th Addition	49.7	26.18	21.77
	194	36	RCCP	1964	Located outside of PS#8	36.5		
	334	42	DI	2005	Installed during the 10th Addition	49.7		
9	4,812	20	DI	1987		11.3	4.92	
	2,197	10	AC	1961		2.8	0.00	

Pumping Station Forcemain No.	Forcemain Characteristics					Nominal FM Capacity (mgd) based on 8 fps velocity	2030 Benchmark Peak Flows (mgd)	
	Segment Length (feet)	Dia. (inches)	Mat'l	Year Installed	Comments		If PS15 pumps to PS8	If PS15 pumps to PS16
10	11,112	36	RCCP	1964		36.5	35.26	
11	3,945	36	RCCP	1965	230' of 36" abandoned during 10th Addition	36.5	39.18	43.16
	91	36	DI	2005	Installed during the 10th Addition			
	0	30	RCCP	1964	All 30" was abandoned during 10th Addition			
12	4,795	36	RCCP	1968		36.5	28.92	33.12
13	2,588	36	RCCP	1969		36.5	25.77	
14	4,354	30	RCCP	1971		25.4	16.19	
15 (to PS 8)	2,467	24	DI	1974	Segment from PS15 to Thorstrand air release	16.2	6.65	
	4,811	20	DI	1974	Segment from Thorstrand to PS5 FM junction	11.3		
	1,746	24	RCCP	1959	Segment from PS5 FM juction to Whitney Way	16.2		
15 (to PS16)	1,378	24	DI	1974	Segment from PS15 to junction near Univ. Ave.	16.2	6.65	
	4,893	30	RCCP	1982	Segment from FM junction to near PS16	25.4		
16	7,214	36	DI	1979	Segment from PS16 to Gammon high point	36.5	10.23	15.20
	2,965	30	DI	1980	Segment from high point to near Min. Pt. Rd.	25.4		
17	13,357	16	DI	1995	Segment from PS17 to Hwy. 18/151 high pt.	7.2	11.24	
	3,071	20	DI	1995	Forced gravity segment from high pt. to NSVI	11.3		

Notes:
1 Benchmark flows per Table 4.3
2 Nominal FM Capacities shown are based on 8 feet/sec velocity in principal FM segments
3 Limiting capacity for the PS7 FM is 55-60 MGD due to maximum allowable transient pressures in 36"-1948 FM.

value at three pumping stations (PS1 Crosstown Forcemain, PS11, PS17). In the case of the PS1 Crosstown Forcemain, the limiting segment under Monona Terrace is approximately 1,000 feet in length. A detailed analysis of the forcemain system should be undertaken to determine if this small stretch of forcemain warrants replacement due to its limited capacity.

The effective capacities of some forcemains may be further limited by the age, condition or pressure rating of the pipe. The original 36” segment of the PS7 forcemain (1948) is rated for a pressure head of approximately 100 feet. Since transient pressures under some scenarios can approach this rating, and since this forcemain did experience a major rupture in 1963, MMSD has considered its limiting capacity to be approximately 50 - 60 mgd.

Gravity Interceptor Capacity Analysis

Tables 4.6 and 4.7 show the pipe capacities and the projected flows for the MMSD network of gravity interceptors. Table 4.7 is a detailed compilation of the entire gravity system broken down into significant segments with similar hydraulic properties. These segments reflect the sub-basin service areas used in CARPC’s *Collection System Evaluation*, but with further breakdown to include each major change in pipe capacity, diameter, or materials of construction. Table 4.6 is a summary of Table 4.7. Both tables organize the gravity interceptors into the 17 pumping station drainage basins. Similar to the pumping station analysis earlier in this chapter, the benchmark peak design flows for the gravity interceptors are computed according to the Madison Design Curve.

Table 4.6 shows that 13% of MMSD’s total gravity interceptor mileage will reach or exceed its benchmark capacity based on predicted flows by 2020, and that 26% is projected to reach or exceed its benchmark capacity by 2030. The most significant areas of capacity shortfalls include the Nine Springs Valley Interceptor in the PS11 service area and the Southeast Interceptor and Far East Interceptor in the PS7 service area. It should be noted that the capacity limitations for the Southeast Interceptor will be relieved with the addition of Pumping Station 18 in 2015. This will be discussed in subsequent chapters.

Table 4.7 shows that some individual interceptor segments are expected to see significant flow increases over 20 years, while others are expected to see little or no growth. To reasonably prioritize capacity improvement projects, both the timing and the relative degree of the predicted hydraulic need should be considered. Consider, for example, a particular segment that has already exceeded its computed benchmark capacity, but just marginally. If it is in a low-growth or zero-growth area, and has not actually experienced chronic backup problems, it might be argued that this segment should not be ranked as a high priority need for capacity relief, even though its capacity in theory has already been exceeded. On the other hand, a high growth interceptor that is within its capacity benchmarks today, but is projected to surpass its capacity within 5 years, may be a project deserving of a fairly high priority. Figure 9.1 (in map pocket) highlights the

Table 4.6
Gravity Interceptor & Force Main Capacity Evaluation
Madison Metropolitan Sewerage District

Pumping Station Service Area	Total Gravity Interceptor Mileage in Service Area (miles)	Total Force Main Mileage in Service Area (miles)	Mileage Predicted to Reach Benchmark Capacity By 2020				Mileage Predicted to Reach Benchmark Capacity By 2030			
			Gravity Interceptors		Force Mains		Gravity Interceptors		Force Mains	
			(miles)	(%)	(miles)	(%)	(miles)	(%)	(miles)	(%)
PS1	1.71	3.67	0.00	0%	0.45	12%	0.00	0%	0.45	12%
PS2	2.73	3.29	0.41	15%	0.00	0%	0.41	15%	0.00	0%
PS3	0.72	0.005	0.72	100%	0.00	0%	0.72	100%	0.00	0%
PS4	1.55	0.03	0.00	0%	0.00	0%	0.00	0%	0.00	0%
PS5	3.00	0.42	0.00	0%	0.00	0%	0.00	0%	0.00	0%
PS6	1.91	1.37	0.00	0%	0.00	0%	0.00	0%	0.00	0%
PS7	19.76	2.96	4.44	22%	0.00	0%	8.39	42%	1.33	45%
PS8	14.64	2.60	2.39	16%	0.00	0%	3.22	22%	0.00	0%
PS9	0.63	1.24	0.00	0%	0.01	1%	0.05	9%	0.01	1%
PS10	6.59	2.10	2.07	31%	0.00	0%	2.07	31%	0.00	0%
PS11	10.04	0.79	1.21	12%	0.00	0%	5.29	53%	0.79	100%
PS12	7.86	0.91	0.67	8%	0.00	0%	0.67	8%	0.00	0%
PS13	2.96	0.49	0.00	0%	0.00	0%	0.36	12%	0.00	0%
PS14	15.84	0.85	0.88	6%	0.00	0%	3.49	22%	0.00	0%
PS15	1.97	2.80	0.00	0%	0.00	0%	0.04	2%	0.00	0%
PS16	1.63	1.93	0.00	0%	0.00	0%	0.53	32%	0.00	0%
PS17	2.52	3.11	0.00	0%	2.53	81%	0.00	0%	2.53	81%
Totals	96.06	28.57	12.80	13%	2.98	10%	25.25	26%	5.10	18%

Table 4.7
Gravity Interceptors - Capacities and Predicted Flows

Flow Type	Segment Description	From	To	Length (ft)	Pipe Characteristics: Pipe Dia. (in)	Year Installed	Pipe Material	Nominal Capacity (mgd)	Peak Flows (mgd) / Percent Nominal Capacity: 2000		2010 UF		2030 UF		Capacity Reached	Capacity Needs	
Pump Station No. 1 Service Area																	
GR	North End Interceptor along Sherman Avenue	MH01-126	MH01-123	650	10	1927	VP	0.45	*0.20*	44%	*0.20*	44%	*0.20*	44%	> 2060		
GR	North End Interceptor along Sherman Avenue	MH01-123	MH01-120	832	12	1927	VP	0.73	*0.20*	27%	*0.20*	27%	*0.20*	27%	> 2060		
GR	North End Interceptor along Commercial Avenue	MH01-120	MH01-617	1,085	18	2002	PVC	2.54	*4.13*	163%	*1.67*	66%	*1.64*	65%	> 2060		
GR	North End Interceptor along Commercial Avenue	MH01-617	MH01-616	534	20	2002	PVC	3.36	*4.13*	123%	*1.67*	50%	*1.64*	49%	> 2060		
GR	North End Interceptor along Pennsylvania Avenue	MH01-616	MH01-604	4,248	36	2002	PVC	16.10	*12.27*	76%	*8.38*	52%	*8.51*	53%	> 2060		
GR	North End Interceptor along E. Johnson Street	MH01-604	MH01-304	787	42	2002	PVC	24.29	*12.27*	51%	*8.38*	34%	*8.51*	35%	> 2060		
GR	Northeast Interceptor Relief	MH01-003	MH01-001	189	30	1937	CI	8.38	*1.37*	16%	*1.37*	16%	*1.36*	16%	> 2060		
GR	East Johnson Street Relief Sewer	MH01-001	MH01-303	38	36	1979	RCP	23.60	*1.37*	6%	*1.37*	6%	*1.36*	6%	> 2060		
GR	East Johnson Street Relief Sewer	MH01-304	PS1	658	36	1979	RCP	23.60	*13.19*	56%	*9.37*	40%	*9.50*	40%	> 2060		
GR	City of Madison Interceptor - Blount Street to PS 1	-	-	-	-	-	-	-	*7.95*		*8.42*		*9.35*				
FM	PS 1 Force Main - PS 1 to PS 6	PS 1	PS 6	2,638	30	1948	RCP	25.40	*N/A*		*N/A*		*N/A*		>2060		
FM	Cross Town Force Main	PS1	PBXT-01337	1,346	24	2000	DI	16.20	*19.06*	118%	*15.95*	98%	*16.90*	104%	2010-2020	X	Y
FM	Cross Town Force Main	PBXT-01337	PBXT-06139	4,987	30	2002	DI	25.40	*19.06*	75%	*15.95*	63%	*16.90*	67%	> 2060		
FM	Cross Town Force Main	PBXT-06139	BDXT-07930	1,791	30	2002	PVC	25.40	*19.06*	75%	*15.95*	63%	*16.90*	67%	> 2060		
FM	Cross Town Force Main	BDXT-07930	RDXT-09244	1,314	30	2002	DI	25.40	*19.06*	75%	*15.95*	63%	*16.90*	67%	> 2060		
FM	Cross Town Force Main	RDXT-09244	PBXT-09256	12	20	2002	DI	11.30	*19.06*	169%	*15.95*	141%	*16.90*	150%	2000	X	Y
FM	Cross Town Force Main	PBXT-09256	PBXT-10254	998	20	1995	PVC	11.30	*19.06*	169%	*15.95*	141%	*16.90*	150%	2000	X	Y
FM	Cross Town Force Main	PBXT-10254	RDXT-10260	6	20	2002	DI	11.30	*19.06*	169%	*15.95*	141%	*16.90*	150%	2000	X	Y
FM	Cross Town Force Main	RDXT-10260	PS2	6,285	30	2002	DI	25.40	*19.06*	75%	*15.95*	63%	*16.90*	67%	> 2060		
		Total Length of Gravity Sewers (mi)		1.71										Total Length of Gravity Interceptors Reaching Capacity by 2020 (mi)	0.00		
		Total Length of Force Mains (mi)		3.67										Total Length of Gravity Interceptors Reaching Capacity by 2030 (mi)	0.00		
														Total Length of Force Mains Reaching Capacity by 2020 (mi)	0.45		
														Total Length of Force Mains Reaching Capacity by 2030 (mi)	0.45		
Pump Station No. 2 Service Area																	
GR	Original West Interceptor on Randall Avenue - Dayton Street to Spring Street	MH02-014A	MH02-316	420	24	1916	CI	7.73	*2.23*	29%	*2.22*	29%	2.19	28%	> 2060		
GR	West Interceptor - Spring Street Relief	MH02-316	MH02-300	4,577	24	1940	CI	6.54	*2.23*	34%	*2.22*	34%	2.19	33%	> 2060		
GR	West Interceptor - Spring Street Relief at West Washington Avenue	MH02-300	MH02-101	3	24	1940	CI	6.54	*7.20*	110%	*7.76*	119%	8.86	135%	2000	X	Y
Junction with Original West Interceptor																	
GR	Original West Interceptor at Regent Street/Randall Avenue	MH02-316	MH02-011	1,115	24	1916	CI	4.62	*0.00*	0%	*1.36*	29%	*1.68*	36%	> 2060		
GR	Original West Interceptor on Regent Street	MH02-011	MH02-008	900	24	1916	CI	4.62	*5.65*	122%	*6.95*	150%	*7.69*	166%	2000	X	Y
GR	Original West Interceptor on Regent Street	MH02-008	MH02-005A	1,260	24	1916	CI	5.27	*5.65*	107%	*6.95*	132%	*7.69*	146%	2000	X	Y
GR	City of Madison Frances Street Interceptor	MH02-005A	MH02-402	1,296	30	1968	RCP	12.43	*5.65*	45%	*6.95*	56%	*7.69*	62%	> 2060		
GR	Original West Interceptor	MH02-005	MH02-101	1,319	24	1916	CI	8.89	*0.23*	3%	*0.22*	2%	*0.21*	2%	> 2060		
Junction with Spring Street Relief																	
GR	West Interceptor to PS 2 along West Washington Avenue	MH02-101	MH02-402	10	36	1963	RCP	26.21	*7.38*	28%	*7.93*	30%	*9.01*	34%	> 2060		
GR	West Interceptor to PS 2 along West Washington Avenue	MH02-402	MH02-401	284	48	1963	RCP	24.55	*11.97*	49%	*13.61*	55%	*15.25*	62%	> 2060		
Junction with Southwest Interceptor																	
GR	SWI on Haywood Street	MH08-106	MH02-606	1,438	24	1936	CI	5.06	*0.15*	3%	*0.16*	3%	*0.18*	4%	> 2060		
GR	SWI on West Shore Drive	MH02-606	MH02-401	1,770	36	2001	PVC	46.95	*1.27*	3%	*1.25*	3%	*1.22*	3%	> 2060		
Junction with Original West Interceptor																	
GR	Interceptor to PS 2	MH02-401	PS2	30	48	1963	RCP	37.12	*12.83*	35%	*14.45*	39%	*16.04*	43%	> 2060		
FM		PS2	TE02-10933	9,890	36	2001	DI	36.50	*28.69*	79%	*27.25*	75%	*29.53*	81%	> 2060		
Junction with PS4 force main																	
FM	From PS4 junction to PS3 junction	TE02-10933	TE02-17328	6,395	36	2001	DI	36.50	*30.93*	85%	*29.56*	81%	*31.88*	87%	> 2060		
Junction with PS3 force main																	
FM	At Nine Springs WWTP	TE02-17328	BD02-18136	757	36	2000-2001	DI	36.50	*31.64*	87%	*30.31*	83%	*32.68*	90%	2030-2060		
FM	At Nine Springs WWTP	BD02-18136	Headworks	354	36	2006	DI	36.50	*31.64*	87%	*30.31*	83%	*32.68*	90%	2030-2060		
		Total Length of Gravity Sewers (mi)		2.73										Total Length of Gravity Interceptors Reaching Capacity by 2020 (mi)	0.41		
		Total Length of Force Mains (mi)		3.29										Total Length of Gravity Interceptors Reaching Capacity by 2030 (mi)	0.41		
														Total Length of Force Mains Reaching Capacity by 2020 (mi)	0.00		
														Total Length of Force Mains Reaching Capacity by 2030 (mi)	0.00		

Capacity Needs: **X** = Capacity reached by 2020 **Y** = Capacity reached by 2030

Table 4.7
Gravity Interceptors - Capacities and Predicted Flows

Flow Type	Segment Description	From	To	Length (ft)	Pipe Characteristics: Pipe Dia. (in)	Pipe Characteristics: Year Installed	Pipe Characteristics: Pipe Material	Nominal Capacity (mgd)	*Peak Flows (mgd)* / Percent Nominal Capacity: 2000		2010 UF		2030 UF		Capacity Reached	Capacity Needs
Pump Station No. 3 Service Area																
GR	Rimrock Interceptor	MH03-311	MH03-102	3,492	12	1959	RCP	1.08	*1.24*	115%	*1.29*	119%	*1.40*	130%	2000	X Y
GR	Rimrock Interceptor at PS 3	MH03-102	PS3	308	10	1958	CI	1.00	*1.24*	124%	*1.29*	129%	*1.40*	140%	2000	X Y
FM	At Nine Springs WWTP	PS3	TE03-00009	9	8	1958	CI	1.80	*1.24*	69%	*1.29*	72%	*1.40*	78%	> 2060	
FM	At Nine Springs WWTP	TE03-00009	TE02-17328	17	8	2001	DI	1.80	*1.24*	69%	*1.29*	72%	*1.40*	78%	> 2060	
Junction with PS2 / PS4 force main																
			Total Length of Gravity Sewers (mi)	0.72										Total Length of Gravity Interceptors Reaching Capacity by 2020 (mi)	0.72	
			Total Length of Force Mains (mi)	0.005										Total Length of Gravity Interceptors Reaching Capacity by 2030 (mi)	0.72	
														Total Length of Force Mains Reaching Capacity by 2020 (mi)	0.00	
														Total Length of Force Mains Reaching Capacity by 2030 (mi)	0.00	
Pump Station No. 4 Service Area																
GR	South Interceptor - Baird Street Extension	MH04-408	MH04-313	1,414	15	1928	VP(L)	2.87	*1.52*	53%	*1.55*	54%	*1.62*	56%	> 2060	
GR	South Interceptor - Baird Street Extension	MH04-313	MH04-312	14	12	1995	PVC	7.27	*1.52*	21%	*1.55*	21%	*1.62*	22%	> 2060	
SI	South Interceptor - Wingra Creek Siphon	MH04-312	MH04-311	156	10&14	1995	DI	4.00	*2.90*	73%	*2.96*	74%	*3.08*	77%	> 2060	
GR	South Interceptor - Beld Street to Wingra Creek Siphon	MH04-315	MH04-311	643	24	1995	PVCPW	5.46	*0.18*	3%	*0.19*	3%	*0.21*	4%	> 2060	
GR	South Interceptor - Wingra Creek Siphon to Sayle Street	MH04-311	MH04-209	3,048	24	1995	PVCPW	5.46	*3.49*	64%	*3.56*	65%	*3.69*	68%	> 2060	
GR	South Interceptor - Sayle Street to PS 4	MH04-209	MH04-201	2,214	24	1967	AC	4.62	*3.49*	76%	*3.56*	77%	*3.69*	80%	> 2060	
GR	South Interceptor - Fairgrounds Branch	MH04-201B	MH04-201	653	15	1967	AC	2.25	*0.40*	18%	*0.41*	18%	*0.41*	18%	> 2060	
GR	South Interceptor to PS 4	MH04-201	PS4	30	24	1967	AC	5.27	*3.89*	74%	*3.96*	75%	*4.09*	78%	> 2060	
FM		PS04	TE04-00098	98	16	1967	CI	7.20	*3.89*	54%	*3.96*	55%	*4.09*	57%	> 2060	
FM		TE04-00098	TE02-10933	55	16	2000	DI	7.20	*3.89*	54%	*3.96*	55%	*4.09*	57%	> 2060	
Junction with PS2 force main																
			Total Length of Gravity Sewers (mi)	1.55										Total Length of Gravity Interceptors Reaching Capacity by 2020 (mi)	0.00	
			Total Length of Force Mains (mi)	0.03										Total Length of Gravity Interceptors Reaching Capacity by 2030 (mi)	0.00	
														Total Length of Force Mains Reaching Capacity by 2020 (mi)	0.00	
														Total Length of Force Mains Reaching Capacity by 2030 (mi)	0.00	
Pump Station No. 5 Service Area																
GR	West Interceptor Diversion at PS 15 - Marshall Park	MH05-102A	MH05-021	555	30	1957	RCP	7.01	*0.19*	3%	*0.00*	0%	*0.00*	0%	> 2060	
GR	West Interceptor Diversion at PS 15 - Marshall Park	MH05-021	MH05-020	238	14	1931	CI	2.11	*0.19*	9%	*0.00*	0%	*0.00*	0%	> 2060	
GR	West Interceptor Diversion - Marshall Park to Lake Mendota Dr.	MH05-020	MH05-011	2,554	16	1931	CI	1.92	*0.19*	10%	*0.00*	0%	*0.00*	0%	> 2060	
Junction with West Interceptor - Gammon Extension																
GR	West Interceptor - Gammon Extension	MH05-230	MH05-214	4,598	14	1966	AC	1.39	*1.16*	83%	*1.18*	85%	*1.21*	87%	> 2060	
GR	West Interceptor - Gammon Extension	MH05-214	MH05-206	2,534	10	1966	AC	1.90	*1.16*	61%	*1.18*	62%	*1.21*	64%	> 2060	
GR	West Interceptor - Gammon Extension	MH05-206	MH05-201	1,517	12	1966	AC	2.01	*1.33*	66%	*1.34*	67%	*1.38*	69%	> 2060	
GR	West Interceptor - Gammon Extension	MH05-201	MH05-011	168	18	1966	AC	2.35	*1.33*	57%	*1.34*	57%	*1.38*	59%	> 2060	
Junction with Original West Interceptor																
GR	Original West Interceptor - Gammon Ext. to PS 5	MH05-011	MH05-402	3,561	18	1931	CI	2.25	*1.98*	88%	*1.81*	80%	*1.86*	83%	> 2060	
GR	West Interceptor to PS 5	MH05-402	MH05-401	92	24	1995	PVC	7.31	*1.98*	27%	*1.81*	25%	*1.86*	25%	> 2060	
GR	West Interceptor to PS 5	MH05-401	PS5	28	24	1995	PVC	7.31	*2.59*	35%	*2.44*	33%	*2.52*	34%	> 2060	
FM	PS5 FM replaced with new station (1994)	PS5	TE05-22834	27	16	1994	DI	7.20	*2.59*	36%	*2.44*	34%	*2.52*	35%	> 2060	
FM	PS5 original FM	TE05-22834	TE05-22376	458	16	1959	PCCP	7.20	*2.59*	36%	*2.44*	34%	*2.52*	35%	> 2060	
Junction with PS 15 force main																
FM		TE05-22376	MH02-547	1,742	24	1959	PCCP	16.2	*7.42*	46%	*7.75*	48%	*8.54*	53%	> 2060	
Junction with West Interceptor Relief																
			Total Length of Gravity Sewers (mi)	3.00										Total Length of Gravity Interceptors Reaching Capacity by 2020 (mi)	0.00	
			Total Length of Force Mains (mi)	0.42										Total Length of Gravity Interceptors Reaching Capacity by 2030 (mi)	0.00	
														Total Length of Force Mains Reaching Capacity by 2020 (mi)	0.00	
														Total Length of Force Mains Reaching Capacity by 2030 (mi)	0.00	

Capacity Needs: **X** = Capacity reached by 2020 **Y** = Capacity reached by 2030

Table 4.7
Gravity Interceptors - Capacities and Predicted Flows

Flow Type	Segment Description	From	To	Length (ft)	Pipe Characteristics: Pipe Dia. (in)	Year Installed	Pipe Material	Nominal Capacity (mgd)	Peak Flows (mgd) / Percent Nominal Capacity: 2000		2010 UF		2030 UF		Capacity Reached	Capacity Needs	
Pump Station No. 6 Service Area																	
GR	East Interceptor - PS 1 FM to Fair Oaks Avenue	MH06-122	MH06-108A	4,813	36	1995	PVCPW	23.88	*0.54*	2%	*0.63*	3%	*0.81*	3%	> 2060		
Junction with Fair Oaks / East Monona Interceptor																	
GR	Fair Oaks/East Monona Interceptor - U/S of Starkweather Creek	MH06-209	MH06-206	1,236	15	1926	VP	1.02	*0.73*	72%	*0.72*	71%	*0.71*	70%	> 2060		
SI	Fair Oaks/East Monona Interceptor - Starkweather Creek crossing	MH06-206	MH06-205	85	14	1925	CI	1.04	*0.73*	70%	*0.72*	69%	*0.71*	68%	> 2060		
GR	Fair Oaks/East Monona Interceptor - D/S of Starkweather Creek	MH06-205	MH06-204	90	14	1925	CI	0.85	*0.73*	86%	*0.72*	85%	*0.71*	84%	> 2060		
GR	Fair Oaks/East Monona Interceptor - D/S of Starkweather Creek	MH06-204	MH06-108A	847	15	1997	PVC	1.64	*0.73*	45%	*0.72*	44%	*0.71*	43%	> 2060		
Junction with East Interceptor																	
GR	East Interceptor - Fair Oaks Avenue to Olbrich Gardens	MH06-108A	MH06-103	1,526	36	1995	PVCPW	23.88	*1.41*	6%	*1.49*	6%	*1.66*	7%	> 2060		
GR	East Interceptor - Olbrich Gardens to PS 6	MH06-103	PS6	1,483	42	1948	RCP	30.48	*1.41*	5%	*1.49*	5%	*1.66*	5%	> 2060		
FM		PS6	MH07-129	7,214	36	1948	RCP	36.5	*5.77*	16%	*5.97*	16%	*6.37*	17%	> 2060		
		Total Length of Gravity Sewers (mi)		1.91								Total Length of Gravity Interceptors Reaching Capacity by 2020 (mi)			0.00		
		Total Length of Force Mains (mi)		1.37								Total Length of Gravity Interceptors Reaching Capacity by 2030 (mi)			0.00		
												Total Length of Force Mains Reaching Capacity by 2020 (mi)			0.00		
												Total Length of Force Mains Reaching Capacity by 2030 (mi)			0.00		
Pump Station No. 7 Service Area																	
GR	FEI Gaston Road Extension	MH07-740	MH07-735	1,693	18	2008	PVC	4.39	*0.00*	0%	*0.00*	0%	*2.38*	54%	>2060		
GR	FEI Gaston Road Extension	MH07-735	PB07-734	38	21	2008	PVC	4.20	*0.00*	0%	*0.00*	0%	*2.38*	57%	>2060		
Junction with Door Creek Extension																	
GR	FEI Door Creek Extension	PB07-734	MH07-728	3,384	21	1998	PVCPW	4.36	*0.18*	4%	*2.77*	64%	*7.14*	164%	2010-2020	X	Y
GR	FEI Door Creek Extension	MH07-728	MH07-723	2,496	21	1998	PVCPW	5.41	*0.18*	3%	*2.77*	51%	*7.14*	132%	2020-2030		Y
GR	FEI Door Creek Extension	MH07-723	MH07-707	7,899	24	1998	PVCPW	5.98	*0.18*	3%	*2.77*	46%	*7.14*	119%	2020-2030		Y
GR	FEI Door Creek Extension	MH07-707	MH07-426	3,474	24	1998	PVCPW	7.12	*0.18*	3%	*2.77*	39%	*8.20*	115%	2020-2030		Y
Junction with Cottage Grove Extension																	
GR	FEI Cottage Grove Extension	MH07-437	MH07-426	5,510	18	1981	RCP(L)	2.71	*1.27*	47%	*2.20*	81%	*3.00*	111%	>2030		Y
Junction with Far East Extension																	
GR	FEI - Far East Extension	MH07-426	MH07-425	153	36	1981	RCP	12.19	*1.68*	14%	*5.31*	44%	*11.11*	91%	2030-2060		
GR	FEI - Far East Extension (Cottage Grove Ext. to I90 east R/W)	MH07-425	MH07-416	3,861	30	1981	RCP	7.49	*1.68*	22%	*5.31*	71%	*11.11*	148%	2010-2020	X	Y
GR	FEI - Far East Extension (I90 crossing)	MH07-416	MH07-415	355	42	1970	RCP	15.92	*1.68*	11%	*5.31*	33%	*11.11*	70%	2030-2060		
Junction with Far East Interceptor																	
GR	FEI - I90 west R/W to junction with NEI	MH07-415	MH07-932	8,067	42	1970	RCP	15.92	*1.96*	12%	*5.59*	35%	*11.44*	72%	2030-2060		
Junction with Northeast Interceptor																	
GR	NEI - D/S of NEI junction	MH07-932	MH07-313	14	42	1970	RCP	15.92	*26.75*	168%	*33.21*	209%	*45.50*	286%	2000	X	Y
GR	NEI - MH07-313 to SEI junction	MH07-313	MH07-215	5,591	48	1964	RCP	32.14	*26.75*	83%	*33.21*	103%	*45.50*	142%	2000-2010	X	Y
Junction with Southeast Interceptor																	
GR	Southeast Interceptor - PS 9 Force Main to Siggelkow Road	MH07-823	MH07-821	760	12	1961	AC	1.46	*0.36*	25%	*0.38*	26%	*0.42*	29%	> 2060		
SI	Southeast Interceptor - Siggelkow Road crossing	MH07-821	MH07-819	184	8	1992	DI	1.46	*0.36*	25%	*0.38*	26%	*0.42*	29%	> 2060		
GR	Southeast Interceptor - North of Siggelkow Road	MH07-819	MH07-818	357	12	1961	AC	1.46	*0.36*	25%	*0.38*	26%	*0.42*	29%	> 2060		
GR	Southeast Interceptor - North of Siggelkow Road to McFarland Court	MH07-818	MH07-810	3,201	12	1961	AC	2.36	*0.36*	15%	*0.38*	16%	*0.42*	18%	> 2060		
GR	Southeast Interceptor - McFarland Ct. to Blooming Grove Ext. junction	MH07-810	MH07-218	3,971	15	1961	AC	1.62	*0.36*	22%	*0.38*	23%	*0.42*	26%	> 2060		
Junction with SEI Blooming Grove Extension																	
GR	Southeast Interceptor - Blooming Grove Ext. junction to NEI junction	MH07-218	MH07-215	1,606	36	1961	RCP	11.4	*4.51*	40%	*6.69*	59%	*10.71*	94%	2030-2060		
Junction with Northeast Interceptor																	
GR	Southeast Interceptor - NEI junction to east of Monona Drive	MH07-215	MH07-211	2,468	60	1961	RCP	37.62	*29.44*	78%	*37.33*	99%	*52.28*	139%	2010-2020	X	Y
GR	Southeast Interceptor - East of Monona Drive to PS 7	MH07-211	PS7	5,342	60	1961	RCP	37.62	*30.09*	80%	*38.01*	101%	*53.01*	141%	2000-2010	X	Y
Junction with East Interceptor																	
GR	NEI - Between Buckeye Road and Helgesen Drive	MH07-955	MH07-954	95	48	2001	DI	40.45	*25.09*	62%	*29.32*	72%	*37.44*	93%	2030-2060		
GR	NEI - Between Buckeye Road and Helgesen Drive	MH07-954	PB07-953	40	48	2001	DI	57.2	*25.09*	44%	*29.32*	51%	*37.44*	65%	> 2060		
GR	NEI - Between Buckeye Road and Helgesen Drive	PB07-953	MH07-949	1,843	48	2005	FRP	67.6	*25.09*	37%	*29.32*	43%	*37.44*	55%	> 2060		
GR	NEI - North and south of Helgesen Drive	MH07-949	MH07-945	1,083	42	2005	FRP	50.37	*25.09*	50%	*29.32*	58%	*37.44*	74%	> 2060		
GR	NEI - Between Helgesen Drive and Pflaum Road	MH07-945	MH07-942	850	36	2005	FRP	60.47	*25.09*	41%	*29.32*	48%	*37.44*	62%	> 2060		
GR	NEI at Pflaum Road	MH07-942	MH07-939	790	42	2005	FRP	68.27	*25.09*	37%	*29.32*	43%	*37.44*	55%	> 2060		
GR	NEI - Pflaum Road to junction with FEI	MH07-939	MH07-932	2,622	54	2005	FRP	52.01	*25.09*	48%	*29.32*	56%	*37.44*	72%	> 2060		
Junction with Far East Interceptor																	

Capacity Needs: **X** = Capacity reached by 2020 **Y** = Capacity reached by 2030

Table 4.7
Gravity Interceptors - Capacities and Predicted Flows

Flow Type	Segment Description	From	To	Length (ft)	Pipe Characteristics: Pipe Dia. (in)	Year Installed	Pipe Material	Nominal Capacity (mgd)	*Peak Flows (mgd)* / Percent Nominal Capacity: 2000		2010 UF		2030 UF		Capacity Reached	Capacity Needs	
GR	SEI Blooming Grove Ext. - Millpond Road to I90 west R/W	MH07-249	MH07-242	2,794	18	1967	RCP	2.25	*0.37*	16%	*2.07*	92%	*5.21*	232%	2010-2020	X	Y
GR	SEI Blooming Grove Ext. - I90 west R/W to Marsh Road	MH07-242	MH07-231	4,974	24	1967	RCP	3.87	*0.37*	10%	*2.07*	53%	*5.21*	135%	2020-2030		Y
GR	SEI Blooming Grove Ext. - Marsh Rd. to SEI McFarland Relief junction	MH07-231	MH07-228	1,347	24	1967	RCP	5.06	*0.37*	7%	*2.07*	41%	*5.21*	103%	2020-2030		Y
Junction with McFarland Relief																	
GR	SEI Blooming Grove Ext. - McFarland Relief junction to Galleon Run	MH07-228	MH07-224	2,001	30	1967	RCP	10.26	*3.84*	37%	*6.02*	59%	*9.98*	97%	2030-2060		
GR	SEI Blooming Grove Ext. - Between Galleon Run and S. Dutch Mill Road	MH07-224	MH07-222	650	30	1967	RCP	10.26	*4.21*	41%	*6.40*	62%	*10.42*	102%	2020-2030		Y
GR	SEI Blooming Grove Ext. - East of S. Dutch Mill Road to SEI junction	MH07-222	MH07-218	1,647	36	1963	RCP	10.55	*4.21*	40%	*6.40*	61%	*10.42*	99%	2030-2060		
Junction with Southeast Interceptor																	
GR	SEI McFarland Relief - Brandenburg Way to Star Spangled Trail	MH07-517	MH07-515	392	20	1987	RCP	11.89	*3.23*	27%	*3.90*	33%	*5.02*	42%	> 2060		
GR	SEI McFarland Relief - Star Spangled Trail to Siggelkow Ext. junction	MH07-515	MH07-512	1,263	30	1987	RCP	8.79	*3.23*	37%	*3.90*	44%	*5.02*	57%	> 2060		
Junction with Siggelkow Extension																	
GR	SEI McFarland Relief - Siggelkow Ext. to Blooming Grove Ext.	MH07-512	MH07-228	5,012	30	1987	RCP	8.79	*3.46*	39%	*4.36*	50%	*5.92*	67%	> 2060		
Junction with Blooming Grove Extension																	
GR	SEI Siggelkow Extension - Red Oak Trail to Siggelkow Road	MH07-618	MH07-610	2,334	12	1996	PVC	2.12	*0.18*	8%	*0.31*	15%	*0.57*	27%	> 2060		
GR	SEI Siggelkow Extension - Siggelkow Road crossing	MH07-610	MH07-609	78	8	1996	PVC	0.72	*0.18*	25%	*0.31*	43%	*0.57*	79%	> 2060		
GR	SEI Siggelkow Ext. - Siggelkow Rd. to FEI McFarland Relief junction	MH07-609	MH07-512	2,666	12	1993	PVC	2.12	*0.18*	8%	*0.31*	15%	*0.57*	27%	> 2060		
Junction with McFarland Relief																	
GR	East Interceptor Replacement - Phase II	MH07-129	MH07-121A	3,126	36	1986	RCPWT	41.05	*7.82*	19%	*8.02*	20%	*8.42*	21%	> 2060		
GR	East Interceptor Replacement - Phase IV	MH07-121A	MH07-111J	2,851	42	1990	RCPWT	36.03	*7.82*	22%	*8.02*	22%	*8.42*	23%	> 2060		
GR	East Interceptor Replacement - Phase I	MH07-111J	MH07-111A	1,844	36	1985	RCPWT	36.01	*7.82*	22%	*8.02*	22%	*8.42*	23%	> 2060		
GR	East Interceptor Replacement - Phase III	MH07-111A	MH07-103	2,610	42	1990	DI	30.48	*7.82*	26%	*8.02*	26%	*8.42*	28%	> 2060		
GR	East Interceptor - MH07-103 to PS 7	MH07-103	PS7	989	42	1948	RCP	30	*7.82*	26%	*8.02*	27%	*8.42*	28%	>2060		
FM	PS7 to Junction at Nine Springs WWTP	PS 7	TE07A-01520	6,996	36	1948	RCP	55.00	*35.13*	64%	*42.99*	78%	*59.86*	109%	2020-2030		Y
FM	PS7 to Junction at Nine Springs WWTP	PS 7	TE07A-01520	6,996	36	1963	PCCP	65.00	*35.13*	54%	*42.99*	66%	*59.86*	92%	2030-2060		
FM	At Nine Springs WWTP	TE07A-01520	PB07A-00186	1,338	48	1963	PCCP	65.00	*35.13*	54%	*42.99*	66%	*59.86*	92%	2030-2060		
FM	At Nine Springs WWTP	PB07A-00186	Headworks	323	48	2005	DI	65.00	*35.13*	54%	*42.99*	66%	*59.86*	92%	2030-2060		
		Total Length of Gravity Sewers (mi)		19.76								Total Length of Gravity Interceptors Reaching Capacity by 2020 (mi)			4.44		
		Total Length of Force Mains (mi)		2.96								Total Length of Gravity Interceptors Reaching Capacity by 2030 (mi)			8.39		
												Total Length of Force Mains Reaching Capacity by 2020 (mi)			0.00		
												Total Length of Force Mains Reaching Capacity by 2030 (mi)			1.33		
Pump Station No. 8 Service Area																	
GR	WI Relief - Between Whitney Way and Merill Springs Road	MH02-547	MH02-546	497	24	1959	RCP	12.57	*7.42*	59%	*7.75*	62%	*8.54*	68%	> 2060		
GR	WI Relief - Between Whitney Way and Merill Springs Road	MH02-546	MH02-545	192	27	1959	RCP	8.95	*7.42*	83%	*7.75*	87%	*8.54*	95%	> 2060		
GR	WI Relief - Merill Springs Road to Maple Terrace	MH02-545	MH02-538	3,121	27	1959	RCP	8.95	*9.79*	109%	*10.22*	114%	*11.21*	125%	2000	X	Y
GR	WI Relief - Maple Terrace to Highbury Road	MH02-538	MH02-536	1,200	24	1959	RCP	8.52	*9.79*	115%	*10.22*	120%	*11.21*	132%	2000	X	Y
GR	WI Relief - Highbury Road to Joyce Erdman Place	MH02-536	MH02-535	600	21	1959	RCP	10.44	*9.79*	94%	*10.22*	98%	*11.21*	107%	2010-2020	X	Y
GR	WI Relief - Joyce Erdman Place to Shorewood Boulevard	MH02-535	MH02-532	841	21	1959	RCP	10.44	*9.79*	94%	*10.22*	98%	*11.21*	107%	2010-2020	X	Y
GR	WI Relief at Shorewood Boulevard	MH02-532	MH02-531A	65	36	1959	RCP	12.19	*9.98*	82%	*10.42*	85%	*11.40*	94%	2030-2060		
Junction with WI - Midvale Relief																	
GR	WI Relief - Midvale Relief junction to east of Highland Avenue	MH02-531A	MH02-519	4,363	36	1959	RCP	12.19	*12.58*	103%	*13.07*	107%	*14.17*	116%	2000	X	Y
GR	WI Relief - Between Highland Avenue and Walnut Street	MH02-519	MH02-518	465	36	1959	RCP	25.85	*12.58*	49%	*13.07*	51%	*14.17*	55%	> 2060		
SI	WI Relief - Walnut Street crossing	MH02-518	MH02-516	204	36	1959	RCP	12.19	*12.58*	103%	*13.07*	107%	*14.17*	116%	2000	X	Y
GR	WI Relief - Walnut Street to Campus Relief (Ph IV) junction	MH02-516	MH08-228	10	36	1959	RCP	12.19	*14.21*	117%	*14.66*	120%	*15.67*	129%	2000	X	Y
Junction with Campus Relief (Ph IV)																	
GR	WI Relief - Campus Relief (Ph IV) junction to Original West Int. junction	MH08-228	MH02-513	1,112	36	1959	RCP	12.19	*6.68*	55%	*6.89*	57%	*7.36*	60%	> 2060		
Junction with Old West Interceptor																	
GR	WI Relief - Original West Int. junction to Campus Relief (Ph II) junction	MH02-513	MH08-209	2,175	36	1959	RCP	12.19	*9.29*	76%	*9.77*	80%	*10.78*	88%	> 2060		
Junction with Campus Relief (Ph II)																	
GR	WI Relief - Between Babcock Drive and Henry Mall	MH08-209	MH08-207	625	36	1959	RCP	12.19	*7.74*	63%	*8.01*	66%	*8.59*	70%	> 2060		
Junction with Campus Relief (Ph II)																	
GR	WI Relief - Henry Mall to Randall Avenue	MH08-207	MH02-503	463	36	1959	RCP	12.19	*3.63*	30%	*3.76*	31%	*4.03*	33%	> 2060		
GR	WI Relief on Randall Avenue - Campus Drive to Engineering Drive	MH02-503	MH02-502	142	36	1959	RCP	12.19	*3.63*	30%	*3.76*	31%	*4.03*	33%	> 2060		
GR	WI Relief on Randall Avenue - Engineering Drive to Randall Relief junction	MH02-502	MH02-014A	513	36	1959	RCP	12.19	*5.34*	44%	*5.48*	45%	*5.78*	47%	> 2060		
Junction with Old West Interceptor & West Interceptor Randall Relief																	
GR	WI Midvale Relief - Midvale Boulevard to WI Relief junction	MH02-708	MH02-531A	2,653	21	1971	RCP	3.55	*3.19*	90%	*3.32*	94%	*3.57*	101%	2020-2030		Y
Junction with West Interceptor Relief																	

Capacity Needs: **X** = Capacity reached by 2020 **Y** = Capacity reached by 2030

Table 4.7
Gravity Interceptors - Capacities and Predicted Flows

Flow Type	Segment Description	From	To	Length (ft)	Pipe Characteristics Pipe Dia. (in)	Year Installed	Pipe Material	Nominal Capacity (mgd)	*Peak Flows (mgd)* / Percent Nominal Capacity 2000		2010 UF		2030 UF		Capacity Reached	Capacity Needs	
GR	WI Campus Relief (Ph IV) - Walnut Street to UW Dairy Barn	MH08-228	MH08-223	1,933	36	2005	DI	15.04	*7.53*	50%	*7.77*	52%	*8.30*	55%	> 2060		
GR	WI Campus Relief (Ph IV) - North of UW Dairy Barn	MH08-223	MH08-221	161	36	2005	DI	15.04	*9.69*	64%	*9.90*	66%	*10.39*	69%	> 2060		
GR	WI Campus Relief (Ph IV) - North of UW Dairy Barn	MH08-221	MH08-220	118	2 @ 24	2005	DI	15.64	*9.69*	62%	*9.90*	63%	*10.39*	66%	> 2060		
GR	WI Campus Relief (Ph IV) - UW Dairy Barn to Campus Relief (Ph III) junction	MH08-220	MH08-216	514	36	2005	DI	15.04	*9.69*	64%	*9.90*	66%	*10.39*	69%	> 2060		
GR	WI Campus Relief (Ph III) - South of Stock Pavilion & Babcock Hall	MH08-216	MH08-210	1,078	36	2000	DI	16.40	*9.69*	59%	*9.90*	60%	*10.39*	63%	> 2060		
GR	WI Campus Relief (Ph II) - South of Babcock Hall	MH08-210	MH08-209	64	36	2000	DI	15.04	*9.69*	64%	*9.90*	66%	*10.39*	69%	> 2060		
Junction with West Interceptor Relief																	
GR	WI Campus Relief (Ph II) - South of Babcock Hall to Material Science Bldg.	MH08-209	MH08-208	629	48	2000	FRP	34.68	*9.52*	27%	*9.87*	28%	*10.63*	31%	> 2060		
GR	WI Campus Relief (Ph II) - Campus Drive at Material Science Building	MH08-208	MH08-207	12	36	2000	DI	15.04	*9.52*	63%	*9.87*	66%	*10.63*	71%	> 2060		
Junction with West Interceptor Relief																	
GR	WI Campus Relief (Ph I) - Material Science Bldg. to Randall Relief junction	MH08-207	MH08-201	1,134	36	1999	DI	17.80	*13.64*	77%	*14.13*	79%	*15.18*	85%	> 2060		
Junction with West Interceptor - Randall Relief																	
GR	Old West Interceptor - State Crime Lab to Shorewood Boulevard	MH02-060	MH02-047	5,066	12-18	1932	VP	2.09	*0.71*	34%	*0.89*	43%	*1.25*	60%	> 2060		
GR	Old West Interceptor - Shorewood Boulevard to west of Franklin Avenue	MH02-047	MH02-041	1,914	18	1932	VP	2.71	*0.71*	26%	*0.89*	33%	*1.25*	46%	> 2060		
GR	Old West Interceptor - West of Franklin Avenue to Farley Avenue	MH02-041	MH02-038	1,063	18	1932	VP	2.71	*1.40*	52%	*1.67*	62%	*2.20*	81%	2030-2060		
GR	Old West Interceptor - Farley Avenue to Highland Avenue	MH02-038	MH02-034	1,460	18	1916	VP	1.92	*1.40*	73%	*1.67*	87%	*2.20*	115%	2010-2020	X	Y
GR	Old West Interceptor - Highland Avenue to Walnut Street	MH02-034	MH02-032	816	20	1916	VP	2.84	*2.41*	85%	*2.76*	97%	*3.47*	122%	2010-2020	X	Y
GR	Old West Interceptor - Walnut Street to West Relief junction	MH02-032	MH02-513	1,704	21	1916	VP	3.24	*2.41*	74%	*2.76*	85%	*3.47*	107%	2020-2030		Y
Junction with West Interceptor Relief																	
GR	Old West Interceptor - Babcock Hall to West Relief junction	MH02-021	MH02-014A	2,153	24	1916	CI	4.85	*3.44*	71%	*3.33*	69%	*3.11*	64%	> 2060		
Junction with West Interceptor Relief & West Interceptor Randall Relief																	
GR	WI Randall Relief - Junction with Old West Int. to jxn with Campus Relief	MH02-014A	MH08-201	29	33	1964	RCP	25.10	*7.97*	32%	*8.02*	32%	*8.15*	32%	> 2060		
Junction with West Interceptor - Campus Relief (Table 4-21)																	
GR	WI Randall Relief - South of Dayton Street to Regent Street	MH08-201	MH08-121	1,127	33	1964	RCP	25.10	*19.93*	79%	*20.45*	81%	*21.58*	86%	> 2060		
GR	WI Randall Relief - At Randall Avenue and Regent Street	MH08-121	MH08-120	16	2@30	1964	CI	21.13	*19.93*	94%	*20.45*	97%	*21.58*	102%	2020-2030		Y
GR	WI Randall Relief - Regent Street to Milton Street	MH08-120	MH08-119	473	42	1964	RCP	25.17	*19.93*	79%	*20.45*	81%	*21.58*	86%	> 2060		
GR	WI Randall Relief - Milton Street to Vilas Avenue	MH08-119	MH08-117	1,201	42	1964	RCP	25.17	*20.67*	82%	*20.45*	81%	*21.58*	86%	> 2060		
GR	WI Randall Relief - Vilas Avenue to SWI junction at Vilas Zoo	MH08-117	MH08-113	1,479	42	1964	RCP	25.17	*20.93*	83%	*20.70*	82%	*21.83*	87%	> 2060		
Junction with Southwest Interceptor																	
GR	WI Randall Relief - Through Vilas Zoo to Vilas Park Drive	MH08-113	MH08-109	1,237	48	1964	RCP	27.84	*20.75*	75%	*20.61*	74%	*21.63*	78%	> 2060		
Junction with Southwest Interceptor																	
GR	WI Randall Relief - Vilas Park Drive to Haywood Drive	MH08-109	MH08-106	1,279	48	1964	RCP	27.84	*21.07*	76%	*20.94*	75%	*21.96*	79%	> 2060		
Junction with Southwest Interceptor																	
GR	WI Randall Relief - Along Wingra Drive from Haywood Drive to PS 8	MH08-106	PS 8	3,179	48	1964	RCP	30.78	*24.90*	81%	*24.74*	80%	*25.94*	84%	> 2060		
GR	SWI North Leg - Whitney Way to Beltline Highway	MH02-189	MH02-186	846	15	1955	RCP(L)	1.89	*1.44*	76%	*1.44*	76%	*1.44*	76%	> 2060		
GR	SWI North Leg - Beltline Highway to east edge of Odana Hills GC	MH02-186	MH02-174	4,693	18	1955	RCP/AC(L)	2.46	*1.44*	59%	*1.44*	59%	*1.44*	59%	> 2060		
GR	SWI North Leg - East edge of Odana Hills GC to junction with SWI South Leg	MH02-174	MH02-173A	100	20	1955	AC	3.48	*1.44*	41%	*1.44*	41%	*1.44*	41%	> 2060		
Junction with Southwest Interceptor - South Leg																	
GR	SWI South Leg - USH 18/151 Frontage Road to Home Depot	MH02-218	MH02-215	1,134	16	2000	PVC	2.62	*0.90*	34%	*0.90*	34%	*0.89*	34%	> 2060		
GR	SWI South Leg - Home Depot to Hammersley Road	MH02-215	MH02-208	1,893	12	1955	RCP(L)	1.13	*0.90*	80%	*0.90*	80%	*0.89*	79%	> 2060		
GR	SWI South Leg - Along Pontiac Trail, Hammersley Road to Boston Court	MH02-208	MH02-203	1,606	14	1955	AC(L)	1.67	*0.90*	54%	*0.90*	54%	*0.89*	53%	> 2060		
GR	SWI South Leg - Along Pontiac Trail, Boston Court to Nokomis Court	MH02-203	MH02-202	348	12	1955	AC/VP/PVC(L)	2.45	*0.90*	37%	*0.90*	37%	*0.89*	36%	> 2060		
GR	SWI South Leg - Nokomis Court, between Pontiac trail and Odana Hills GC	MH02-202	MH02-201	315	12	1955	VP(L)	2.35	*0.90*	38%	*0.90*	38%	*0.89*	38%	> 2060		
GR	SWI South Leg - Nokomis Court extended to SWI North Leg junction	MH02-201	MH02-173A	160	12	1994	PVC	2.35	*0.90*	38%	*0.90*	38%	*0.89*	38%	> 2060		
Junction with Southwest Interceptor - North Leg																	
GR	SWI - North & South Leg junction to 1994 Replacement	MH02-173A	MH02-172	700	20	1955	AC	3.48	*2.34*	67%	*2.34*	67%	*2.33*	67%	> 2060		
GR	SWI - 1994 Replacement to Midvale Boulevard	MH02-172	MH02-171B	307	15	1994	PVC	4.87	*2.34*	48%	*2.34*	48%	*2.33*	48%	> 2060		
GR	SWI - At Midvale Boulevard	MH02-171B	MH02-171	92	15	1994	PVC	4.87	*2.64*	54%	*2.64*	54%	*2.63*	54%	> 2060		
GR	SWI - Midvale Boulevard to east along SW Bike Path	MH02-171	MH02-170	396	21	1955	RCP	3.96	*2.64*	67%	*2.64*	67%	*2.63*	66%	> 2060		
GR	SWI - East of Midvale Boulevard to Cherokee Drive & Chippewa Drive	MH02-170	MH02-163	1,950	12	1955/1994	VP/PVC	4.49	*2.64*	59%	*2.64*	59%	*2.63*	59%	> 2060		
GR	SWI - Along Cherokee Drive, Chippewa Drive to Oneida Place	MH02-163	MH02-159	695	24	1932	VP	12.31	*3.58*	29%	*3.57*	29%	*3.55*	29%	> 2060		
GR	SWI - Cherokee Drive between Oneida Place and Nakoma Road	MH02-159	MH02-157	302	18	1932	VP	13.87	*3.58*	26%	*3.57*	26%	*3.55*	26%	> 2060		
GR	SWI - Cherokee Drive between Oneida Place and Nakoma Road	MH02-157	MH02-154	380	20	1932	VP	8.99	*3.58*	40%	*3.57*	40%	*3.55*	39%	> 2060		
GR	SWI - Nakoma Road between Cherokee Drive and Spring Trail	MH02-154	MH02-150	1,021	18	1955	RCP	5.26	*3.58*	68%	*3.57*	68%	*3.55*	67%	> 2060		
GR	SWI - Nakoma Road & Spring Trail to Glenway Street	MH02-150	MH02-145	1,215	24	1955	RCP	5.84	*5.32*	91%	*5.39*	92%	*5.55*	95%	2030-2060		
GR	SWI - Along UW Arboretum from Glenway Street to Western Avenue	MH02-145	MH02-142	741	24	1955	RCP	13.00	*5.32*	41%	*5.39*	41%	*5.55*	43%	> 2060		
GR	SWI - UW Arboretum from Western Ave. to Arbor Drive & Knickerbocker St.	MH02-142	MH02-136	1,669	27	1955	RCP	5.66	*5.32*	94%	*5.39*	95%	*5.55*	98%	2030-2060		
GR	SWI - Wingra Park from Knickerbocker Street to Woodrow Street	MH02-136	MH02-133	1,161	30	1955	RCP	7.49	*5.32*	71%	*5.39*	72%	*5.55*	74%	> 2060		
GR	SWI - Along Lake Wingra, Woodrow Street to WI Randall Relief junction	MH02-133	MH08-113	3,959	30	1955	RCP	7.49	*5.40*	72%	*5.48*	73%	*5.63*	75%	> 2060		
Junction with West Interceptor Randall Relief																	
GR	SWI - At Vilas Zoo	MH08-113	MH02-124	193	30	1955	RCP	7.49	*4.05*	54%	*4.02*	54%	*4.20*	56%	> 2060		
GR	SWI - Through Vilas Zoo to Vilas Park Drive	MH02-124	MH08-109	1,060	24	1936	CI	5.06	*4.05*	80%	*4.02*	79%	*4.20*	83%	> 2060		

Capacity Needs: **X** = Capacity reached by 2020 **Y** = Capacity reached by 2030

Table 4.7
Gravity Interceptors - Capacities and Predicted Flows

Flow Type	Segment Description	From	To	Length (ft)	Pipe Characteristics: Pipe Dia. (in)	Year Installed	Pipe Material	Nominal Capacity (mgd)	*Peak Flows (mgd)* / Percent Nominal Capacity: 2000		2010 UF		2030 UF		Capacity Reached	Capacity Needs
Junction with West Interceptor Randall Relief																
GR	SWI - Vilas Park Drive to Haywood Drive	MH08-109	MH08-106	1,288	24	1936	CI	5.06	*3.72*	74%	*3.69*	73%	*3.88*	77%	> 2060	
Junction with West Interceptor Randall Relief																
FM	PS8 to 200 feet east	PS 8	RD08-13205	194	36	1964	PCCP	36.50	*25.13*	69%	*24.97*	68%	*26.17*	72%	> 2060	
FM	200 feet east of PS8 to Nine Springs WWTP	RD08-13205	PB08-00192	13,210	42	1964	PCCP	49.70	*25.13*	51%	*24.97*	50%	*26.17*	53%	> 2060	
FM	At Nine Springs WWTP	PB08-00192	Headworks	334	42	2005	DI	49.70	*25.13*	51%	*24.97*	50%	*26.17*	53%	> 2060	
		Total Length of Gravity Sewers (mi)		14.64									Total Length of Gravity Interceptors Reaching Capacity by 2020 (mi)		2.39	
		Total Length of Force Mains (mi)		2.60									Total Length of Gravity Interceptors Reaching Capacity by 2030 (mi)		3.22	
													Total Length of Force Mains Reaching Capacity by 2020 (mi)		0.00	
													Total Length of Force Mains Reaching Capacity by 2030 (mi)		0.00	
Pump Station No. 9 Service Area																
GR	SEI - USH 51 from Yahara Drive to Farwell Street	MH09-108	MH09-104	1,678	24	1961	RCP	4.13	*2.05*	50%	*2.59*	63%	*3.67*	89%	2030-2060	
GR	SEI - USH 51 from Farwell Street to Larson Beach Road	MH09-104	MH09-101	1,373	27	1961	RCP	5.66	*3.22*	57%	*3.86*	68%	*4.93*	87%	2030-2060	
GR	SEI - Larson Beach Road to PS 9	MH09-101	PS9	285	24	1961	RCP	4.62	*3.22*	70%	*3.86*	84%	*4.93*	107%	2020-2030	Y
FM	PS9 to 40 feet east	PS9	TE09-20598	40	14	1961	CI	2.8	*3.22*	115%	*3.86*	138%	*4.93*	176%	2000	X Y
FM	PS9 to SEI McFarland Relief at Brandenburg Way	TE09-20598	MH07-517	4,334	20	1987	DI	11.3	*3.22*	28%	*3.86*	34%	*4.93*	44%	> 2060	
FM	PS9 to Southeast Interceptor	TE09-20598	MH09-20594	4	10	1961	CI	2.8	*0.00*	0%	*0.00*	0%	*0.00*	0%	> 2060	
FM	PS9 to Southeast Interceptor	MH09-20594	PB09-20296	298	10	1961	AC	2.8	*0.00*	0%	*0.00*	0%	*0.00*	0%	> 2060	
FM	PS9 to Southeast Interceptor	PB09-20296	PB09-20118	178	10	1961	CI	2.8	*0.00*	0%	*0.00*	0%	*0.00*	0%	> 2060	
FM	PS9 to Southeast Interceptor	PB09-20118	PB09-19463	655	10	1961	AC	2.8	*0.00*	0%	*0.00*	0%	*0.00*	0%	> 2060	
FM	PS9 to Southeast Interceptor	PB09-19463	PB09-19199	264	10	1961	CI	2.8	*0.00*	0%	*0.00*	0%	*0.00*	0%	> 2060	
FM	PS9 to Southeast Interceptor	PB09-19199	MH07-823	798	10	1961	AC	2.8	*0.00*	0%	*0.00*	0%	*0.00*	0%	> 2060	
		Total Length of Gravity Sewers (mi)		0.63									Total Length of Gravity Interceptors Reaching Capacity by 2020 (mi)		0.00	
		Total Length of Force Mains (mi)		1.24									Total Length of Gravity Interceptors Reaching Capacity by 2030 (mi)		0.05	
													Total Length of Force Mains Reaching Capacity by 2020 (mi)		0.01	
													Total Length of Force Mains Reaching Capacity by 2030 (mi)		0.01	
Pump Station No. 10 Service Area																
GR	NEI - Near Rieder Road & Old Gate Road to Lien Road at Thierer Road	MH10-145	MH10-426	10,948	48	1969	RCP	24.55	*19.09*	78%	*22.30*	91%	*28.47*	116%	2010-2020	X Y
GR	NEI Replacement - Between Lien Road & Sycamore Avenue	MH10-426	MH10-420	1,804	48	2010	FRP	45.78	*20.06*	44%	*23.30*	51%	*29.54*	65%	>2060	
GR	NEI Replacement - North of Sycamore Avenue to NEI Lien Extension	MH10-420	MH10-419	640	54	2010	FRP	44.58	*20.06*	45%	*23.30*	52%	*29.54*	66%	>2060	
Junction with NEI Lien Extension																
GR	NEI Replacement - Sycamore Avenue crossing	MH10-419	MH10-418	546	63	2010	FRP	49.95	*20.85*	42%	*23.30*	47%	*29.54*	59%	>2060	
GR	NEI Replacement - Sycamore Avenue to NEI Junction at Wal-Mart	MH10-418	MH10-415	1,011	63	2010	FRP	49.95	*21.26*	43%	*25.44*	51%	*33.44*	67%	>2060	
Junction with Northeast Interceptor																
GR	NEI Replacement - NEI Junction at Wal-Mart to MH10-412	MH10-415	MH10-412	1,509	54	2010	FRP	29.46	*12.76*	43%	*15.26*	52%	20.06	68%	>2060	
Junction with Northeast Interceptor																
GR	NEI Replacement - MH10-412 to MH10-403	MH10-412	MH10-403	2,680	54	2010	FRP	29.46	*12.76*	43%	*15.26*	52%	20.06	68%	>2060	
GR	NEI Replacement - MH10-403 to MH10-402	MH10-403	MH10-402	360	54	2010	FRP	29.46	*12.78*	43%	*15.28*	52%	20.1	68%	>2060	
Junction with Northeast Interceptor																
GR	NEI Replacement - NEI Junction to PS 10	MH10-402	PS 10	672	54	2010	FRP	29.46	*13.04*	44%	*15.55*	53%	20.35	69%	>2060	
GR	NEI - NEI Replacement Junction at Wal-Mart to east of USH 51	MH10-112	MH10-412	1,528	48	1964	RCP	20.75	*8.50*	41%	*10.18*	49%	*13.38*	64%	>2060	
Junction with Northeast Interceptor Replacement																
GR	NEI - USH 51 & STH 30 crossing	MH10-412	MH10-104A	1,476	48	1964	RCP	20.75	*8.50*	41%	*10.18*	49%	13.38	64%	>2060	
Junction with NEI Highway 30 Extension																
GR	NEI - South of STH 30 to MH10-402	MH10-104A	MH10-402	1,463	48	1964	RCP	20.75	*8.96*	43%	*10.63*	51%	13.81	67%	>2060	
Junction with Northeast Interceptor Replacement																
GR	NEI - MH10-402 to MH10 to PS 10	MH10-402	PS 10	714	48	1964	RCP	20.75	*10.09*	49%	*11.73*	57%	*14.91*	72%	>2060	
GR	NEI Lien Interstate Extension	MH10-220	MH10-214	2,075	24	1995	PVC	12.33	*0.03*	0%	*1.30*	11%	*3.86*	31%	> 2060	
GR	NEI Lien Extension - Lien Interstate Extension to east of Zeier Road	MH10-214	MH10-212	804	24	1973	RCP	8.00	*1.27*	16%	*2.87*	36%	*5.69*	71%	> 2060	
GR	NEI Lien Extension - East of Zeier Road to NEI Replacement junction	MH10-212	MH10-419	4,831	27	1970 & 1973	RCP	7.75	*1.27*	16%	*2.87*	37%	*5.69*	73%	> 2060	
Junction with Northeast Replacement Interceptor																

Capacity Needs: **X** = Capacity reached by 2020 **Y** = Capacity reached by 2030

Table 4.7
Gravity Interceptors - Capacities and Predicted Flows

Flow Type	Segment Description	From	To	Length (ft)	Pipe Characteristics: Pipe Dia. (in)	Year Installed	Pipe Material	Nominal Capacity (mgd)	*Peak Flows (mgd)* / Percent Nominal Capacity: 2000		2010 UF		2030 UF		Capacity Reached	Capacity Needs
GR	NEI Highway 30 Ext. - Railroad crossing at Commercial Ave. (extended)	MH10-305	BD10-303X227	307	12	1966	AC	0.86	*0.75*	87%	*0.75*	87%	*0.76*	88%	> 2060	
GR	NEI Highway 30 Ext. - Bend in interceptor west of Starkweather Creek	BD10-303X227	BD10-303X202	50	12	1996	DI	0.86	*0.75*	87%	*0.75*	87%	*0.76*	88%	> 2060	
GR	NEI Highway 30 Ext. - Starkweather Creek to NEI junction	BD10-303X202	MH10-104A	1,371	16	1996	DI	1.85	*0.75*	41%	*0.75*	41%	*0.76*	41%	> 2060	
Junction with Northeast Interceptor																
FM	PS10 to Buckeye Road	PS10	BD10-17400	11,039	36	1964	PCCP	36.5	*23.13*	63%	*27.28*	75%	*35.26*	97%	2030-2060	
FM	Buckeye Road crossing	BD10-17400	MH07-955	70	36	2001	DI	36.5	*23.13*	63%	*27.28*	75%	*35.26*	97%	2030-2060	

Total Length of Gravity Sewers (mi) 6.59
Total Length of Force Mains (mi) 2.10

Total Length of Gravity Interceptors Reaching Capacity by 2020 (mi) 2.07
Total Length of Gravity Interceptors Reaching Capacity by 2030 (mi) 2.07
Total Length of Force Mains Reaching Capacity by 2020 (mi) 0.00
Total Length of Force Mains Reaching Capacity by 2030 (mi) 0.00

Pump Station No. 11 Service Area

Flow Type	Segment Description	From	To	Length (ft)	Pipe Dia. (in)	Year Installed	Pipe Material	Nominal Capacity (mgd)	2000		2010 UF		2030 UF		Capacity Reached	Capacity Needs
GR	NSVI MP Ext. - Along US 18/151 from Cottonwood Drive to CTH PD	MH11-171	MH11-169	812	42	1968	RCP	24.32	*14.12*	58%	*19.29*	79%	*28.93*	119%	2020-2030	Y
GR	NSVI MP Ext. - CTH PD from US 18/151/ to east	MH11-169	MH11-167	465	42	1965 & 1968	RCP	24.32	*14.99*	62%	*20.13*	83%	*29.76*	122%	2010-2020	X Y
GR	NSVI - CTH PD to 2001 Relocation behind Certco	MH11-167	MH11-161E	1,436	42	1965	RCP	25.17	*14.99*	60%	*20.13*	80%	*29.76*	118%	2020-2030	Y
GR	NSVI - 2001 Relocation behind Certco	MH11-161E	MH11-161A	1,146	30	2001	PVC	42.59	*14.99*	35%	*20.13*	47%	*29.76*	70%	> 2060	
GR	NSVI - South of Chalet Gardens	MH11-161A	MH11-159	1,321	36	1965	RCP	27.25	*14.99*	55%	*20.13*	74%	*29.76*	109%	2020-2030	Y
GR	NSVI - Chalet Gardens to Allied Drive	MH11-159	MH11-158	340	36	1965	RCP	27.25	*15.91*	58%	*20.99*	77%	*30.53*	112%	2020-2030	Y
GR	NSVI - South of Crescent Road between Allied Drive & Red Arrow Trail	MH11-158	MH11-156	1,103	30	1965	RCP	36.04	*15.91*	44%	*20.99*	58%	*30.53*	85%	> 2060	
GR	NSVI - Through Dunn's Marsh to east of Seminole Highway	MH11-156	MH11-151A	2,220	42	1965	RCP	29.07	*15.91*	55%	*20.99*	72%	*30.53*	105%	2020-2030	Y
GR	NSVI - East of Seminole Highway to Ashbourne Lane	MH11-151A	MH11-145	3,784	42	1965	RCP	29.07	*16.23*	56%	*21.39*	74%	*31.09*	107%	2020-2030	Y
GR	NSVI - Ashbourne Lane to Longford Terrace	MH11-145	MH11-141	1,558	36	1965	RCP	37.81	*19.82*	52%	*25.03*	66%	*34.91*	92%	2030-2060	
GR	NSVI - Longford Terrace to west of High Ridge Trail (extended)	MH11-141	MH11-137	1,648	30	1965	RCP	35.75	*19.82*	55%	*25.03*	70%	*34.91*	98%	2030-2060	
GR	NSVI - High Ridge Trail (extended) to east of Fish Hatchery Road	MH11-137	MH11-129	3,995	33	1965	RCP	31.31	*19.82*	63%	*25.03*	80%	*34.91*	111%	2020-2030	Y
GR	NSVI - N/S segment through marsh 1000 feet east of Fish Hatchery Road	MH11-129	MH11-127	733	36	1965	RCP	35.00	*19.82*	57%	*25.03*	72%	*34.91*	100%	2030-2060	
GR	NSVI - E/W segment through marsh to NSVI Syene Ext. junction	MH11-127	MH11-116A	4,855	54	1965	RCP	31.12	*19.82*	64%	*25.03*	80%	*34.91*	112%	2020-2030	Y
Junction with NSVI - Syene Extension																
GR	NSVI - Syene Road to west of Highway 14	MH11-116A	MH11-111A	2,788	54	1965	RCP	31.12	*20.53*	66%	*25.74*	83%	*35.63*	114%	2020-2030	Y
GR	NSVI - Highway 14 crossing to Highway 14 Ext. junction	MH11-111A	MH11-106A	2,716	54	1965	RCP	31.12	*20.58*	66%	*26.40*	85%	*37.39*	120%	2010-2020	X Y
Junction with NSVI - Highway 14 Extension																
GR	NSVI - Highway 14 Ext. junction to east to MH11-104	MH11-106A	MH11-104	1,689	54	1965	RCP	31.12	*21.29*	68%	*27.08*	87%	*38.03*	122%	2010-2020	X Y
GR	NSVI - MH11-104 to NSVI Waubesa Ext. junction at PS 11	MH11-104	PS11	1,525	54	1965	RCP	31.12	*21.70*	70%	*27.65*	89%	*38.90*	125%	2010-2020	X Y
Junction with NSVI - Waubesa Extension																
GR	NSVI Syene Ext. - Along Syene Road from Post Road to south to MH11-304	MH11-306	MH11-304	223	12	1975	RCP	2.12	*1.15*	54%	*1.20*	57%	*1.30*	61%	> 2060	
GR	NSVI Syene Ext. - Along Syene Road from MH11-304 to NSVI junction	MH11-304	MH11-116A	1,599	16	1975	RCP	2.8	*1.15*	41%	*1.20*	43%	*1.30*	46%	> 2060	
Junction with Nine Springs Valley Interceptor																
GR	NSVI Hwy 14 Ext. - Beltline Highway to Ski Court	MH11-423	MH11-416	1,929	10	1977	PVC	1.17	*0.83*	71%	*0.84*	72%	*0.86*	74%	> 2060	
GR	NSVI Hwy 14 Ext. - Ski Court to Ski Lane & USH 14	MH11-416	MH11-414	719	12	1977	PVC	1.33	*0.83*	62%	*0.84*	63%	*0.86*	65%	> 2060	
GR	NSVI Hwy 14 Ext. - Pheasant Ridge Trail to Ski Lane	MH11-414C	MH11-414	834	10	1977	PVC	1.31	*0.01*	1%	*0.01*	1%	*0.01*	1%	> 2060	
GR	NSVI Hwy 14 Ext. - Ski Lane & USH 14 to Clausen Street	MH11-414	MH11-410	1,190	15	1977	PVC	1.97	*0.83*	42%	*0.84*	43%	*0.86*	44%	> 2060	
GR	NSVI Hwy 14 Ext. - Clausen Street to MH11-402, 1800 feet east of USH 14	MH11-410	MH11-402	2,385	15	1977	PVC	2.56	*1.15*	45%	*1.15*	45%	*1.16*	45%	> 2060	
GR	NSVI Hwy 14 Ext. - MH11-402 to NSVI junction	MH11-402	MH11-106A	491	15	1977	PVC	3.04	*1.15*	38%	*1.15*	38%	*1.16*	38%	> 2060	
Junction with Nine Springs Valley Interceptor																
GR	NSVI Waubesa Ext. - Meadowview Road (extended) to north	MH11-226	MH11-223	992	15	1971	RCP	1.67	*0.46*	28%	*0.47*	28%	*0.50*	30%	> 2060	
GR	NSVI Waubesa Ext. - 700 feet east of Lake Farm Road to Lake Farm Road	MH11-223	MH11-221	696	18	1971	RCP	2.8	*0.46*	16%	*0.47*	17%	*0.50*	18%	> 2060	
GR	NSVI Waubesa Ext. - Lake Farm Road to Meadowview Road	MH11-221	MH11-212	3,506	21	1971	RCP	3.24	*0.46*	14%	*0.47*	15%	*0.50*	15%	> 2060	
GR	NSVI Waubesa Ext. - Meadowview Road to NSVI junction at PS 11	MH11-212	PS11	4,317	27	1971	RCP	6.33	*0.46*	7%	*0.47*	7%	*0.50*	8%	> 2060	
Junction with Nine Springs Valley Interceptor																
FM	PS11 to Nine Springs WWTP	PS11	PB11-XXXX	4,081	36	1965	PCCP	36.5	*21.98*	60%	*27.92*	76%	*39.17*	107%	2020-2030	Y
FM	At Nine Springs WWTP	PB11-XXXX	Headworks	92	36	2006	DI	36.5	*21.98*	60%	*27.92*	76%	*39.17*	107%	2020-2030	Y

Total Length of Gravity Sewers (mi) 10.04
Total Length of Force Mains (mi) 0.79

Total Length of Gravity Interceptors Reaching Capacity by 2020 (mi) 1.21
Total Length of Gravity Interceptors Reaching Capacity by 2030 (mi) 5.29
Total Length of Force Mains Reaching Capacity by 2020 (mi) 0.00
Total Length of Force Mains Reaching Capacity by 2030 (mi) 0.79

Capacity Needs: **X** = Capacity reached by 2020 **Y** = Capacity reached by 2030

Table 4.7
Gravity Interceptors - Capacities and Predicted Flows

Flow Type	Segment Description	From	To	Length (ft)	Pipe Characteristics: Pipe Dia. (in)	Year Installed	Pipe Material	Nominal Capacity (mgd)	Peak Flows (mgd) / Percent Nominal Capacity: 2000		2010 UF		2030 UF		Capacity Reached	Capacity Needs
Pump Station No. 12 Service Area																
GR	NSVI MP Ext. - PS 16 FM discharge to Gammon Rd. & Mineral Point Rd.	MH12-177	MH12-176	400	33	1968	RCP	17.42	*5.67*	33%	*8.30*	48%	*10.24*	59%	> 2060	
GR	NSVI MP Ext. - Gammon & Mineral Point Roads to Beltline Highway	MH12-176	MH12-166	3,920	33	1968	RCP	17.42	*7.42*	43%	*9.97*	57%	*11.90*	68%	> 2060	
GR	NSVI MP Ext. - Beltline Highway crossing to Seybold Road	MH12-166	MH12-164	732	30	1968	RCP	17.77	*7.42*	42%	*9.97*	56%	*11.90*	67%	> 2060	
GR	NSVI MP Ext. - Seybold Road to Greentree Landfill	MH12-164	MH12-157	2,942	30	1968	RCP	17.77	*8.15*	46%	*10.66*	60%	*12.58*	71%	> 2060	
GR	NSVI MP Ext. - Greentree Landfill	MH12-157	MH12-156	544	30	1968	RCP	17.77	*9.18*	52%	*11.76*	66%	*13.86*	78%	> 2060	
GR	NSVI MP Ext. - Through Greentree Landfill & Elver Park to Midtown Ext. junction	MH12-156	MH12-133	10,101	36	1968	RCP	21.11	*9.18*	43%	*11.76*	56%	*13.86*	66%	> 2060	
Junction with Midtown Extension																
GR	NSVI MP Ext. - Midtown Ext. junction to East Pass	MH12-133	MH12-121	5,740	36	1968	RCP	21.11	*9.49*	45%	*13.76*	65%	*17.06*	81%	> 2060	
GR	NSVI MP Ext. - East Pass to Maple Grove Road & Nesbitt Road	MH12-121	MH12-112	4,284	36	1968	RCP	21.11	*12.16*	58%	*16.61*	79%	*20.46*	97%	> 2060	
GR	NSVI MP Ext. - Maple Grove & Nesbitt Rd. to PS 17 FM junction at USH 18/151	MH12-112	MH12-110	970	48	1968	RCP	22.73	*12.16*	53%	*16.61*	73%	*20.46*	90%	> 2060	
Junction with PS 17 Force Main																
GR	NSVI MP Ext. - PS 17 FM junction to MH12-101 at PS 12	MH12-110	MH12-101	3,484	48	1968	RCP	22.73	*13.97*	61%	*19.09*	84%	*28.64*	126%	2010-2020	X Y
GR	NSVI MP Ext. - MH12-101 to PS 12	MH12-101	PS12	38	48	1968	RCP	22.73	*14.12*	62%	*19.29*	85%	*28.93*	127%	2010-2020	X Y
GR	NSVI Midtown Ext. - Hawks Landing to CTH M crossing	MH12-220	MH12-210	3,771	24	1999	PVC	12.21	*0.04*	0%	*1.81*	15%	*2.24*	18%	> 2060	
GR	NSVI Midtown Ext. - CTH M crossing to MH12-207	MH12-210	MH12-207	1,505	24	1999	PVC	13.38	*0.04*	0%	*2.35*	18%	*3.86*	29%	> 2060	
GR	NSVI Midtown Ext. - MH12-207 to NSVI junction	MH12-207	MH12-133	3,050	30	1999	PVC	14.69	*0.04*	0%	*2.35*	16%	*3.86*	26%	> 2060	
Junction with Nine Springs Valley Interceptor																
FM	PS12 to USH 18/151 at Cottonwood Drive	PS12	MH11-171	4,786	36	1968	PCCP	36.50	*14.12*	39%	*19.29*	53%	*28.93*	79%	> 2060	
		Total Length of Gravity Sewers (mi)		7.86									Total Length of Gravity Interceptors Reaching Capacity by 2020 (mi)		0.67	
		Total Length of Force Mains (mi)		0.91									Total Length of Gravity Interceptors Reaching Capacity by 2030 (mi)		0.67	
													Total Length of Force Mains Reaching Capacity by 2020 (mi)		0.00	
													Total Length of Force Mains Reaching Capacity by 2030 (mi)		0.00	
Pump Station No. 13 Service Area																
GR	NEI WD Ext. - MH13-137 on Golf Parkway to Sherman Avenue	MH13-137	MH13-132	2,059	48	1971	RCP	20.75	*11.72*	56%	*13.49*	65%	*16.90*	81%	> 2060	
GR	NEI WD Ext. - Sherman Avenue to railroad, south of CTH CV	MH13-132	MH13-122A	4,397	48	1971	RCP	20.75	*12.01*	58%	*13.82*	67%	*17.31*	83%	2030-2060	
GR	NEI WD Ext. - West of railroad, south of CTH CV	MH13-122A	MH13-116H	153	48	1971	RCP	20.75	*16.94*	82%	*18.83*	91%	*22.52*	109%	2020-2030	Y
GR	NEI - Airport Relocation	MH13-116H	MH13-116A	1,989	48	2006 & 2007	FRP	34.68	*16.94*	49%	*18.83*	54%	*22.52*	65%	> 2060	
GR	NEI Truax Ext. - To east across Airport lands to easterly perimeter road	MH13-116A	MH13-105A	5,168	48	1969	RCP(L)	26.66	*16.94*	64%	*18.83*	71%	*22.52*	84%	> 2060	
GR	NEI Truax Ext. - Across easterly Airport perimeter road	MH13-105A	MH13-105	125	48	1969	RCP(L)	26.66	*17.00*	64%	*20.00*	75%	*25.77*	97%	2030-2060	
GR	NEI Truax Ext. - Across Airport lands from Starkweather Creek to PS 13	MH13-105	PS13	1,758	48	1969	RCP	24.55	*17.00*	69%	*20.00*	81%	*25.77*	105%	2020-2030	Y
FM	PS 13 to near Rieder Road & Old Gate Road	PS13	MH10-145	2,588	36	1969	PCCP	36.50	*17.00*	47%	*20.00*	55%	*25.77*	71%	> 2060	
		Total Length of Gravity Sewers (mi)		2.96									Total Length of Gravity Interceptors Reaching Capacity by 2020 (mi)		0.00	
		Total Length of Force Mains (mi)		0.49									Total Length of Gravity Interceptors Reaching Capacity by 2030 (mi)		0.36	
													Total Length of Force Mains Reaching Capacity by 2020 (mi)		0.00	
													Total Length of Force Mains Reaching Capacity by 2030 (mi)		0.00	
Pump Station No. 14 Service Area																
GR	NEI DeForest Ext. - N. Main Street to Mayapple Circle	MH14-209	MH14-196	4,386	21	1971	RCP	3.39	*1.81*	53%	*2.00*	59%	*2.36*	70%	> 2060	
GR	NEI DeForest Ext. - Mayapple Circle to Riverview Court	MH14-196	MH14-193	1,203	21	1971	RCP	3.39	*2.69*	79%	*2.99*	88%	*3.61*	106%	2020-2030	Y
GR	NEI DeForest Ext. - Riverview Court to west of River Road	MH14-193	MH14-182	4,062	21	1971	RCP	5.51	*2.86*	52%	*3.24*	59%	*4.00*	73%	> 2060	
GR	NEI DeForest Ext. - West of River Road to MH14-171	MH14-182	MH14-171	5,724	21	1971	RCP	5.51	*2.97*	54%	*3.44*	62%	*4.32*	78%	2030-2060	
GR	NEI DeForest Ext. - MH14-171 to MH14-166 near Paradise Circle	MH14-171	MH14-166	2,351	21	1971	RCP	5.51	*3.13*	57%	*3.60*	65%	*4.45*	81%	2030-2060	
GR	NEI DeForest Ext. - MH14-166 near Paradise Circle to MH14-165	MH14-166	MH14-165	488	21	1971	RCP	5.51	*3.76*	68%	*4.33*	79%	*5.35*	97%	2030-2060	
GR	NEI DeForest Ext. - MH14-165 to MH14-162 near Diamond Drive	MH14-165	MH14-162	1,401	24	1971	RCP	7.01	*3.76*	54%	*4.33*	62%	*5.35*	76%	2030-2060	
GR	NEI DeForest Ext. - MH14-162 near Diamond Drive to Windsor Road	MH14-162	MH14-156	2,687	24	1971	RCP	7.01	*3.81*	54%	*4.48*	64%	*5.72*	82%	2030-2060	
GR	NEI DeForest Ext. - Windsor Road to Lake Windsor GC	MH14-156	MH14-145	4,625	27	1971	RCP	9.17	*4.62*	50%	*5.27*	57%	*6.52*	71%	> 2060	
GR	NEI DeForest Ext. - Lake Windsor GC to I90/94	MH14-145	MH14-143	964	30	1971	RCP	9.18	*4.62*	50%	*5.27*	57%	*6.52*	71%	> 2060	
GR	NEI DeForest Ext. - I90/94 to Highway 19 Extension junction	MH14-143	MH14-134	4,895	36	1971	RCP	9.63	*4.77*	50%	*5.47*	57%	*6.83*	71%	> 2060	
Junction with Highway 19 Extension																
GR	NEI DeForest Ext. - NEI Hwy 19 Ext. junction to NEI Waunakee Ext. junction	MH14-134	MH14-102	16,679	36	1971	RCP	9.63	*5.57*	58%	*6.60*	69%	*8.58*	89%	2030-2060	
Junction with Waunakee Extension																

Capacity Needs: **X** = Capacity reached by 2020 **Y** = Capacity reached by 2030

Table 4.7
Gravity Interceptors - Capacities and Predicted Flows

Flow Type	Segment Description	From	To	Length (ft)	Pipe Characteristics: Pipe Dia. (in)	Year Installed	Pipe Material	Nominal Capacity (mgd)	*Peak Flows (mgd)* / Percent Nominal Capacity: 2000		2010 UF		2030 UF		Capacity Reached	Capacity Needs
GR	NEI Highway 19 Ext. - North across Highway 19, east of CTH CV	MH14-416	MH14-415	193	12	1971	RCP	1.15	*0.17*	15%	*0.26*	22%	*0.44*	38%	> 2060	
GR	NEI Highway 19 Ext. - Along Hwy 19 across I90/94	MH14-415	MH14-411	1,619	15	1971	RCP	2.21	*0.81*	37%	*1.23*	56%	*2.08*	94%	2030-2060	
GR	NEI Highway 19 Ext. - South across Highway 19	MH14-411	MH14-409	622	15	1971	RCP	3.23	*0.81*	25%	*1.23*	38%	*2.08*	64%	> 2060	
GR	NEI Highway 19 Ext. - South of Highway 19 between IH90/94 & DeForest Ext.	MH14-409	MH14-407	771	18	1971	RCP	3.32	*0.81*	24%	*1.23*	37%	*2.08*	63%	> 2060	
GR	NEI Highway 19 Ext. - South of Highway 19 between IH90/94 & DeForest Ext.	MH14-407	MH14-134	3,059	18	1971	RCP	2.35	*0.81*	34%	*1.23*	52%	*2.08*	89%	2030-2060	
Junction with DeForest Extension																
GR	NEI Waunakee Ext. - MH14-359 to MH14-358	MH14-359	MH14-358	494	24	1971	RCP	5.47	*2.10*	38%	*2.49*	46%	*3.25*	59%	> 2060	
GR	NEI Waunakee Ext. - MH14-362 to MH14-358	MH14-362	MH14-358	775	10	1971	RCP	1.54	*1.34*	87%	*1.42*	92%	*1.58*	103%	2020-2030	Y
GR	NEI Waunakee Ext. - MH14-358 to Division Street	MH14-358	MH14-356	674	24	1971	RCP	5.47	*3.45*	63%	*3.91*	71%	*4.69*	86%	2030-2060	
GR	NEI Waunakee Ext. - Division Street to near Woodland & Manchester	MH14-356	MH14-345	4,659	24	1971	RCP	5.85	*4.45*	76%	*5.33*	91%	*7.03*	120%	2010-2020	X Y
GR	NEI Waunakee Ext. - Near Woodland & Manchester to MH14-338	MH14-345	MH14-338	2,859	21	1971	RCP	6.31	*4.45*	71%	*5.33*	84%	*7.03*	111%	2020-2030	Y
GR	NEI Waunakee Ext. - MH14-338 to MH14-333 near Eldorado Court	MH14-338	MH14-333	2,110	21	1971	RCP	7.99	*4.45*	56%	*5.33*	67%	*7.03*	88%	2030-2060	
GR	NEI Waunakee Ext. - MH14-133 near Eldorado Ct. to MH14-323 near Kennedy Rd.	MH14-333	MH14-323	4,889	30	1971	RCP	7.01	*4.45*	63%	*5.33*	76%	*7.03*	100%	2020-2030	Y
GR	NEI Waunakee Ext. - MH14-323 near Kennedy Road to CTH M & Hwy 113	MH14-323	MH14-315	4,055	30	1971	RCP	7.01	*4.86*	69%	*5.82*	83%	*7.65*	109%	2020-2030	Y
GR	NEI Waunakee Ext. - CTH M & Hwy 113 to near DeForest junction	MH14-315	MH14-301	5,251	30	1971	RCP	9.18	*5.46*	59%	*6.42*	70%	*8.28*	90%	2030-2060	
GR	NEI Waunakee Ext. - MH14-301 to DeForest junction	MH14-301	MH14-102	248	30	1971	RCP	26.23	*5.46*	21%	*6.42*	24%	*8.28*	32%	> 2060	
Junction with DeForest Extension																
GR	NEI WD Ext. - Yahara River crossing to near PS 14	MH14-102	MH14-101	1,873	42	1971	RCP	20.55	*9.88*	48%	*11.68*	57%	*15.12*	74%	> 2060	
GR	NEI WD Ext. - MH14-101 to PS 14	MH14-101	PS14	34	42	1971	RCP	20.55	*11.00*	54%	*12.77*	62%	*16.18*	79%	> 2060	
FM	PS14 to Comanche Way	PS14	TE14-11057	3,108	30	1971	PCCP	25.40	*11.00*	43%	*12.77*	50%	*16.18*	64%	> 2060	
FM	Comanche Way to MH13-137 on Golf Parkway	TE14-11057	MH13-137	1,358	30	1971	PCCP	25.40	*11.72*	46%	*13.49*	53%	*16.90*	67%	> 2060	
		Total Length of Gravity Sewers (mi)		15.84							Total Length of Gravity Interceptors Reaching Capacity by 2020 (mi)				0.88	
		Total Length of Force Mains (mi)		0.85							Total Length of Gravity Interceptors Reaching Capacity by 2030 (mi)				3.49	
											Total Length of Force Mains Reaching Capacity by 2020 (mi)				0.00	
											Total Length of Force Mains Reaching Capacity by 2030 (mi)				0.00	
Pump Station No. 15 Service Area																
FM	At Westport No. 2 Lift Station in Mendota County Park	MHWP-00005	TEWP-04470	5	6	1966	CI	1.01	*0.59*	58%	*1.02*	101%	*1.87*	185%	> 2060	
FM	Force main from Westport LS in Mendota County Park to near Waconia Lane	MHWP-04488	MH05-119	2,585	14	1966	AC	5.50	*0.59*	11%	*1.02*	19%	*1.87*	34%	> 2060	
GR	WI West Point Ext. - Near Waconia Lane to Roosevelt St., east of Baskerville Ave.	MH05-119	MH05-117	584	18	1966	AC	3.39	*0.59*	17%	*1.02*	30%	*1.87*	55%	> 2060	
GR	WI West Point Ext. - Along Rossevelt Street towards Baskerville Avenue	MH05-117	MH05-116	108	18	1966	AC	7.50	*0.59*	8%	*1.02*	14%	*1.87*	25%	> 2060	
SI	WI West Point Ext. - Siphon underneath Pheasant Branch Creek	MH05-116	MH05-115	2,099	14	1957 & 1966	RCP/AC	3.43	*1.50*	44%	*2.10*	61%	*3.30*	96%	2030-2060	
GR	West Int. Ext. - Across Allen Boulevard on Century Avenue	MH05-115	MH05-113	769	18	1957	RCP	5.12	*1.50*	29%	*2.10*	41%	*3.30*	64%	> 2060	
GR	West Int. Ext. - Century Avenue to north of Middleton Springs Drive	MH05-113	MH05-112A	227	24	1957	RCP	5.85	*4.74*	81%	*5.14*	88%	*5.93*	101%	2020-2030	Y
GR	West Int. Ext. - Near Middleton Springs Drive	MH05-112A	MH15-113	10	30	1997	RCP	8.79	*4.74*	54%	*5.14*	58%	*5.93*	67%	> 2060	
GR	West Int. Ext. - Near Middleton Springs Drive to Lakeview Park	MH15-113	MH15-104	2,248	36	2007	PVC	19.05	*4.74*	25%	*5.14*	27%	*5.93*	31%	> 2060	
GR	West Int. Ext. - Lakeview Park to Mendota Avenue	MH15-104	MH15-101	991	42	2007	PVC	25.50	*4.74*	19%	*5.14*	20%	*5.93*	23%	> 2060	
GR	West Int. Ext. - Along Mendota Avenue between Gateway St. & Allen Blvd.	MH05-106	MH15-101	31	30	1999	PVC	10.60	*5.40*	51%	*5.79*	55%	*6.56*	62%	> 2060	
GR	West Int. Ext. - Along Mendota Avenue between Gateway St. & Allen Blvd.	MH15-101	MH05-105	529	30	1999	PVC	10.60	*5.40*	51%	*5.79*	55%	*6.56*	62%	> 2060	
GR	West Int. Ext. - Along Allen Boulevard from Mendota Avenue to near PS 15	MH05-105	MH05-103	808	30	1957	RCP	7.01	*5.40*	77%	*5.79*	83%	*6.56*	94%	> 2060	
GR	West Int. Ext. - Gateway Street to near PS 15	MH05-025A	MH05-103	880	12	1931	CI	2.06	*0.02*	1%	*0.02*	1%	*0.02*	1%	> 2060	
GR	West Int. Ext. - Allen Boulevard crossing near PS 15	MH05-103	MH05-102A	147	30	1957	RCP	7.01	*5.42*	77%	*5.81*	83%	*6.58*	94%	> 2060	
GR	West Int. Ext. - MH05-102A in Marshall Park to PS 15	MH05-102A	PS15	130	30	1974	RCP	8.79	*5.42*	62%	*5.89*	67%	*6.65*	76%	> 2060	
FM	PS15 to west	PS15	BD15-00000	10	24	1972	DI	16.20	*5.42*	33%	*5.89*	36%	*6.65*	41%	> 2060	
FM	PS15 to south along Allen Boulevard	BD15-00000	BD15-00489	546	24	1981	DI	16.20	*5.42*	33%	*5.89*	36%	*6.65*	41%	> 2060	
FM	PS15 to near intersection of Allen Boulevard & University Avenue	BD15-00489	TE15-01350	804	24	1972	DI	16.20	*5.42*	33%	*5.89*	36%	*6.65*	41%	> 2060	
FM	PS15 FM diversion to PS16	TE15-01350	RD15D-05583	17	24	1982	DI	25.40	*0.00*	0%	*0.00*	0%	*0.00*	0%	> 2060	
FM	PS15 FM diversion to PS16	RD15D-05583	MH16-105	4,871	30	1982	PCCP	25.40	*0.00*	0%	*0.00*	0%	*0.00*	0%	> 2060	
GR	PS 15 FM diversion - Across Stonefield Park and Elm Lawn School to MH16-102	MH16-105	MH16-102	833	30	1982	PCCP	44.02	*0.00*	0%	*0.00*	0%	*0.00*	0%	> 2060	
GR	PS 15 FM diversion - MH16-102 to PS 16	MH16-102	PS 16	30	36	1981	DI	27.25	*0.00*	0%	*0.00*	0%	*0.00*	0%	> 2060	
FM	Near Allen Blvd. & University Ave. to Thorstrand Rd. & University Ave.	TE15-01350	BD15-02421	1,071	24	1972	DI	16.20	*5.42*	33%	*5.89*	36%	*6.65*	41%	> 2060	

Capacity Needs: **X** = Capacity reached by 2020 **Y** = Capacity reached by 2030

Table 4.7
Gravity Interceptors - Capacities and Predicted Flows

Flow Type	Segment Description	From	To	Length (ft)	Pipe Characteristics: Pipe Dia. (in)	Year Installed	Pipe Material	Nominal Capacity (mgd)	*Peak Flows (mgd)* / Percent Nominal Capacity: 2000		2010 UF		2030 UF		Capacity Reached	Capacity Needs
FM	Thorstrand Rd. & University Ave. to Spring Harbor Park	BD15-02421	RD15-07254	4,837	20	1972	DI	11.30	*5.42*	48%	*5.89*	52%	*6.65*	59%	> 2060	
FM	Spring Harbor Park	RD15-07254	MH15-07264	10	24	1972	DI	16.20	*5.42*	33%	*5.89*	36%	*6.65*	41%	> 2060	
FM	Spring Harbor Park	MH15-07264	TE05-22376	8	24	1959	DI	16.20	*5.42*	33%	*5.89*	36%	*6.65*	41%	> 2060	
Junction with PS 5 force main																

Total Length of Gravity Sewers (mi) 1.97
Total Length of Force Mains (mi) 2.80

Total Length of Gravity Interceptors Reaching Capacity by 2020 (mi) 0.00
Total Length of Gravity Interceptors Reaching Capacity by 2030 (mi) 0.04
Total Length of Force Mains Reaching Capacity by 2020 (mi) 0.00
Total Length of Force Mains Reaching Capacity by 2030 (mi) 0.00

Pump Station No. 16 Service Area

Flow Type	Segment Description	From	To	Length (ft)	Pipe Dia. (in)	Year Installed	Pipe Material	Nominal Capacity (mgd)	2000		2010 UF		2030 UF		Capacity Reached	Capacity Needs
GR	WI Esser Pond Ext. - West Beltline crossing	MH05-317	MH05-315	638	21	1986	RCP	7.24	*2.85*	39%	*4.32*	60%	*6.74*	93%	> 2060	
GR	WI Esser Pond Ext. - West Beltline to High Point Road & Parmenter Street	MH05-315	MH05-310	1,002	18	1978	RCP	6.18	*2.85*	46%	*4.32*	70%	*6.74*	109%	2020-2030	Y
GR	WI Esser Pond Ext. - High Point Rd. & Parmenter St. to Westfield Rd. & Voss Pkwy.	MH05-310	MH05-306	824	18	1978	RCP	7.74	*2.85*	37%	*4.32*	56%	*6.74*	87%	> 2060	
GR	WI Esser Pond Ext. - Along Voss Pkwy. from Westfield Rd. to Middleton St.	MH05-306	MH05-236	1,771	24	1978	RCP	6.03	*2.85*	47%	*4.32*	72%	*6.74*	112%	2020-2030	Y
Junction with WI Gammon Extension																
GR	WI Gammon Ext. - Middleton Street from Middleton city limit to Voss Parkway	MH05-240	MH05-236	1,252	24	1966	RCP	4.62	*2.78*	60%	*4.60*	100%	*4.32*	94%	> 2060	
GR	WI Gammon Ext. - Voss Parkway & Middleton Street	MH05-236	MH16-211	12	24	1966	RCP	4.62	*5.43*	118%	*8.08*	175%	*10.03*	217%	2000	X Y
GR	WI Gammon Ext. - Voss Parkway between Middleton Street & Shirley Street	MH16-211	MH16-210	282	36	2002	PVC	17.64	*5.43*	31%	*8.08*	46%	*10.03*	57%	> 2060	
GR	WI Gammon Ext. - Voss Pkwy. & Shirley St. to Fortune Dr. & Gammon Rd.	MH16-210	MH16-202	1,734	36	2002	PVC	17.64	*5.61*	32%	*8.24*	47%	*10.19*	58%	> 2060	
GR	WI Gammon Ext. - Fortune Drive & Gammon Road to PS 16	MH16-202	PS16	228	36	1981	DI	15.54	*5.61*	36%	*8.24*	53%	*10.19*	66%	> 2060	
GR	PS 15 FM - Across Stonefield Park and Elm Lawn School to MH16-102	MH16-105	MH16-102	833	30	1982	PCCP	44.02	*0.00*	0%	*0.00*	0%	*0.00*	0%	> 2060	
GR	PS 16 - MH16-102 to PS 16	MH16-102	PS 16	30	36	1981	DI	27.25	*0.08*	0%	*0.08*	0%	*0.08*	0%	> 2060	
FM	PS 16 to Gammon Road	PS16	BD16-00162	162	36	1981	DI	36.50	*5.67*	16%	*8.30*	23%	*10.24*	28%	> 2060	
FM	Gammon Road - PS16 to Old Sauk Road	BD16-00162	PB16-05500	4,561	36	1979	DI	36.50	*5.67*	16%	*8.30*	23%	*10.24*	28%	> 2060	
FM	Gammon Road - Old Sauk Road to 600' north of Colony Drive	PB16-05500	MH16-03385	2,491	36	1980	DI	36.50	*5.67*	16%	*8.30*	23%	*10.24*	28%	> 2060	
FM	Gammon Road - 600' north of Colony Drive to NSVI Mineral Point Extension	MH16-03385	MH12-177	2,965	30	1980	DI	25.40	*5.67*	22%	*8.30*	33%	*10.24*	40%	> 2060	
Junction with Nine Springs Valley Interceptor																

Total Length of Gravity Sewers (mi) 1.63
Total Length of Force Mains (mi) 1.93

Total Length of Gravity Interceptors Reaching Capacity by 2020 (mi) 0.00
Total Length of Gravity Interceptors Reaching Capacity by 2030 (mi) 0.53
Total Length of Force Mains Reaching Capacity by 2020 (mi) 0.00
Total Length of Force Mains Reaching Capacity by 2030 (mi) 0.00

Pump Station No. 17 Service Area

Flow Type	Segment Description	From	To	Length (ft)	Pipe Dia. (in)	Year Installed	Pipe Material	Nominal Capacity (mgd)	2000		2010 UF		2030 UF		Capacity Reached	Capacity Needs
GR	LBMC Int. (Ph II) - Northern Lights Road & Nine Mound Road to Basswood Ave.	MH17-146	MH17-137	2,968	30	2008	PVC	15.15	*1.00*	7%	*1.73*	11%	*9.04*	60%	> 2060	
GR	LBMC Int. (Ph II) - Basswood Avenue to Edward Street	MH17-137	MH17-129	2,288	30	2008	PVC	24.93	*1.00*	4%	*1.73*	7%	*9.04*	36%	> 2060	
GR	LBMC Int. (Ph I/II) - Edward Street to south	MH17-129	MH17-127	330	27	2006/2008	PVCPW	16.21	*1.00*	6%	*1.73*	11%	*9.04*	56%	> 2060	
GR	LBMC Int. (Ph I) - South of Edward Street to W. Verona Avenue	MH17-127	MH17-121	1,003	30	2006	PVCPW	21.47	*1.00*	5%	*1.73*	8%	*9.04*	42%	> 2060	
GR	LBMC Int. (Ph I) - W. Verona Avenue crossing	MH17-121	MH17-120	405	30	2006	DI	18.17	*1.00*	6%	*1.73*	10%	*9.04*	50%	> 2060	
GR	LBMC Int. (Ph I) - W. Verona Avenue to Cleary Building Systems	MH17-120	MH17-112	2,496	30	2006	PVCPW	23.01	*1.00*	4%	*1.73*	8%	*9.04*	39%	> 2060	
GR	LBMC Int. (Ph I) - Cleary Building Systems to south of Paoli Street & Bruce Street	MH17-112	MH17-105	2,848	36	2006	PVCPW	20.37	*1.00*	5%	*1.73*	8%	*9.04*	44%	> 2060	
GR	LBMC Int. (Ph I) - South of Paoli St. & Bruce St. to Bruce St. at Badger Mill Creek	MH17-105	MH17-103	591	36	2006	DI	17.23	*1.00*	6%	*1.73*	10%	*9.04*	52%	> 2060	
GR	LBMC Int. (Ph I) - Badger Mill Creek crossing along Bruce Street	MH17-103	MH17-102	162	36	2006	PVC	20.37	*1.00*	5%	*1.73*	8%	*9.04*	44%	> 2060	
GR	LBMC Int. (Ph I) - Along Bruce Street between Badger Mill Creek & PS 17	MH17-102	MH17-101	126	36	2006	DI	17.23	*1.00*	6%	*1.73*	10%	*9.04*	52%	> 2060	
GR	LBMC Int. (Ph I) - MH17-101 to PS 17	MH17-101	PS17	70	36	2006	DI	29.53	*1.00*	3%	*1.73*	6%	*9.04*	31%	> 2060	
FM	PS 17 to Nesbitt Rd. between E. Verona Ave. & Cross Country Road	PS17	MH17-14450	13,357	16	1995	DI	7.20	*2.69*	37%	*3.90*	54%	*11.25*	156%	2010-2020	X Y
FM	Nesbitt Road between E. Verona Avenue & Cross Country Road to NSVI junction	MH17-14450	MH12-110	3,071	20	1995	DI	11.30	*2.69*	24%	*3.90*	35%	*11.25*	100%	2030-2060	
Junction with Nine Springs Valley Interceptor (Table 4-5)																

Total Length of Gravity Sewers (mi) 2.52
Total Length of Force Mains (mi) 3.11

Total Length of Gravity Interceptors Reaching Capacity by 2020 (mi) 0.00
Total Length of Gravity Interceptors Reaching Capacity by 2030 (mi) 0.00
Total Length of Force Mains Reaching Capacity by 2020 (mi) 2.53
Total Length of Force Mains Reaching Capacity by 2030 (mi) 2.53

Capacity Needs: **X** = Capacity reached by 2020 **Y** = Capacity reached by 2030

location of these capacity needs as well as the location of other projects discussed in later chapters.

Discussion

Significant growth has occurred in the MMSD system, and substantial additional growth is projected. As shown in Table 4.6, 26% of MMSD's gravity interceptor footage is expected to reach or exceed benchmark capacity by 2030. In general, about 1% of MMSD's interceptor mileage per year (or approximately 1.3 miles per year) may need hydraulic relief during the next 20-year period if they are to meet their benchmark capacities. These projections consider hydraulic capacity needs only. As detailed in following chapters, additional mileage will also likely need replacement or repair due to old age, pipe corrosion, and structural condition.

Seven of MMSD's 17 pumping stations are expected to be short of their benchmark maximum pumping capacities by 2030. In terms of *firm* capacities (i.e. capacities assuming the largest pump is out of service), eight of MMSD's 17 stations are expected to be short of their benchmark values by 2030.

The above capacity assessments should not be considered as a definitive or final conclusion about each component of the collection system. As discussed earlier, it is important to remember the general nature of benchmark design guides, the common limitations of wastewater flow measurements, and the variability between drainage basins. It is likely that some individual segments of the MMSD collection system may be better than projected and that some may be worse. The analyses in this chapter, however, are intended to provide a basis for identifying the most apparent strengths and challenges for the MMSD collection system in 2010, and to discuss how best to meet the challenges over the next 20 years. As individual replacement and relief projects are planned and designed in more detail, basin-specific high flow data and backup events should be studied to determine an appropriate design capacity for any particular project.

Given the challenges referenced in the preceding paragraph and in an effort to identify and prioritize the most critical projects with regard to hydraulic capacity, further analysis was conducted on those facilities that are predicted to reach capacity prior to 2030. The analysis that is summarized in Table 4.7 compares anticipated peak flows in the facility, as developed in CARPC's *MMSD Collection System Evaluation,* to the hydraulic capacity of the facility. While this type of analysis is useful for providing a general overview of interceptor capacity, it has several limitations. Peak flowrates are calculated by Manning's equation, which was developed for conditions of uniform flow in which the hydraulic grade line is parallel to the pipe slope. Given the physical characteristics and complexity of the collection system, this assumption during peak flow events is not valid in some instances due to backwater effects. Further, energy losses in the system are not accounted for in this type of analysis. Energy losses at manholes associated with expansion and contraction of flow are usually minor for average flow conditions but can be significant at peak conditions. Thus, analysis of capacity for each individual segment of the system can be misleading due to the water surface profile.

Another limitation of the analysis used to produce Table 4.7 involves the input location of dry weather and wet weather flows. In order to keep the number of facility segments and subbasins manageable, peak flowrates in portions of the collection system are misrepresented in some instances. Inputting peak flowrates from subbasins too far upstream generally leads to the overestimation of flows in downstream parts of the system.

The District's hydraulic model was used as an additional resource to analyze those facilities identified as having inadequate capacity to overcome the limitations mentioned previously. The hydraulic model can more readily and easily assess the impact of surcharged conditions in any particular interceptor segment and relate this impact to conditions both upstream and downstream. Development of a hydraulic grade line, or water surface profile, for interceptor segments can provide useful information in addition to the capacity analysis used to generate Table 4.7. The ability to model both dry weather and wet weather flows over various time increments is an additional feature to aid in the analysis.

Tables 4.8 and 4.9. were prepared to summarize MMSD facilities reaching capacity in ten year increments, starting in 2000 and ending in 2020. For each of these facilities the hydraulic model was used to assess the capacity needs identified in Table 4.7. In most instances the conclusions reached were confirmed. For other facilities the use of the hydraulic model determined that the capacity limitations were minor due to factors such as the size or length of the facility, or it demonstrated that the capacity exceedance would not cause any adverse effects in the collection system. A summary of the hydraulic modeling results for each of the facilities can be found in Tables 4.8 and 4.9 along with a recommendation for future action. In addition to the summary tables, copies of the hydraulic model results and other supporting documentation are provided in Appendix 5.

Table 4.8

MMSD Facilities Reaching Capacity 2000-2010

Madison Metropolitan Sewerage District

Identifier	Facility Name	From	To	Pipe Diameter (in)	Length (ft)	Summary of Hydraulic Modeling Results and/or Additional Comments	Recommended Action
2010A	Pump Station 1 Force Main	RDXT-09244	PBXT-10254	20	998	Exceedance of maximum velocity permissible for short length of force main.	No improvements recommended
2010B	Pump Station 7	-	-	-	-		
2010C	Pump Station 9 Force Main	PS 9	TE09-20598	14	40	Exceedance of maximum velocity permissible for short length of force main.	No improvements recommended
2010D	Pump Station 11	-	-	-	-	Not modeled.	N/A
2010E	Pump Station 12	-	-	-	-	Not modeled.	N/A
2010F	West Interceptor/Gammon Extension	MH05-236	MH05-211	24	12	Minimal surcharging. See Appendix A5 for results.	No improvements recommended
2010G	West Interceptor Relief	MH02-545	MH02-536	24 & 27	4,321	Moderate to significant surcharging from MH02-545 to MH08-228. See Appendix A8 for analysis of West Side conveyance system and HGL profile.	Program relief project into Capital Budget
2010G	West Interceptor Relief	MH02-531A	MH02-519	36	4,363		
2010G	West Interceptor Relief	MH02-518	MH08-228	36	214		
2010H	West Interceptor/Spring Street Relief	MH02-300	MH02-101	24	3	Minimal surcharging for short length of sewer. Negligible backwater effects. See Appendix A8 for results.	No improvements recommended
2010I	West Interceptor	MH02-011	MH02-005A	24	2,160	Redistribution of flows along length of interceptor shows that capacity is not exceeded in section. See Appendix A8 for revised analysis.	No improvements recommended. Additional analysis was performed to assess impact of heavy iron deposits on 24" cast iron sewer. Surcharging of less than one foot was modeled assuming buildup of 1" deposits and Manning's n= 0.018. It is recommended that this sewer be televised in the near future to assess condition and capacity.
2010J	Rimrock Interceptor	MH03-311	PS 3	10 & 12	3,800	Recommended peaking factor not achieved for existing (2009) flows. Infiltration is significant in this basin. See capacity analysis in Appendix A5.	Conduct infiltration study. Construct replacement sewer with adequate capacity or line existing sewer if I/I source can be found.
2010K	Northeast Interceptor	MH10-121	PS 10	36, 42 & 48	9,200	Relief sewer under construction in 2010	N/A
2010L	Northeast Interceptor	MH07-932	MH07-215	42 & 48	5,605	Significant surcharging confirmed via hydraulic modeling and flow monitoring in wet weather. See Appendix A5 for results.	Coordinate interceptor relief project with PS 18 construction.
2010M	Southeast Interceptor	MH07-211	PS 7	60	5,342	Significant surcharging confirmed via hydraulic modeling. See Appendix A5 for results.	Relief sewer not needed with new PS 18.

Table 4.9

MMSD Facilities Reaching Capacity 2010-2020

Madison Metropolitan Sewerage District

Identifier	Facility Name	From	To	Pipe Diameter (in)	Length (ft)	Summary of Hydraulic Modeling Results and/or Additional Comments	Recommended Action
2020A	Pump Station 13	-	-	-	-	Not modeled.	
2020B	Pump Station 15	-	-	-	-	Not modeled.	
2020C	Pump Station 17	-	-	-	-	Not modeled.	
2020D	Pump Station 1 Force Main	PS 1	PBXT-01337	24	1,346	Not modeled.	Capacity shown in Table 4.7 is exceeded assuming nominal diameter of 24". Using the actual diameter of 25.06" for this segment, capacity is not exceeded for 2030 UF flows.
2020E	Pump Station 17 Force Main	PS 17	MH17-14450	16	13,357	Not modeled.	
2020F	Nine Springs Valley Interceptor	MH12-110	PS 12	48	3,522	Surcharging of approximately 4-5 feet at 2020 UF flows from PS 12 to MH 12-121. See Appendix A5 for results.	Continue to monitor flows in PS 12 basin. CARPC's 2010 UF average flow at PS 12 is 6.5 mgd. Existing average daily flow at PS 12 for January 2010 was only 5.5 mgd, however. PS 12 capacity improvements should help to mitigate surcharging.
2020G	Nine Springs Valley Interceptor	MH11-169	MH11-167	42	465	Surcharging of approximately 1-2 feet at 2020 UF flows from MH 11-161D to MH 11-171. See Appendix A5 for results.	Continue to monitor flows in PS 12 basin. CARPC's 2010 UF average flow at PS 12 is 6.5 mgd. Existing average daily flow at PS 12 for January 2010 was only 5.5 mgd, however.
2020H	Nine Springs Valley Interceptor	MH11-111A	PS 11	54	5,930	Surcharging of approximately 1-2 feet at 2020 UF flows from MH11-111A to PS 11. See Appendix A5 for results.	Continue to monitor flows at PS 11. CARPC's 2010 UF average flow at PS 11 is 10.1 mgd. Existing average daily flow at PS 11 for January 2010 was only 9.2 mgd, however.
2020I	West Interceptor Relief	MH02-536	MH02-532	21	1,441	See Appendix A8 for analysis of West Side conveyance system.	
2020J	West Interceptor	MH02-038	MH02-032	18 & 20	2,276	Surcharging of up to 2-3 feet observed from MH02-032 to MH02-042 at 2020 UF CARPC flows. See Appendix A8 for further analysis.	Rehabilitate aging pipe with cured-in-place pipe as part of City of Madison road reconstruction project. Divert portion of West Interceptor upstream of MH02-043 to West Interceptor Relief system.

Table 4.9

MMSD Facilities Reaching Capacity 2010-2020

Madison Metropolitan Sewerage District

Identifier	Facility Name	From	To	Pipe Diameter (in)	Length (ft)	Summary of Hydraulic Modeling Results and/or Additional Comments	Recommended Action
2020K	Northeast Interceptor/Waunakee Extension	MH14-356	MH14-345	24	4,659	Significant surcharging modeled in upper reach of section (MH14-352 to MH14-356). See Appendix A5 for results. MMSD's average daily flow at downstream monitoring manhole MH14-325 in 2009 was 1.74 mgd. CARPC's flow estimate at MH14-325 for 2010 UF conditions is 1.41 mgd. Thus, existing flows seem to be at or slightly above CARPC projections.	Further study is recommended to better determine the average daily flow in the section in question. Additional subbasins should be developed to aid in this effort. If projected flows are confirmed, consideration should be given to capacity relief prior to 2020.
2020L	Northeast Interceptor/Truax Extension	MH10-145	MH10-121	48	10,973	Significant surcharging (~4.5 feet) at discharge of PS 13 force main. Modeling done with capacity improvements from Lien Road to PS 10. See Appendix A5 for results.	Modeling performed using 2030 UF peak flow of 25.77 mgd. 2030 TAZ peak flow of 21.56 mgd is significantly less. See Appendix A3 ('Station 13 Flow Diversion to Station 1') for further analysis and recommendations.
2020M	Far East Interceptor/Door Creek Extension	MH07-734	MH07-728	21	2,917	Significant surcharging modeled in FEI/Gaston Road Extension and upper reaches of Door Creek Extension. See Appendix A5 for results.	Modeled surcharging is due to rapid development in lands north of I-94. As of 2010 no development has taken place on these lands and planning is ongoing. No action necessary at this time.
2020N	Far East Interceptor/Cottage Grove Extension	MH07-437	MH07-426	18	5,510	Not modeled. Additional capacity constructed in 2009 to serve Village of Cottage Grove.	N/A
2020O	Far East Interceptor	MH07-425	MH07-416	30	3,861	Surcharging less than two feet at upstream end of section for 2020 UF CARPC flows. See Appendix A5 for results.	Surcharging is relatively minor for indicated flows and no local connections are present in surcharged area.
2020P	Southeast Interceptor	MH07-215	MH07-211	60	2,468	Significant surcharging confirmed via hydraulic modeling. See Appendix A5 for results.	Relief sewer not needed with new PS 18.
2020Q	Southeast Interceptor/Blooming Grove Extension	MH07-249	MH07-242	18	2,794	Significant surcharging at upstream end for 2020 UF CARPC flows (up to 8 feet). See Appendix A5 for results.	CARPC's projections for 2020 UF flows include rapid development of lands in the Door Creek valley. To date there has been little to no development in this sewershed and nothing appears imminent in the near-term. Significant capacity is available for flows based on 2030 TAZ numbers.
2020R	West Interceptor/Gammon Extension	MH05-240	MH05-236	24	1,252	Surcharging of one foot or less for 2020UF CARPC flows. See Appendix A5 for results.	This analysis assumes that flows from City of Madison's South Point Road Lift Station continue to PS 16 until 2020. If flows are diverted, surcharging is not expected to be a problem.

APPENDIX 4-1

ALTERNATIVE 1 - PS 15 TO PS 8

Capital Costs for Nine Springs Valley Interceptor (2010-2060)

Assumptions:	
Base Interest Rate	*3.00%*
Base Year	*2010*
End of Analysis Period	*2060*
Construction Cost Escalation Rate	*3.20%*
Interceptor Service Life (yrs)	*75*
Lining Service Life (yrs)	*50*

Notes:

1. *Estimates for Relief Year are based on CARPC's Collection System Capacity Evaluation (2009) with regard only to capacity. Condition not considered in this analysis.*
2. *A Relief Year of 2060 infers that capacity is adequate until the Year 2060 or beyond.*
3. *Construction of sewer segments infers that a relief sewer will be built roughly parallel to the existing sewer at the same size. In these instances the old sewer will be lined upon completion of the replacement sewer*
4. *For sewer segments not replaced within the analysis period based on capacity considerations, the segment was rehabilitated (lined) upon reaching its useful life*

							Construction of NSVI Segments					Lining of NSVI Segments					
							Capital Costs		Salvage Value			Capital Costs		Salvage Value			
From	To	Length	Size	Year of Original Construction	Relief Year	Lining Year	2010 Present Worth	Cost in Year Constructed	Year 2060 Value	2010 Present Worth	Construction Present Worth	2010 Present Worth	Cost in Year Lined	Year 2060 Value	2010 Present Worth	Lining Present Worth	Total Present Worth
PS 16 to MH12-177																	
PS 16	MH16-03385	7,214	36	1980	2060	N/A	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0
MH16-03385	MH12-177	2,965	30	1980	2060	N/A	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0
NSVI - Mineral Point Extension																	
12-177	12-176	400	33	1968	2060	2043	$0	$0	$0	$0	$0	$60,000	$169,661	$111,976	$25,543	$34,457	$34,457
12-176	12-166	3,920	33	1968	2060	2043	$0	$0	$0	$0	$0	$588,000	$1,662,675	$1,097,365	$250,317	$337,683	$337,683
12-166	12-164	732	30	1968	2060	2043	$0	$0	$0	$0	$0	$91,500	$258,733	$170,763	$38,952	$52,548	$52,548
12-164	12-157	2,942	30	1968	2060	2043	$0	$0	$0	$0	$0	$367,750	$1,039,879	$686,320	$156,554	$211,196	$211,196
12-157	12-156	544	30	1968	2060	2043	$0	$0	$0	$0	$0	$68,000	$192,282	$126,906	$28,948	$39,052	$39,052
12-156	12-133	10,101	36	1968	2060	2043	$0	$0	$0	$0	$0	$1,767,675	$4,998,416	$3,298,955	$752,515	$1,015,160	$1,015,160
12-133	12-121	5,740	36	1968	2060	2043	$0	$0	$0	$0	$0	$1,004,500	$2,840,403	$1,874,666	$427,625	$576,875	$576,875
12-121	12-112	4,284	36	1968	2060	2043	$0	$0	$0	$0	$0	$749,700	$2,119,910	$1,399,141	$319,154	$430,546	$430,546
12-112	12-110	970	48	1968	2060	2043	$0	$0	$0	$0	$0	$218,250	$617,141	$407,313	$92,911	$125,339	$125,339
12-110	12-101	3,484	48	1968	2017	2017	$2,787,200	$3,474,770	$1,482,568	$338,184	$2,449,016	$783,900	$977,279	$136,819	$31,209	$752,691	$3,201,706
12-101	PS 12	38	48	1968	2017	2017	$30,400	$37,899	$16,170	$3,689	$26,711	$8,550	$10,659	$1,492	$340	$8,210	$34,921
PS 12 to MH11-171																	
PS 12	MH11-171	4,786	36	1968	2060	N/A	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0
NSVI (PS 12 to PS 11)																	
11-171	11-169	812	42	1968	2022	2022	$568,400	$829,489	$409,214	$93,345	$475,055	$162,400	$236,997	$56,879	$12,975	$149,425	$624,481
11-169	11-167	465	42	1965	2020	2020	$325,500	$446,013	$208,140	$47,478	$278,022	$93,000	$127,432	$25,486	$5,814	$87,186	$365,208
11-167	11-161E	1,436	42	1965	2020	2020	$1,005,200	$1,377,366	$642,771	$146,621	$858,579	$287,200	$393,533	$78,707	$17,954	$269,246	$1,127,826
11-161E	11-161A	1,146	30	2001	2060	2076	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0
11-161A	11-159	1,321	36	1965	2025	2025	$792,600	$1,271,304	$678,029	$154,663	$637,937	$231,175	$370,797	$111,239	$25,374	$205,801	$843,737
11-159	11-158	340	36	1965	2023	2023	$204,000	$307,232	$155,664	$35,508	$168,492	$59,500	$89,609	$23,298	$5,315	$54,185	$222,677
11-158	11-156	1,103	30	1965	2060	2040	$0	$0	$0	$0	$0	$137,875	$354,712	$212,827	$48,547	$89,328	$89,328
11-156	11-151A	2,220	42	1965	2028	2028	$1,554,000	$2,739,590	$1,570,698	$358,287	$1,195,713	$444,000	$782,740	$281,786	$64,277	$379,723	$1,575,435
11-151A	11-145	3,784	42	1965	2026	2026	$2,648,800	$4,384,543	$2,396,883	$546,746	$2,102,054	$756,800	$1,252,727	$400,872	$91,442	$665,358	$2,767,412
11-145	11-141	1,558	36	1965	2056	2056	$934,800	$3,980,932	$3,768,616	$859,648	$75,152	$272,650	$1,161,105	$1,068,217	$243,668	$28,982	$104,134
11-141	11-137	1,648	30	1965	2037	2037	$824,000	$1,928,764	$1,337,276	$305,042	$518,958	$206,000	$482,191	$260,383	$59,395	$146,605	$665,563
11-137	11-129	3,995	33	1965	2023	2023	$2,197,250	$3,309,143	$1,676,632	$382,452	$1,814,798	$599,250	$902,494	$234,648	$53,525	$545,725	$2,360,523
11-129	11-127	733	36	1965	2031	2031	$439,800	$852,175	$522,667	$119,224	$320,576	$128,275	$248,551	$104,391	$23,812	$104,463	$425,038
11-127	11-116A	4,855	54	1965	2022	2022	$4,612,250	$6,730,839	$3,320,547	$757,440	$3,854,810	$1,213,750	$1,771,273	$425,106	$96,970	$1,116,780	$4,971,590
11-116A	11-111A	2,788	54	1965	2021	2021	$2,648,600	$3,745,355	$1,797,771	$410,084	$2,238,516	$697,000	$985,620	$216,836	$49,462	$647,538	$2,886,054
11-111A	11-106A	2,716	54	1965	2019	2019	$2,580,200	$3,425,868	$1,553,060	$354,264	$2,225,936	$679,000	$901,544	$162,278	$37,017	$641,983	$2,867,919
11-106A	11-104	1,689	54	1965	2018	2018	$1,604,550	$2,064,386	$908,330	$207,196	$1,397,354	$422,250	$543,259	$86,922	$19,827	$402,423	$1,799,776
11-104	PS11	1,525	54	1965	2016	2016	$1,448,750	$1,750,135	$723,389	$165,010	$1,283,740	$381,250	$460,562	$55,267	$12,607	$368,643	$1,652,383
PS 11 to NSWWTP																	
PS11	NSWWTP	4,173	36	1965	2025	N/A	$1,669,200	$2,677,342	$1,427,916	$325,718	$1,343,482	$0	$0	$0	$0	$0	$1,343,482
TOTALS											**$23,264,900**					**$9,487,151**	**$32,752,051**

APPENDIX 4-1

ALTERNATIVE 1 - PS 15 TO PS 8

Capital Costs for West Interceptor Relief (2010-2060)

Assumptions:	
Base Interest Rate	*3.00%*
Base Year	*2010*
End of Analysis Period	*2060*
Construction Cost Escalation Rate	*3.20%*
Interceptor Service Life (yrs)	*75*
Lining Service Life (yrs)	*50*

Notes:

1. Estimates for Relief Year are based on CARPC's Collection System Capacity Evaluation (2009) with regard only to capacity. Condition not considered in this analysis.

2. A Relief Year of 2060 infers that capacity is adequate until the Year 2060 or beyond.

3. Construction of sewer segments infers that a relief sewer will be built roughly parallel to the existing sewer at the same size. In these instances the old sewer will be lined upon completion of the replacement sewer

4. For sewer segments not replaced within the analysis period based on capacity considerations, the segment was rehabilitated (lined) upon reaching its useful life

							Construction of WI Segments					Lining of WI Segments					
							Capital Costs		Salvage Value			Capital Costs		Salvage Value			
From	To	Length	Size	Year of Original Construction	Relief Year	Lining Year	2010 Present Worth	Cost in Year Constructed	Year 2060 Value	2010 Present Worth	Construction Present Worth	2010 Present Worth	Cost in Year Lined	Year 2060 Value	2010 Present Worth	Lining Present Worth	Total Present Worth
PS 15 to MH02-547																	
PS 15	TE15-01350	1,360	24	1972	2060	N/A	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0
TE15-01350	BD15-02421	1,071	24	1972	2060	N/A	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0
BD15-02421	RD15-07254	4,837	20	1972	2060	N/A	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0
RD15-07254	TE05-22376	18	24	1972	2060	N/A	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0
TE05-22376	MH02-547	1,742	24	1959	2060	N/A	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0
West Interceptor Relief																	
02-547	02-546	497	24	1959	2060	2034	$0	$0	$0	$0	$0	$74,550	$158,767	$76,208	$17,384	$57,166	$57,166
02-546	02-545	192	27	1959	2060	2034	$0	$0	$0	$0	$0	$28,800	$61,335	$29,441	$6,716	$22,084	$22,084
02-545	02-538	3,121	27	1959	2010	2010	$3,121,000	$3,121,000	$1,040,333	$237,307	$2,883,693	$390,125	$390,125	$0	$0	$390,125	$3,273,818
02-538	02-536	1,200	24	1959	2010	2010	$1,200,000	$1,200,000	$400,000	$91,243	$1,108,757	$150,000	$150,000	$0	$0	$150,000	$1,258,757
02-536	02-535	600	21	1959	2014	2014	$600,000	$680,566	$263,152	$60,027	$539,973	$75,000	$85,071	$6,806	$1,552	$73,448	$613,421
02-535	02-532	841	21	1959	2014	2014	$1,009,200	$1,144,711	$442,622	$100,965	$908,235	$147,175	$166,937	$13,355	$3,046	$144,129	$1,052,363
02-532	02-531A	65	36	1959	2055	2055	$78,000	$321,870	$300,412	$68,526	$9,474	$11,375	$46,939	$42,245	$9,636	$1,739	$11,212
02-531A	02-519	4,363	36	1959	2010	2010	$5,235,600	$5,235,600	$1,745,200	$398,092	$4,837,508	$763,525	$763,525	$0	$0	$763,525	$5,601,033
TOTALS											**$10,287,639**					**$1,602,215**	**$11,889,855**

APPENDIX 4-1

ALTERNATIVE 2 - PS 15 TO PS 16

Capital Costs for Nine Springs Valley Interceptor (2010-2060)

Assumptions:

Base Interest Rate	*3.00%*
Base Year	*2010*
End of Analysis Period	*2060*
Construction Cost Escalation Rate	*3.20%*
Interceptor Service Life (yrs)	*75*
Lining Service Life (yrs)	*50*

Notes:

1. *Estimates for Relief Year are based on CARPC's Collection System Capacity Evaluation (2009) with regard only to capacity. Condition not considered in this analysis.*
2. *A Relief Year of 2060 infers that capacity is adequate until the Year 2060 or beyond.*
3. *Construction of sewer segments infers that a relief sewer will be built roughly parallel to the existing sewer at the same size. In these instances the old sewer will be lined upon completion of the replacement sewer*
4. *For sewer segments not replaced within the analysis period based on capacity considerations, the segment was rehabilitated (lined) upon reaching its useful life*

							Construction of NSVI Segments					Lining of NSVI Segments					
							Capital Costs		Salvage Value			Capital Costs		Salvage Value			
From	To	Length	Size	Year of Original Construction	Relief Year	Lining Year	2010 Present Worth	Cost in Year Constructed	Year 2060 Value	2010 Present Worth	Construction Present Worth	2010 Present Worth	Cost in Year Lined	Year 2060 Value	2010 Present Worth	Lining Present Worth	Total Present Worth
PS 16 to MH12-177																	
PS 16	MH16-03385	7,214	36	1980	2060	N/A	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0
MH16-03385	MH12-177	2,965	30	1980	2060	N/A	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0
NSVI - Mineral Point Extension																	
12-177	12-176	400	33	1968	2060	2043	$0	$0	$0	$0	$0	$60,000	$169,661	$111,976	$25,543	$34,457	$34,457
12-176	12-166	3,920	33	1968	2049	2049	$2,156,000	$7,364,732	$6,284,571	$1,433,555	$722,445	$588,000	$2,008,563	$1,566,679	$357,371	$230,629	$953,074
12-166	12-164	732	30	1968	2059	2059	$366,000	$1,713,114	$1,690,272	$385,563	-$19,563	$91,500	$428,278	$419,713	$95,739	-$4,239	-$23,803
12-164	12-157	2,942	30	1968	2041	2041	$1,471,000	$3,905,560	$2,916,151	$665,195	$805,805	$367,750	$976,390	$605,362	$138,087	$229,663	$1,035,468
12-157	12-156	544	30	1968	2025	2025	$272,000	$436,279	$232,682	$53,076	$218,924	$68,000	$109,070	$32,721	$7,464	$60,536	$279,460
12-156	12-133	10,101	36	1968	2060	2043	$0	$0	$0	$0	$0	$1,767,675	$4,998,416	$3,298,955	$752,515	$1,015,160	$1,015,160
12-133	12-121	5,740	36	1968	2028	2028	$3,444,000	$6,071,524	$3,481,007	$794,042	$2,649,958	$1,004,500	$1,770,861	$637,510	$145,421	$859,079	$3,509,037
12-121	12-112	4,284	36	1968	2014	2014	$2,570,400	$2,915,543	$1,127,343	$257,155	$2,313,245	$749,700	$850,367	$68,029	$15,518	$734,182	$3,047,427
12-112	12-110	970	48	1968	2022	2022	$776,000	$1,132,448	$558,674	$127,438	$648,562	$218,250	$318,501	$76,440	$17,437	$200,813	$849,376
12-110	12-101	3,484	48	1968	2010	2010	$2,787,200	$2,787,200	$929,067	$211,927	$2,575,273	$783,900	$783,900	$0	$0	$783,900	$3,359,173
12-101	PS 12	38	48	1968	2010	2010	$30,400	$30,400	$10,133	$2,311	$28,089	$8,550	$8,550	$0	$0	$8,550	$36,639
PS 12 to MH11-171																	
PS 12	MH11-171	4,786	36	1968	2056	2056	$1,914,400	$8,152,649	$7,717,841	$1,760,494	$153,906	$0	$0	$0	$0	$0	$153,906
NSVI (PS 12 to PS 11)																	
11-171	11-169	812	42	1968	2012	2012	$568,400	$605,360	$217,929	$49,711	$518,689	$162,400	$172,960	$6,918	$1,578	$160,822	$679,511
11-169	11-167	465	42	1965	2011	2011	$325,500	$335,916	$116,451	$26,563	$298,937	$93,000	$95,976	$1,920	$438	$92,562	$391,499
11-167	11-161E	1,436	42	1965	2013	2013	$1,005,200	$1,104,820	$412,466	$94,086	$911,114	$287,200	$315,663	$18,940	$4,320	$282,880	$1,193,993
11-161E	11-161A	1,146	30	2001	2060	2076	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0
11-161A	11-159	1,321	36	1965	2018	2018	$792,600	$1,019,745	$448,688	$102,349	$690,251	$231,175	$297,426	$47,588	$10,855	$220,320	$910,571
11-159	11-158	340	36	1965	2015	2015	$204,000	$238,797	$95,519	$21,789	$182,211	$59,500	$69,649	$6,965	$1,589	$57,911	$240,123
11-158	11-156	1,103	30	1965	2040	2040	$551,500	$1,418,850	$1,040,490	$237,343	$314,157	$137,875	$354,712	$212,827	$48,547	$89,328	$403,484
11-156	11-151A	2,220	42	1965	2018	2018	$1,554,000	$1,999,349	$879,714	$200,669	$1,353,331	$444,000	$571,243	$91,399	$20,849	$423,151	$1,776,482
11-151A	11-145	3,784	42	1965	2018	2018	$2,648,800	$3,407,899	$1,499,476	$342,041	$2,306,759	$756,800	$973,685	$155,790	$35,537	$721,263	$3,028,022
11-145	11-141	1,558	36	1965	2028	2028	$934,800	$1,647,985	$944,845	$215,526	$719,274	$272,650	$480,662	$173,038	$39,471	$233,179	$952,453
11-141	11-137	1,648	30	1965	2024	2024	$824,000	$1,280,687	$665,957	$151,910	$672,090	$206,000	$320,172	$89,648	$20,449	$185,551	$857,641
11-137	11-129	3,995	33	1965	2015	2015	$2,197,250	$2,572,041	$1,028,817	$234,680	$1,962,570	$599,250	$701,466	$70,147	$16,001	$583,249	$2,545,819
11-129	11-127	733	36	1965	2022	2022	$439,800	$641,818	$316,630	$72,226	$367,574	$128,275	$187,197	$44,927	$10,248	$118,027	$485,601
11-127	11-116A	4,855	54	1965	2015	2015	$4,612,250	$5,398,975	$2,159,590	$492,618	$4,119,632	$1,213,750	$1,420,783	$142,078	$32,409	$1,181,341	$5,300,973
11-116A	11-111A	2,788	54	1965	2013	2013	$2,648,600	$2,911,089	$1,086,807	$247,908	$2,400,692	$697,000	$766,076	$45,965	$10,485	$686,515	$3,087,207
11-111A	11-106A	2,716	54	1965	2012	2012	$2,580,200	$2,747,975	$989,271	$225,660	$2,354,540	$679,000	$723,151	$28,926	$6,598	$672,402	$3,026,942
11-106A	11-104	1,689	54	1965	2011	2011	$1,604,550	$1,655,896	$574,044	$130,943	$1,473,607	$422,250	$435,762	$8,715	$1,988	$420,262	$1,893,869
11-104	PS11	1,525	54	1965	2010	2010	$1,448,750	$1,448,750	$482,917	$110,157	$1,338,593	$381,250	$381,250	$0	$0	$381,250	$1,719,843
PS 11 to NSWWTP																	
PS11	NSWWTP	4,173	36	1965	2025	N/A	$1,669,200	$2,677,342	$1,427,916	$325,718	$1,343,482	$0	$0	$0	$0	$0	$1,343,482
TOTALS											**$33,424,147**					**$10,662,743**	**$44,086,890**

APPENDIX 4-1

ALTERNATIVE 2 - PS 15 TO PS 16

Capital Costs for West Interceptor (2010-2060)

Assumptions:

Base Interest Rate	*3.00%*
Base Year	*2010*
End of Analysis Period	*2060*
Construction Cost Escalation Rate	*3.20%*
Interceptor Service Life (yrs)	*75*
Lining Service Life (yrs)	*50*

Notes:

1. Estimates for Relief Year are based on CARPC's Collection System Capacity Evaluation (2009) with regard only to capacity. Condition not considered in this analysis.

2. A Relief Year of 2060 infers that capacity is adequate until the Year 2060 or beyond.

3. Construction of sewer segments infers that a relief sewer will be built roughly parallel to the existing sewer at the same size. In these instances the old sewer will be lined upon completion of the replacement sewer

4. For sewer segments not replaced within the analysis period based on capacity considerations, the segment was rehabilitated (lined) upon reaching its useful life

							Construction of WI Segments					Lining of WI Segments					
							Capital Costs		Salvage Value			Capital Costs		Salvage Value			
From	To	Length	Size	Year of Original Construction	Relief Year	Lining Year	2010 Present Worth	Cost in Year Constructed	Year 2060 Value	2010 Present Worth	Construction Present Worth	2010 Present Worth	Cost in Year Lined	Year 2060 Value	2010 Present Worth	Lining Present Worth	Total Present Worth
PS 15 to MH02-547																	
PS 15	TE15-01350	1,360	24	1972	2060	N/A	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0
TE15-01350	BD15-02421	1,071	24	1972	2060	N/A	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0
BD15-02421	RD15-07254	4,837	20	1972	2060	N/A	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0
RD15-07254	TE05-22376	18	24	1972	2060	N/A	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0
TE05-22376	MH02-547	1,742	24	1959	2060	N/A	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0
West Interceptor Relief																	
02-547	02-546	497	24	1959	2060	2034	$0	$0	$0	$0	$0	$74,550	$158,767	$76,208	$17,384	$57,166	$57,166
02-546	02-545	192	27	1959	2060	2034	$0	$0	$0	$0	$0	$28,800	$61,335	$29,441	$6,716	$22,084	$22,084
02-545	02-538	3,121	27	1959	2060	2034	$0	$0	$0	$0	$0	$390,125	$830,838	$398,802	$90,970	$299,155	$299,155
02-538	02-536	1,200	24	1959	2060	2034	$0	$0	$0	$0	$0	$150,000	$319,451	$153,336	$34,977	$115,023	$115,023
02-536	02-535	600	21	1959	2060	2034	$0	$0	$0	$0	$0	$75,000	$159,725	$76,668	$17,489	$57,511	$57,511
02-535	02-532	841	21	1959	2060	2034	$0	$0	$0	$0	$0	$147,175	$313,434	$150,449	$34,318	$112,857	$112,857
02-532	02-531A	65	36	1959	2060	2034	$0	$0	$0	$0	$0	$11,375	$24,225	$11,628	$2,652	$8,723	$8,723
02-531A	02-519	4,363	36	1959	2060	2034	$0	$0	$0	$0	$0	$763,525	$1,626,058	$780,508	$178,039	$585,486	$585,486
TOTALS											**$0**					**$1,258,005**	**$1,258,005**

APPENDIX 4-1

UNIT COSTS FOR INTERCEPOR REPLACEMENT AND REHABILITATION

Pipe Diameter (in)	NSVI Interceptor Replacement Cost per L.F.(1)	WI Interceptor Replacement Cost per L.F.(2)	Force Main Replacement Cost per L.F.(1)	Lining Cost per L.F.
18	$275	$550	$175	
21	$300	$600	$200	
24	$450	$900	$250	
27	$475	$950		
30	$500	$1,000	$325	$125
33	$550	$1,100		$150
36	$600	$1,200	$400	$175
42	$700	$1,400	$500	$200
48	$800	$1,600		$225
54	$950	$1,900		$250
60	$1,100	$2,200		

Notes:

(1). Unit costs taken from Technical Memo 3 of MMSD's *50-Year Master Plan Report* (December 2009).

(2). Unit costs for West Interceptor Replacement are assumed to be twice the unit costs for NSVI Interceptor Replacement due to factors such as traffic congestion, utility conflicts, etc.

Appendix 4-2
Life Cycle Pumping Costs for MMSD Pumping Station 15

Madison Metropolitan Sewerage District

PS15 Pumping Alternative	2009 Unit Pumping Rates for Pump Station Service Areas ($/Mgal Pumped)					Year 2009					2010 Annual Pumping Costs ($)	2010 Present Worth Pumping Costs ($)
	PS8	PS11	PS12	PS15[1]	PS16	Total Pumping Station Costs ($/Mgal)	Effluent Pumping Costs[5] ($/Mgal)	Total Pumping Costs ($/Mgal)	PS 15 Pumped Volume (Mgal)	Annual Pumping Costs ($)		
15 to 8	$26.50	-	-	$38.18	-	$64.69	$37.08	$101.77	496.1484	$50,000	$53,000	$5,656,000
15 to 16	-	$26.51	$23.86	$52.35	$129.32	$232.04	$37.08	$269.12	496.1484	$134,000	$142,000	$15,155,000

Notes/Assumptions:

(1). 2009 unit rate for pumping from PS15 to PS16 is estimated from actual power costs and flow volumes from September 1995 to August 1996. This corresponds to the last time period that PS15 pumpe to PS16 on a routine basis. Energy escalation rate of 4.9% per annum applied to unit pumping rate from PS15 to PS16 to convert to 2009 dollars, corresponding to the increase seen in the unit pumping rate between PS15 and PS8 from 1997 to 2009.

(2). Base interest rate = 3.00%

(3). Energy escalation rate = 6.00%

(4). Analysis Period (yrs) = 50

(5). Effluent pumping costs represent costs associated with discharge of final effluent to Badfish Creek and Lower Badger Mill Creek.

Chapter 5
Condition and Needs Assessment

Chapter Outline

This chapter is organized into the following sections:

- Introduction
- Pumping Station Priority Rankings
- Pumping Station Rating Criteria
- Pumping Station Summary Observations
- Forcemains
- Gravity Interceptors

Introduction

This chapter will examine the overall improvement needs for MMSD's existing pumping stations, forcemains, and gravity interceptors. The physical condition of each major facility will be evaluated in this chapter and will be considered together with the flow and capacity needs developed in previous chapters.

Pumping Station Priority Rankings

Table 5.1 presents a rating system developed to prioritize the need for improvements at MMSD's seventeen pumping stations. This system was introduced for the 2002 Collection System Facilities Plan and successfully achieved its intended purpose of ranking pumping stations by criticality, condition, and capacity needs. The rating system evaluates each pumping station for adequacy in six mission-critical categories:

- Maximum Capacity – Can the station meet its benchmark peak flow requirements? To what extent?
- Firm Capacity – Can the station meet its benchmark peak flow requirements without the largest pumping unit in service? To what extent?
- Power Supply Redundancy – Is the power supply system redundant and to what extent?
- Mechanical System Condition – What is the physical condition and reliability of the mechanical equipment, especially the largest pumping units?
- Building and Structural Condition – What is the condition of the wetwell structure, drywell structure, and control room?
- Electrical System Condition – What is the condition of the electrical equipment and control equipment? Of most critical importance is providing proper power and control to the pumping units.

Table 5.1
Pumping Station Rating Sheet
Assessment of Adequacy and Criticality

Facility	Adequacy/Condition of Mission Critical Category Likert Scale (1-5) - Category dependent (see text for explanatic						Total		Mean Weighting Factor (Sliding scale of 1 to 2)		Overall Rating	Ordinal Ranking (1 - 17)
	Peak Flow Capacity Qp (5 points)	Firm Flow Capacity Qf (5 points)	Power System Redundancy (5 points)	Mechanical Condition/ (5 points)	Structural Integrity (5 points)	Electrical Condition (5 points)						
PS NO. 1	1	1	1.5	1	1	1	6.5		1.75		11.38	**13**
PS NO. 2	1	1	1.5	1	1	1	6.5		1.95		12.68	**11**
PS NO. 3	2.5	1.5	3	1.5	4	1	13.5		1.00		13.50	**9**
PS NO. 4	3	2	3	1.5	2	3	14.5		1.15		16.68	**7**
PS NO. 5	1	1	1	1	1	1	6		1.20		7.20	**17**
PS NO. 6	1	1	1.5	1	1	1	6.5		1.30		8.45	**16**
PS NO. 7	3.5	3.5	2	2.5	1	2	14.5		2.00		29.00	**2**
PS NO. 8	1	1	1.5	1	1	1	6.5		1.85		12.03	**12**
PS NO. 9	2	2	1	1	2	1	9		1.10		9.90	**15**
PS NO. 10	1.5	1	1.5	1.5	1	1	7.5		1.70		12.75	**10**
PS NO. 11	3	3	3	3	2	4	18		1.70		30.60	**1**
PS NO. 12	2.5	4	4	2	2	3.5	18		1.50		27.00	**3**
PS NO. 13	3.5	3	4	1	3	3.5	18		1.30		23.40	**4**
PS NO. 14	2.5	2.5	4	1	3	3.5	16.5		1.15		18.98	**6**
PS NO. 15	1	2.5	4	2.5	4	3	17		1.25		21.25	**5**
PS NO. 16	1	1	2	2.5	2	1.5	10		1.10		11.00	**14**
PS NO. 17	3.5	3	1	4	1	1	13.5		1.15		15.53	**8**

Assumptions:

1). Recently completed projects include updated capacity and equipment condition assessments (e.g., PS 13 & PS 14 Firm Capacity Improvements and PS 6 & 8 Rehabilitation).
2). All flow in the Lower Badger Mill Creek valley is assumed to be flowing to Pumping Station 17 in Year 2030. For Year 2010 all flows in the LBMC valley south of Valley View Roa assumed to flow to PS 17. Station upgrades at PS 17 are not anticipated until the LBMC Interceptor is fully constructed (~2015-2020).
3). No satellite treatment facilities are considered (e.g., Sugar River Treatment Plant).

Prepared by Michael E. Simon 1/7/2010
(updated by TWG 4/2711)

As shown in Table 5.1, the six categories are each rated on a generalized Likert scale of 1 to 5 points (1–Excellent, 2–Good, 3–Adequate, 4–Poor, 5–Very Poor). The sum of the ratings is multiplied by a station weighting factor to arrive at an overall score. Thus the higher the overall score, the greater the need for improvements.

The weighting factor reflects an MMSD staff evaluation of the relative importance or criticality of each station within the MMSD system. A sliding scale from 1.0 to 2.0 is used for the weighting factor. Considerations in weighting the stations include the relative amount of flow through the station, how many other stations pump to the station, the availability of alternative flow diversion routes, and the amount of time the station can be down without basement backups or bypassing. The stations were weighted independently by several experienced members of the MMSD staff, and the mean values are used in Table 5.1.

Table 5.1 is the result of this rating process. The ordinal ranking column shows the relative priority for improvements at the pumping stations. These ratings were conducted in mid 2008 as part of the District's Master Planning effort and reviewed in 2010 to confirm the proper ratings prior to completing the update of the facilities plan. Several assumptions were made, including the following: 1) Recently completed projects include updated capacity and equipment condition assessments (e.g., PS 13 & 14 Firm Capacity Improvements, PS 6 & 8 Rehabilitation), 2) All flow in the Lower Badger Mill Creek valley is assumed to be flowing to Pumping Station 17 in Year 2030. For Year 2010 all flows in the LBMC valley south of Valley View Road are assumed to flow to PS 17; and 3) No satellite treatment plant facilities were considered. Based on this approach, the three stations with the greatest overall need for improvements are Pumping Station 11, Pumping Station 7, and Pumping Station 12, followed by Pumping Station 13, Pumping Station 15, and Pumping Station 14. The firm and maximum pumping capacity at Pumping Station 17 will have to be increased when the Lower Badger Mill Creek Interceptor is completed from Northern Lights Road to Midtown Road. This section of the LBMC Interceptor is scheduled for completion between 2015 and 2020. The results and implications of the Table 5.1 pumping station ratings are discussed in the Summary Observations section later in this chapter.

The flows, physical condition and operating experiences at the individual MMSD stations will continue to evolve with time and as future improvement projects are undertaken. It is therefore recommended that the station rating exercise continue to be updated regularly, maintaining a current assessment of the MMSD pumping stations.

Pumping Station Rating Criteria

This section explains how each station was rated within each of the mission-critical categories. Although no rating system is without some subjectivity, the ratings are intended to reflect each category and pumping station as objectively as possible

Maximum and Firm Station Capacities

The maximum and firm capacity scores shown in Table 5.1 are based on the adequacy ratio analysis presented in Chapter 4. The adequacy ratio is the ratio of a station's actual installed capacity divided by its desired benchmark capacity. The Year 2010 and Year 2030 adequacy ratios from Table 4.3 were averaged for each station, thus taking into consideration both the present and future needs. It should be noted that the Year 2010 adequacy ratios are based on actual rather than projected flowrates. Actual flowrates are measured via flowmetering equipment or computed based on pump run times and ratings for each pumping station. Scores were then assigned to each station using the following scoring scheme.

Score Assigned	Adequacy Ratio for Maximum Capacity	Adequacy Ratio for Firm Capacity
1. Excellent	> 1.25	> 1.15
2. Good	1.10 – 1.25	1.00 – 1.15
3. Adequate	0.90 – 1.10	0.80 – 1.00
4. Poor	0.75 – 0.90	0.65 – 0.80
5. Very Poor	< 0.75	< 0.65

Power System Redundancy

A number of considerations went into rating MMSD's pumping stations for power system redundancy and electrical condition. The intent of this summary is to give a general overview of why the stations were rated as they were. The rating system is qualitative, but the results should give a reasonable picture of where the greatest improvement needs exist.

Power supply redundancy at the pumping stations is an important practical criterion and is also required by applicable codes. In many cases, basement flooding or sewer overflows can occur within a short period of time after a pumping facility loses power. Therefore, redundant power in the form of an alternate feed from the utility or backup generation must be provided.

Although these four stations have redundant power sources, the worst-case situations (4 points) for MMSD at this time include Pumping Stations Nos. 12, 13, 14, and 15. These four pumping stations are each rated at 4 points because their particular design includes several weak links in the power system. The power to each station is fed through a single transfer switch, bank of transformers, and low voltage feed to the station. As a result, longer than desirable outages can occur if any of these parts of the system fail. To mitigate the problems that can occur during such failures, provisions to connect portable generation could be made. Future changes should include improvements to the power system design.

Although Pumping Station No. 4 is similar to the four pumping stations above, it is rated at 3 points, rather than 4, since it also contains a generator transfer switch, is near the Plant, and can be powered relatively easily using a portable generator set. MMSD has a limited number of portable generators, however, and a major power outage may require more generators than are available. If necessary, the flow to Pumping Station 4 could also be trucked to the Plant in the event of a catastrophic failure at the station. It should be noted that a second feed was added to this site based upon the outcome of the 2002 Facilities Plan. The second feed feeds an MG&E transfer switch, which then feeds the transformers that serve the station. Should the service feeding the pumping station fail, the transfer switch transfers power to the other feed. Pumping Station 4 should be in relatively good condition with this arrangement until other revisions are conducted at the pumping station. At that time, a more robust system with two service feeds in a main-tie-main arrangement similar to Pumping Station 9 should be considered. Another possibility would be to consider an on-site generator as an alternative to a second feed. Further use of portable generation is still another option; however, the District may need to consider additional portable generators if this path is taken. Another disadvantage of this option is that more portable generators require more manpower to operate and this could potentially overstretch the District's human resources during a major power outage.

Pumping Station 11 is also rated at 3 points. Its electrical system includes two 4.16 kV feeds from the utility. In general, this type of system provides reasonably adequate (not the best) redundancy. They have a common bus, which provides power to the entire station. This common bus is the weak link for everything powered from it and downstream of it. In addition, Pumping Station 11's electrical services are not entirely redundant and this is also a matter of concern that should be addressed when improvements at the pumping station are considered in the near future.

Pumping Station No. 3 is a small station at the plant. It currently does not have a redundant power service. It is rated at 3 points, rather than 4 or 5 points, because provisions are available for backup power via a portable generator connection and the small flow could also be hauled by truck if necessary. Again, the problem with this situation is that MMSD manages numerous facilities which require either backup generation or hauling in the event of a major outage. A long-term plan to minimize the number of these special-need facilities may be beneficial. It is possible that the power for Pumping Station 3 could be fed from the plant in the future.

Pumping Station No. 7 is rated at 2 points. Although it has dual feeds from Madison Gas and Electric, it was discovered that the feeds were not as "redundant" as initially thought. The feeds were on the same pole line and even though they were routed from different directions, an automobile striking a pole caused an outage of over 4 hours. Since that time, changes have been made to the way Pumping Station 7 is fed. These changes routed a new feed in from the southeast up Metropolitan Lane improving the situation significantly. The station can be fed from three separate MG&E circuits with the normal two entering the station area from different directions. Unfortunately, at this time, a single pole on Bridge Road is still the common point for all three of these circuits. MG&E has placed pole barriers along side of this pole to help protect it from potential damage by traffic and in the future they will be upgrading their Femrite Substation to provide a totally redundant feed to the pumping station. MGE estimates that the Femrite Substation improvements could happen as early as 2011. At the site itself, the services are kept as separated as possible, but they are still enclosed in a single substation enclosure. Even with redundant electrical systems, there is still always an electrical connection between the two systems at least somewhat susceptible to storm events that could take out both feeds. This, in addition to construction of a new Pumping Station 18, will help to make the pumping station as well as the collection system more reliable.

Pumping Station 16 is also rated at 2 points. It also has dual feeds from MG&E, but, similar to Pumping Station 7, there are areas that could be improved. Its main-tie-main arrangement within the pumping station provides a relatively reliable approach to providing backup power to the pumping station. However, the greater question is the reliability of the power system ahead of the pumping station. As with many of the District's pumping stations, the power system redundancy ahead of the pumping station should be investigated further to determine the level of reliability.

A number of pumping stations were rated 1.5 points. Pumping Station Nos. 1, 2, 6, 8, and 10 were recently rehabilitated. The numbers given assume the construction is completed for purposes of this facility planning effort. All five of these pumping stations have redundant power sources from the utility (MG&E) and care was taken to ensure the systems are relatively redundant; however, in all five cases, a major outage on MG&E's system can result in an outage to the pumping station and these five stations may be difficult to power from one of the District's portable generators. It may be possible to power pumps at PS 1, PS 6, and possibly PS 8 with portable generation (using PS 17's generator); however, it is unlikely that this could be done during peak flows. Portable generators sized to operate PS 2 and PS 10 are large, not easily obtained within a short time frame, and would take significant effort to connect to the pumping station's power system. As with all of the District's pumping stations, each power system should be reviewed with MG&E to determine the full level of redundancy available to serve the pumping station. Although the District's staff prefers redundant power feeds over onsite generators, generators are generally considered a more reliable option and should be considered as a good potential option during any design effort. It should be noted that either option meets the requirements found in NR 110 related to emergency operation.

Pumping Stations Nos. 5, 9, and 17 are rated at 1 point for having excellent redundancy. The two services from the electric utility to Pumping Station Nos. 5 and 9 should provide excellent redundancy. In addition, both PS 5 and PS 9 have provisions for connection of a portable generator. PS 17 has an on-site backup generator set. This provides excellent redundancy provided the unit and transfer controls are well maintained and should, in theory, be more reliable than two utility feeds.

Electrical Condition

The condition of the electrical equipment at the pumping stations is another important criterion. In some cases, the line-up of multiple parallel pumping units and corresponding parallel electrical equipment within a station makes this issue somewhat less important than the power supply system redundancy issue. However, it remains a critical aspect of a reliable pumping facility. Some of the factors mentioned in the power system redundancy analysis will overlap with electrical condition factors.

Pumping Station 11 is rated at 4 points. The pumping station is over 40 years old and physically its electrical system is not in good condition and in need of replacement or a major overhaul. A few major electrical improvements were made to the pumping station in the 1980s; however, these improvements are now approaching 30 years old and are not physically in much better condition than the original equipment. The pumping station's electrical system exhibits a relatively significant amount of corrosion and some of the equipment has been problematic.

Pumping Stations No. 12, 13, and 14 were rated at 3.5 points. These stations are all approaching 40 years old and they all exhibit a significant degree of corrosion to their electrical equipment. The stations electrical systems should be inspected for replacement or refurbishment. Some changes and improvements have been made to these stations over the years, but it is time that a much closer look is taken at all of them.

Pumping Stations No. 4 and 15 were rated at 3 points. These two stations are not quite as old as the stations which were rated at 3.5 and 4 points. The two stations both exhibit electrical system problems similar to those rated at 4 points (e.g., corrosion, obsolete equipment and parts, aging wiring and fixtures, etc.). Some of the equipment in these two pumping stations has been replaced. For example at Pumping Station 4, the main breaker was replaced when the original circuit breaker failed. At Pumping Station 15, some of the equipment was replaced during the Pumping Stations 11, 12, and 15 Project in the early 1980s.

Pumping Station 7's electrical condition is rated at 2 points. Its electrical system was replaced as part of a major rehabilitation of the pumping station in the early 1990s. Although the pumping station has operated and continues to operate well, the control system includes some early programmable controllers that the District should consider replacing; the programmable controllers are now obsolete and the functionality of the controllers is somewhat limited. In addition, other components within the electrical system should also be reviewed to determine if suitable and simple replacements can be found in the event of failure. The system is approaching twenty years of age and many of the components are no longer manufactured in the same form as the original equipment.

Pumping Station No. 16 is rated at 1.5 points. It went on-line in 1982. The motor starting equipment, power distribution equipment, and most of the electrical equipment is in excellent working condition. District electrical staff replaced the control system in the 2008 to 2009 timeframe. This replaced the original obsolete electronic control equipment with programmable controllers and operator interface terminals (Allen-Bradley PanelViews). These changes have significantly improved the reliability and operation of the pumping station. Although the pumping station is in excellent condition, the 30-year old age of the electrical equipment is the primary reason the pumping station scored 1.5 points versus 1 point.

Pumping Stations No. 1, 2, 3, 5, 6, 8, 9, 10, and 17 are each rated at 1 point. They all have relatively new electrical systems and the electrical equipment and controls are in excellent working condition. There may be some electrical changes to these pumping stations during the planning period; however, any changes will most likely be driven by mechanical or other system changes. In addition, since changes in the electrical and control industry occur quite frequently, the District anticipates some related changes will occur at these pumping stations throughout the planning period.

Mechanical Condition

The ratings for mechanical condition in Table 5.1 were based primarily on a pump condition assessment conducted by MMSD in 2000 and updated in November, 2010. The assessment examines the condition of MMSD's 57 sewage pumping units and is detailed in Appendix A2 of the Facilities Plan. Since the largest units at each station are the most critical for overall station reliability, the Table 5.1 station mechanical ratings place special emphasis on the largest units within each station.

Key issues that were noted in preparing the pump condition assessment include the various methods that are available and used to evaluate pump performance and the determination of a pump's service life. A primary goal for each of the District's pumping units in the collection system should be to provide 20 or more years of reliable service without accumulating excessive maintenance costs. Several key thoughts from the assessment are summarized in this section.

Sewage pumps are robust units that can have very long service lives if they are well maintained. Age alone is not a good criterion for a pump's condition. MMSD has numerous pumps in service that are 60 or more years old and still providing adequate service, and five pumps with more than 100,000 operating hours. Many parts on a pump are replaceable as they wear, including bearings, shafts, impellers, wear rings, and mechanical seals. Replacement parts can be obtained relatively easily for any MMSD pump, in some cases from the original manufacturer and in other cases from companies that manufacture specialty parts. Significant wear on a pump's volute or casing could make the pump unreliable or perhaps so inefficient that it should be replaced. Motors are generally long lived, have few problems, and are repairable or replaceable when problems occur.

MMSD's Mechanical Maintenance Department rated the condition of MMSD's 57 raw sewage pumps into categories of Good, Fair or Poor. The vast majority of the pumps (47 of the 57 pumps) were rated Good. Six pumps were rated Fair (Pumps 12A, 15A, 15B, 16A, 16B, and 16C). Four pumps were rated Poor, including Pump 11B and all three pumps at PS17.

The following specific recommendations were made in the November, 2010 condition assessment memo in Appendix A2:

1) Plans should be made to address the ten pumps that received a rating of less than Good.

    a) Rehabilitation projects at PS11, PS12, PS15, and PS17 are currently included in the District's ten-year Capital Projects Budget. All projects are scheduled to begin construction in or about the year 2015. These rehabilitation projects will provide an opportunity to address deficiencies with seven of the ten problematic pumps identified in the pump condition assessment.

b) The remaining three pumps receiving a rating of less than Good are located at PS16. All of these pumps are scheduled to be rebuilt in 2011 and their performance will be monitored to determine if further improvements are needed.

2) MMSD should continue to implement predictive maintenance procedures and/or strategies in pumping stations as they are rehabilitated or as the need arises. These procedures and strategies include the following:

   a) Installation of sensors on pump bearing housings to monitor unusual vibrations.
   b) Installation of limit switches on check valves to ensure that pumps do not run dry.
   c) Installation of flowmeters downstream of individual pumping units to provide early indication of declining pump capacity.
   d) Installation of bearing temperature sensors on the pump and motor.
   e) Use of motor soft starters.

3) Continue to monitor and evaluate the effect of pump plugging at the four major pumping stations (PS2, PS7, PS8, and PS11).

   a) Investigate the coarse screening of all flow from the Northeast Interceptor system as part of the PS18 improvements to mitigate pump plugging at PS7.
   b) Continue to track labor and material costs associated with pump plugging to ensure that staff time is spent as efficiently as possible and that other mechanical maintenance activities are not being neglected.
   c) Develop a risk-based assessment model for the District's collection system to identify the most critical areas of the system and to use as an aid in prioritizing improvement projects. This risk-based model should include the effect of pump plugging on pump station reliability.
   d) Perform a detailed economic analysis for re-installation of bar screens at the four major pumping stations, including life cycle costs. The analysis should include several alternatives for screening and removal of debris.

4) As a long-range goal, develop a formal program for the periodic internal inspection of all pumps to check for wear of critical components.

5) In general, avoid the use of extended vertical drive shafts for pumps in future designs. Vertical shafts tend to be labor intensive and more prone to causing pump vibration.

Building and Structural Condition

This criterion was included to assess the overall adequacy of a station's building, structure, and appurtenances. In general, MMSD's pumping stations are considered to be structurally sound. However, the age of the facility, its physical characteristics, layout, and any other operational deficiencies were considered in determining the rating for this category. As shown in Table 5.1, eight stations (PS1, PS2, PS5, PS6, PS7, PS8, PS10, and PS17) received excellent ratings. PS5 and PS17, both placed in service in 1996, are MMSD's newest stations. PS7, constructed in 1949, was extensively rehabilitated in 1992 as were PS1 (1948), PS2 (1963), and PS10 (1963) in 2006. PS6 (1948) and PS8 (1962) were rehabilitated in 2010. Five stations (PS4, PS9, PS11, PS12, and PS16) received good ratings. Except for PS16 (1982), these medium-aged stations were all placed on-line during the period 1962-1969. Two stations (PS13 and PS14) received adequate ratings. Although PS 13 (1970) and PS14 (1971) are somewhat newer stations, they were rated only as adequate, rather than good, due to heating and ventilating problems. Two stations, PS3 (1959) and PS15 (1974) were rated as poor. PS3 is a small two-pump station with a cramped pump room accessible only by ladder. PS15 has no superstructure, and its electrical control room is located below ground.

Pumping Station Summary Observations

Generally, the stations that ranked poorest in Table 5.1 have significant needs in several of the mission-critical categories. These stations are likely to have the greatest need for an overall station rehabilitation project. Various systems within a station are influenced by one another, and multiple needs often lead to an overall station rehabilitation rather than just an individual system upgrade. For example, a need for larger pumping capacity may drive a need for new pumps that, in turn, will require larger valves and larger motors. The larger motors may call for new electrical equipment and possibly a larger control room to house it. Such major electrical and mechanical and building work may present a logical opportunity or need to also improve heating and ventilating systems, lighting and other appurtenances. The purpose of the Table 5.1 rating exercise is not to finalize the details of a given rehabilitation project, but to point out the apparent leading candidates with the greatest needs. In all cases, a detailed design study would be needed to determine the precise scope of each project.

From Table 5.1, the MMSD pumping stations ranking highest in their need for improvements are PS11, PS7, PS12, PS13, PS15 and PS14. The stations are discussed individually in turn.

PS11, located at 4670 E. Clayton Road in the Town of Dunn, was constructed and placed into service in 1966. A major rehabilitation was performed in 1983 which added three new pumps to the station. No major rehabilitations have been performed since this time. PS11 is in need of major upgrades across all six of the scoring criteria listed in Table 5.1. The adequacy ratios for firm and maximum capacity for existing conditions are 1.03 and 1.25, respectively. Development in upstream basins such as the Lower Badger Mill Creek valley have the potential to reduce the ratios for firm and maximum capacity to 0.65 and 0.80, respectively, by 2030. Pump 11A is one of the oldest pumps in the District's collection system and has over 150,000 hours of recorded run-time. Pump 11B has the highest recorded maintenance costs in the previous ten years. The greatest need at this station, however, is with regard to the condition of the electrical equipment. Much of the original equipment from the 1966 construction is still in place and needs replacement to ensure reliable operation for this critical station.

PS7, at 6300 Metropolitan Lane in the City of Monona, was placed in service in 1950. Major station rehabilitations occurred in 1963 and 1992. PS7 is currently the largest of the District's stations in terms of average daily flow and pumping capacity and as a result it is deemed the most critical station in the collection system. Approximately 40% of the average daily flow to the Nine Springs Treatment Plant passes through this facility. As indicated in Table 4.3 in Chapter 4, the adequacy ratio for firm capacity at this station is below 1.00 for existing flows. The adequacy ratio for maximum capacity for existing flows is only 1.05. There is a strong potential for new and accelerated development in this service area between the City of Madison and the Village of Cottage Grove and a significant increase in average daily flowrates could be seen over the next twenty years (up to 40% of 2010 flowrates). As discussed in more detail in Chapter 6, it is not practical or prudent to provide the required capacity at PS7 due to site limitations and for reasons of system reliability. A new Pumping Station 18, working in tandem with PS7, will act to alleviate the firm and maximum capacity concerns at PS7. Some additional electrical and control work is required at PS7 in the near-term and will be completed in conjunction with or shortly after placing PS18 in service. Some of the electrical equipment at PS7 has outlived its useful service life and it is expected that some additional control and telemetry work will be required at PS7 so that it can operate in tandem with PS18.

PS12, located at 2739 Fitchrona Road in the Town of Verona, was constructed and placed into service in 1969. It is located upstream of PS11 and thus it is susceptible to the same increase in flowrates from the Lower Badger Mill Creek Valley as PS11. The adequacy ratio for firm capacity is 0.98 for existing conditions and is projected to decrease to 0.57 by 2030 for high-growth scenarios. The adequacy ratios for maximum capacity for 2010 and 2030 are 1.39 and 0.81, respectively. Pumps 12A and 12B have service lives in excess of 40 years, while Pumps 12C and 12D are approaching 30 years of service. Pump 12A is nearing 150,000 hours of run-time and is rated in fair condition by the District's Mechanical Maintenance Department. Significant deficiencies are also present in the power and electrical systems. The power to the station is fed through a single low voltage feed and significant outages can occur if any components related to this feed were to fail. Similar to PS11, the electrical equipment in this station has exceeded its service life and requires a major upgrade.

PS13, located at 3634 Amelia Earhart Drive in the City of Madison, was constructed and placed into service in 1971. Firm capacity improvements involving all three pumps were completed in 2008, but no other major rehabilitation work has been done since 1971. As a result, the electrical equipment is in poor condition and in need of replacement. Improvements to the design of the power system need to be implemented as well, similar to PS12. PS13 has no substantive heating systems for the interior spaces and minimal ventilation. These systems will need to be upgraded to meet current code requirements as part of any major rehabilitation work. Even with the firm capacity improvement project that was completed in 2008, the 2010 and 2030 adequacy ratios for firm capacity are 1.06 and 0.78, respectively. The scope of the 2008 project was constrained by downstream capacity and thus further firm capacity improvements will be needed at PS13 upon upgrades to interceptor capacity. It is likely that the PS13 firm and maximum capacity improvements will be needed prior to 2020. However, it may be possible to divert a portion of flows in the PS13 service area to PS1. This diversion could postpone the need for capacity improvements at PS13 by up to ten years or more. More information on this diversion can be found in Appendix A3.

PS15, located at 2115 Allen Boulevard in the City of Middleton, was constructed and placed into service in 1974 and serves primarily lands in the City of Middleton and Town of Westport. The primary needs for this station include those relating to power system redundancy and structural integrity. Similar to PS12 and PS13, the power to this station is fed through a single transfer switch, transformer bank, and low voltage feed. Damage to any of these components can result in significant interruptions of power to the station. The lack of a superstructure at this station presents challenges for access to equipment and shortens the expected life of electrical and control equipment. Capacity at this station is not an immediate concern, although the adequacy ratio for firm capacity could decrease to 0.87 by 2030 under high-growth scenarios. District staff should continue to monitor and assess the flow requirements for the proposed Bishops Bay Development in the City of Middleton and Town of Westport. This development includes 650 acres of land and has the potential to add approximately 7,300 people to the PS15 service area over the next twenty years.

PS14, located at 5000 School Road in the City of Madison, was constructed and placed into service in 1972. PS14 is similar to PS13 in age, service area, and capacity and thus has many of the same rehabilitation needs as PS13. Electrical equipment, the power system, and the HVAC system are all antiquated and need to be upgraded. The firm capacity at this station was also upgraded in 2008 at the same time as PS13, but the existing adequacy ratio is still only 1.11. Unlike PS13, diverting flow from the PS13 service area will do nothing to alleviate the firm capacity requirements at this station. Improvements to firm and maximum capacity will likely be needed prior to 2020.

The remaining eleven pumping stations generally received ratings of adequate, good or excellent in most of the scoring categories. Major rehabilitation work has been completed at PS1, PS2, PS6, PS8 and PS10 since 2005 to address condition and capacity deficiencies. Each rehabilitated station currently has a rating of either excellent or good across all of the categories and no major upgrade projects are contemplated at these stations in the near term.

Of the remaining six stations, the priority ranking for improvements is as follows: PS4, PS17, PS3, PS16, PS9, and PS5. Work that should be considered prior to 2020 includes improvements to the power system and electrical equipment at PS4, modifications and/or expansion of the pump house at PS3, and capacity upgrades at PS9. Rehabilitation work at PS17 depends in large part on the pace of development in the Lower Badger Mill Creek valley and the completion of the Lower Badger Mill Creek Interceptor between

Northern Lights Trail in the City of Verona and Midtown Road in the City of Madison. It is likely that these improvements will be needed sometime between 2015 and 2020. PS5 and PS16 score strongly across all of the categories and no major work is currently planned for these two stations.

Forcemains

The characteristics and capacities of MMSD's seventeen wastewater forcemains were examined in Chapter 4 and are summarized in Table 4.5.

Forcemains have the potential for very long service lives, sometimes approaching or exceeding 100 years. Wastewater forcemains are generally in service and under live pressure 24 hours per day, 365 days per year. They cannot easily be taken out of service, and are generally not accessible for internal inspection or televising. Within the past five years MMSD staff has had the opportunity to inspect the exterior and interior surfaces of very small segments of the PS6, PS7, and PS8 forcemains as part of short station outages or pipe abandonment projects. In general the concrete surfaces that were inspected looked very good and showed no evidence of corrosion or deterioration. From these very limited observations it appears that the concrete in fully submerged forcemains is in good to excellent condition, even after fifty years in service.

Measurements of flow and operating pressures can provide an indicator for some types of forcemain problems, such as major solids deposition or major air binding. In general, though, the most common and direct tool for assessing the condition of a given forcemain is its particular history of leaks and breaks and emergency repairs.

As might be expected, MMSD's oldest forcemains have exhibited the most problems. The old 30" cast iron PS2 Forcemain (1926) suffered a number of leaks and failures in its later years and was replaced by MMSD with a new facility in 2001. The old 20" cast iron Crosstown Forcemain (1914) also suffered numerous joint leaks and failures and was replaced by a new facility in 2002. Old cast iron forcemains are more susceptible to leaks and breaks than other common forcemain pipe materials such as ductile iron and prestressed concrete cylinder pipe. Old cast iron pipe is more brittle than ductile iron pipe. Cast iron pipe was also typically assembled with lead joints. These lead joints took considerable skill to construct and had a higher probability of failure if not constructed properly.

With the completion of the PS2 Forcemain Replacement and the Crosstown Forcemain Replacement projects, MMSD significantly reduced the age of its forcemain piping network. As can be seen in Figure 5.1, almost one-third of the network was installed during the 1960's, while installation of the remainder of the network is fairly evenly distributed from 1940 through 2010. Approximately 88% of the District's forcemains have service lives of 50 years or less at this time.

Figure 5.1 - Forcemain Age

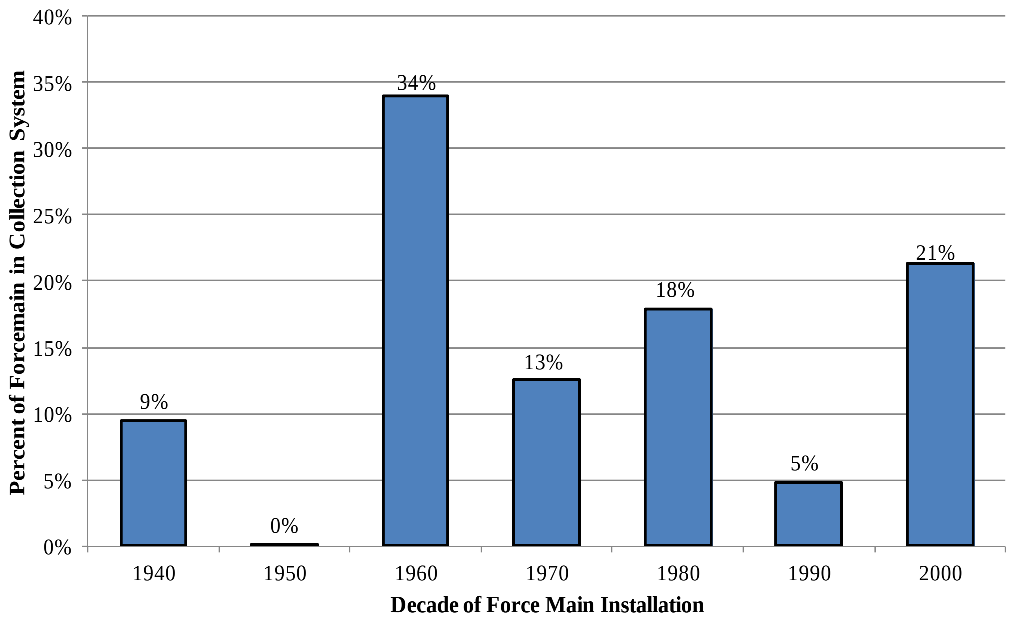
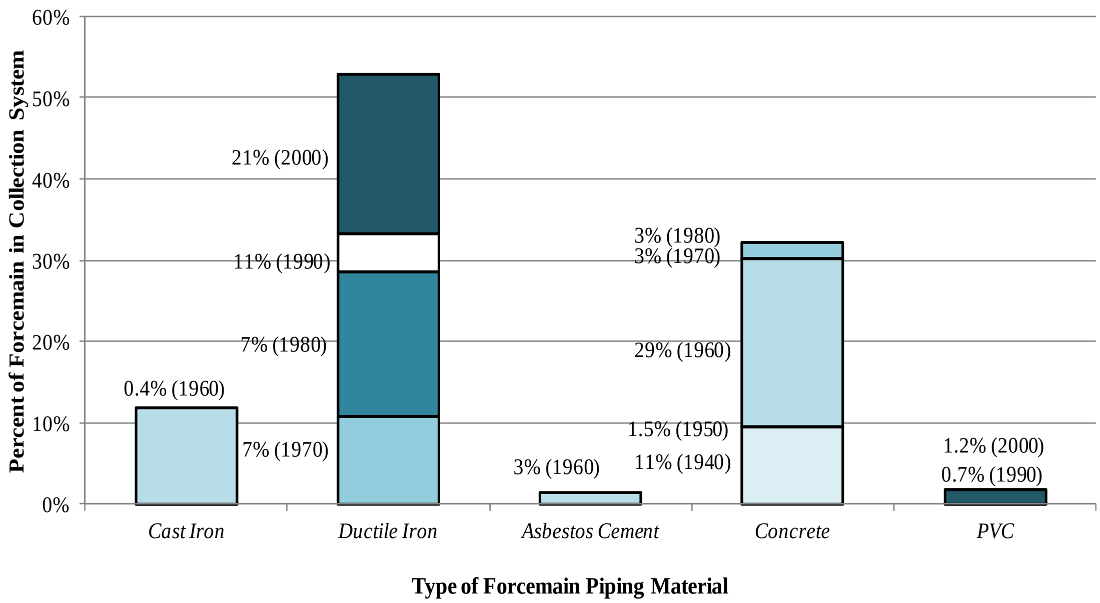


Figure 5.2 shows the relative age of the forcemain system in terms of piping material. The predominant materials in the system are ductile iron (46%) and concrete (48%). Concrete includes both reinforced concrete pipe and prestressed concrete cylinder pipe. As discussed previously, cast iron pipe is the pipe material most prone to failure and there is very little of it that remains in the District's system (0.4%).

Figure 5.2 - Classification of Forcemains by Material and Age



Given the age of the forcemains in service, the fact that ductile iron and concrete comprise the vast majority of piping materials, and the lack of recent forcemain leaks or breaks throughout the system, there are no present needs to replace forcemains in the system from a condition perspective.

Gravity Interceptors

As part of MMSD's interceptor maintenance program, approximately 10% of the 96-mile MMSD gravity system is televised each year. Table 5.2, located at the end of this chapter, tracks the history of MMSD's televised interceptor inspections and summarizes any major defects discovered. Condition scores for each interceptor segment are also shown and are used to develop ordinal rankings of sewer condition by pump station service area. These rankings are used as a guide in prioritizing future televising efforts and identifying possible rehabilitation projects. Interceptor segments in particular need of rehabilitation or replacement work are discussed in more detail in this subsection.

The following summary of needs is based on the physical condition of the interceptors as televised and the capacity status as developed in Chapter 4. Specific repair or replacement projects are recommended for certain interceptors. Locations of the interceptor projects are highlighted in Figure 9.1 (see enclosed map pocket). Interceptors that are functional but may be developing problems are recommended to be placed on a "watch list" and closely examined again at their next televising. Major interceptor repair and replacement projects already completed by MMSD are also summarized in this section.

PS1 Basin Interceptors

Significant improvements have been made to the PS1 basin since the 2002 Collection System Plan was developed. In 2002 the District's 54" x 24" Burke Outfall (1911) and 30" Burke Pressure Sewer (1912) on Pennsylvania Avenue were replaced with a new 36" PVC interceptor sewer. An 18" cast iron sewer on Commercial Avenue has also been replaced. These old facilities had experienced severe corrosion and were structurally unsound.

Some old facilities in the PS1 basin remain, however. The North End Interceptor on Sherman Avenue was constructed in 1927. This clay sewer was last televised in 1999 and was found to be in good condition, although it should be televised within the next 2-3 years to reassess its condition. In addition, the Northeast Interceptor Relief sewer was built in 1937. This is a cast iron sewer that does not convey very much flow due to the new sewers constructed in 2002 on Pennsylvania Avenue. As a result it has significant silt deposition and should be cleaned and televised within the next 2-3 years.

PS2 Basin Interceptors

There are several old cast iron interceptor sewers within the PS2 drainage basin that are displaying signs of hard iron deposits and tuberculation. The Southwest Interceptor on Haywood Street (1936) was last televised in 2000 and showed tuberculation at that time. The West Interceptor on Regent Street from Randall Avenue to PS2 has not been televised in the last ten years, although a piece of the sewer was removed during this period for a service connection and the pipe wall was found to be in excellent condition. Both of these sewers should be televised in the next 1-2 years to assess the deterioration due to tuberculation. Consideration should also be given to replacing the 24" sewer on Haywood Street with a new 36" sewer to serve as an inter-connection for PS2 and PS8 (see Chapter 6 for more details).

The Spring Street Relief (1940) is another cast iron sewer in the PS2 basin that was last televised in 2006. No significant defects were found during this inspection. The Southwest Interceptor on Shore Drive (2001) was televised in 2007 and was found to be in excellent condition.

PS3 Basin Interceptors

The Rimrock Interceptor was televised in 2009 and showed a variety of deficiencies including areas with root intrusion, sags, and infiltration. This sewer section should be evaluated further for rehabilitation. It should also be noted that the Rimrock Interceptor has capacity needs, as discussed in more detail in Chapter 4 and Appendix 5. It is recommended that an independent study of this interceptor be conducted to further evaluate its condition and capacity.

PS4 Basin Interceptors

The *2002 Collection System Facilities Plan* identified the South Interceptor - Baird Street Extension (1928) as a sewer that should be watched due to its age and structural stability. In 2009 the District rehabilitated this sewer with a cured-in-place liner. All other gravity sewers in this drainage basin have been televised in the last five years and are in good condition.

PS5 Basin Interceptors

The West Interceptor (1931) between PS15 and PS5 is an aging cast iron sewer that has significant iron deposits and tuberculation. It was placed on the watch list in the *2002 Collection System Facilities Plan*. Televising of this sewer in 2009 verified that the iron deposits continue to grow. The District intends to rehabilitate a portion of this sewer in 2011 via a cured-in-place liner, from MH05-021 to MH05-011.

PS6 Basin Interceptors

The East Interceptor/East Monona Interceptor has one of the worst scores in the District's rating database. This section of sewer is located on Fair Oaks Avenue north of Starkweather Creek. The sewer was constructed in 1925 and 1926 and includes sections of vitrified clay and cast iron. Televising of this sewer in 2006 showed several segments with deficiencies, including root intrusion and cracked pipe. The District intends to re-televise this sewer in 2010 and rehabilitate it with a cured-in-place liner in 2011.

The East Monona Interceptor downstream of Starkweather Creek was replaced in 1997 and is in good condition, as is the East Interceptor, which was sliplined in 1995 with PVC.

PS7 Basin Interceptors

The gravity interceptors in the PS7 drainage basin consist primarily of reinforced concrete pipe. Most of the interceptor segments in the basin have been televised in the last ten years to check for evidence of corrosion and other defects, although two notable sections have not been televised: (1). Southeast Interceptor (60") from PS 7 to the Northeast Interceptor, and (2). Northeast Interceptor (48") from the Southeast Interceptor to the Far East Interceptor. The District intends to televise both sections in either 2010 or 2011. The Northeast Interceptor segment is projected to reach its benchmark capacity by 2010 and is scheduled for replacement in 2013. The Southeast Interceptor is also projected to reach its benchmark capacity by 2010, although the construction of a new PS 18 will decrease flows through this interceptor such that benchmark capacity will not be exceeded.

In 2005 the District completed replacement of the Northeast Interceptor from the end of the PS10 force main to its junction with the Far East Interceptor (1.39 miles). The old concrete sewer had suffered from severe corrosion and was also in need of capacity relief. In 2010 the District will be rehabilitating the Far East Interceptor – Cottage Grove Extension (1.0 miles) with a new cured-in-place liner. This will address corrosion deficiencies noted in this section.

The East Interceptor and Far East Interceptor sections were televised in 2006 and found to be in reasonably good condition. Similarly, the Blooming Grove and McFarland Relief extensions to the Southeast Interceptor were televised in 2004 and no significant deficiencies were found.

PS8 Basin Interceptors

As noted in the *2002 Collection System Facilities Plan*, the pipe compromising the West Interceptor (1916 and 1932) in the PS8 basin is old and in mediocre condition on average. Numerous spot repairs have been made along its length. Approximately one mile was replaced in 2005 as part of the West Interceptor – Campus Relief (Phase IV) improvements.

No televising of the West Interceptor within the PS8 drainage basin has been done in the last ten years. It is recommended that televising of the entire length be performed in 2010 or 2011. A small portion of the West Interceptor on University Avenue between Midvale Boulevard and Shorewood Boulevard was televised in 2009 and found to be in good condition, however.

The West Interceptor Relief and West Interceptor – Randall Relief systems were televised in 2007 and found to be in generally good condition. Some areas of minor infiltration and mineral deposits at joints were found.

Extensive rehabilitation of the North and South legs of the Southwest Interceptor was done in 2007. Both legs were rehabilitated with a cured-in-place liner along their entire lengths. The Southwest Interceptor downstream of the confluence of the north and south legs was televised in 2007 with very few deficiencies found. A spot repair was made in 2009 to a short section of sewer near Thoreau Elementary School using a cured-in-place liner. The District intends to convey ownership of portions of the Southwest Interceptor sewer system to the City of Madison in 2010 or 2011.

PS9 Basin Interceptors

No significant gravity interceptor needs within the PS9 basin have been identified.

PS10 Basin Interceptors

Approximately 3,900 feet of the Northeast Interceptor will be replaced in 2010 between Nakoosa Trail and Lien Road. The existing 36”-48” concrete sewer is suffering from corrosion and requires capacity relief as well. The portion of the existing Northeast Interceptor from Nakoosa Trail to PS 10 will remain and serve as a relief for the new sewer to be installed. This section was televised in 2005 and found to be in good condition, with only minor corrosion noted.

The Truax Extension to the Northeast Interceptor was also televised in 2005 and found to be in good condition, although it is projected to reach its benchmark capacity within the next ten years and may require relief. The Lien Extension to the Northeast Interceptor was televised in 2007 and appears to be in good condition, with areas of moderate infiltration present in the concrete portions.

PS11 Basin Interceptors

The majority of the Nine Springs Valley Interceptor sewer system was televised in 2003. Much of this system is reinforced concrete pipe. In general, the ratings for this system were very good, with only small sections noted for minor root intrusion and infiltration.

Portions of this system may need capacity relief prior to 2030, depending on the fate of the Sugar River Wastewater Treatment Plant in the City of Verona.

PS12 Basin Interceptors
The Mineral Point (1968) and Midtown (1999) Extensions to the Nine Springs Valley Interceptor system were televised in 2004. No significant deficiencies were reported in either of these sections.

PS13 Basin Interceptors
Television inspection of the Northeast Interceptor in 2006 showed evidence of corrosion at the junction with the City of Madison's 36" Truax Interceptor (MH13-122A). The District is investigating the rehabilitation of the affected manhole with a lining system. There is also evidence of significant corrosion in the 48" interceptor sewer upstream of the Truax Interceptor junction. It is recommended that the section from MH13-116H to MH13-137 be monitored for further corrosion within the next five years and that the section from MH13-116H to MH13-125 be scheduled for rehabilitation prior to 2020.

Further downstream, the District relocated approximately 2,000 feet of the Northeast Interceptor at the northwest corner of the Dane County Regional Airport as part of improvements to the airport in 2006-07. In addition, the 48" Northeast Interceptor (1971) across the Dane County Regional Airport was rehabilitated in 2006-07 with installation of a cured-in-place liner. This concrete sewer had also experienced moderate corrosion in numerous segments.

PS14 Basin Interceptors
The DeForest Extension (1971) to the Northeast Interceptor was televised in 2004. Moderate infiltration was documented along the 9.4 miles that were televised, although no major deficiencies were found. The Waunakee Extension (1971) to the Northeast Interceptor was televised in 2007. In addition to moderate infiltration, areas of corrosion were also noted in certain areas. This section of sewer should be put on a watch list for future televising. Approximately 4,600 feet of this sewer is expected to reach its benchmark capacity within the next ten years (MH14-345 to MH14-356).

PS15 Basin Interceptors
The District replaced approximately 3,800 feet of the West Interceptor Extension in 2007 from Mendota Avenue to the north. The old concrete sewer had experienced problems related to joints, dips, and grease due to poor soil conditions. Further upstream, the District's West Point Extension to the West Interceptor has one of the worst condition scores. This rating is primarily due to the presence of corrosion in the asbestos cement pipe that was documented in 1999. This section of sewer should be re-televised within the next 1-2 years to reassess the corrosion.

The West Interceptor (1931) within the PS 15 drainage basin is another interceptor with a relatively poor rating. This cast iron sewer was last televised in 1999 and was noted in mediocre condition, with evidence of heavy mineral deposits and joint buildup. This section should be televised in the next 1-2 years.

PS16 Basin Interceptors

All gravity interceptors within the PS16 drainage basin have been televised since 2003 and are in excellent condition. This is due most likely to the age of the sewers in this basin. A 0.38-mile segment of the West Interceptor – Gammon Extension on Voss Parkway and Fortune Drive was replaced in 2002.

PS17 Basin Interceptors

The District's only interceptor in the PS17 drainage basin is the Lower Badger Mill Creek Interceptor, which was constructed in phases in 2006 and 2008. No deficiencies have been noted in this interceptor.

Table 5.2
Televising History for Gravity Interceptors

Segment Description	From	To	Segment Length (ft)	Pipe Characteristics: Pipe Dia. (in)	Pipe Characteristics: Year Installed	Pipe Characteristics: Pipe Material	Year Last Televised	Condition Score: Average Score per Segment	Condition Score: Worst Score per Segment	Ordinal Ranking	Significant Defects?	Comments and/or Defects
Pump Station No. 1 Service Area												
East Interceptor - North End Interceptor (Sherman Avenue)	MH01-126	MH01-120	1,482	10, 12	1927	VP	1999	NR	NR			
East Interceptor - North Basin Interceptor	MH01-120	MH01-304	6,670	18-20 & 36-42	2002	PVC	2007	25.00	25			Sewer replaced in 2002.
East Interceptor - Northeast Interceptor Relief	MH01-003	MH01-001	189	30	1937	CI	1999	NR	NR		W	
East Interceptor - East Johnson Street Relief Sewer	MH01-304	PS1	696	36	1979	RCP	NR	NR	NR			
East Interceptor - Burr Jones Park Leg	City sewer	PS1	10	42	1950	RCP	NR	NR	NR			
WEIGHTED COMPOSITE SCORE									25.0	15		
Pump Station No. 2 Service Area												
West Interceptor - Spring Street Relief	MH02-316A	MH02-101	4,580	24	1916 & 1940	CI	2006	25.27	31			
West Interceptor - Regent/Randall to PS2	MH02-014	MH02-101	5,164	24	1916	CI	NR	NR	NR		W	
Southwest Interceptor (Haywood St)	MH08-106	MH02-606	1,438	24	1936	CI	2000	NR	NR		W	Build-up of iron deposits and tuberculation.
Southwest Interceptor (West Shore Dr)	MH02-606	MH02-401	1,770	36	2001	PVC	2007	25.00	25			
Interceptor to PS 2 along West Washington Avenue	MH02-101	PS2	324	36-48	1963	RCP	NR	NR	NR			
WEIGHTED COMPOSITE SCORE									29.3	10		
Pump Station No. 3 Service Area												
Rimrock Interceptor	MH03-311	PS3	3,800	10 & 12	1958-59	RCP, CI	2009	30.40	37		X	Roots, sags, and infiltration noted throughout section.
WEIGHTED COMPOSITE SCORE									37.0	1		
Pump Station No. 4 Service Area												
South Interceptor - Baird Street Extension	MH04-408	MH04-311	1,584	10-15	1928 & 1955	VP(L), PVC, DI	2009	25.00	25			Vitrified clay section re-lined in 2009.
South Interceptor Relief	MH04-315	MH04-209	3,691	24	1995	PVCPW	2005	25.00	25			
South Interceptor - Lakeside Extension	MH04-209	PS4	2,271	24	1967	AC, VP & RCP	2005	26.00	33			
South Interceptor - Lakeside Extension (Coliseum Leg)	MH04-201B	MH04-201	653	15	1967	AC	2005	25.00	25			
WEIGHTED COMPOSITE SCORE									27.2	14		
Pump Station No. 5 Service Area												
West Interceptor Extension	MH05-102A	MH05-021	555	30	1957	RCP	1999	33.00	37			Grease.
West Interceptor (Marshall Park to PS5)	MH05-021	MH05-402	6,373	14-18	1931	CI	2009	26.70	33		X	Significant mineral deposition along entire length.
West Interceptor - Gammon Extension	MH05-230	MH05-011	8,833	10-18	1966	AC	2003	25.68	33			
Interceptor at PS 5	MH05-402	PS5	120	24	1995	PVC	2009	25.00	25			Line sag.
WEIGHTED COMPOSITE SCORE									33.1	4		

Scoring : 25 (best) to 100 (worst)
Ranking: 1 (worst) to 17 (best)

Defect Codes: X -= Segment in poor condition
W = Segment in mediocre condition

NR = No record for sewer inspection available from 1999-2010

Table 5.2
Televising History for Gravity Interceptors

				Pipe Characteristics				Condition Score				
Segment Description	From	To	Segment Length (ft)	Pipe Dia. (in)	Year Installed	Pipe Material	Year Last Televised	Average Score per Segment	Worst Score per Segment	Ordinal Ranking	Significant Defects?	Comments and/or Defects
Pump Station No. 6 Service Area												
East Interceptor (PS 1 Force Main to Olbrich Gardens)	MH06-122	MH06-103	6,339	36	1995	PVCPW	2006	25.30	29			
East Interceptor (Olbrich Gardens to PS 6)	MH06-103	PS6	1,483	42	1948	RCP	2006	27.67	31			Heavy grease in line to PS 6 wet well.
East Interceptor - East Monona Interceptor	MH06-209	MH06-204	1,411	14 & 15	1925-26 & 1997	CI, VP & PVC	2006	35.50	41		X	Cracked pipe and roots. Section scheduled for relining in 2007.
East Interceptor - East Monona Interceptor	MH06-204	MH06-108A	847	15	1997	PVC	2006	26.00	27			
WEIGHTED COMPOSITE SCORE									36.1	2		
Pump Station No. 7 Service Area												
Far East Interceptor - Gaston Road Extension	MH07-740	PB07-734	1,731	18 & 21	2008	PVC			NR			
Far East Interceptor - Door Creek Extension	PB07-734	MH07-426	17,253	21 & 24	1998	PVCPW	2005	25.09	27			
Far East Interceptor - Cottage Grove Extension	MH07-437	MH07-426	5,510	18	1981	RCP(L) & DI(L)	2006	32.36	34		X	Moderate corrosion throughout section. Lined in 2010.
Far East Interceptor - Far East Extension	MH07-426	MH07-416	4,014	30 & 36	1981	RCP & DI	2006	26.20	29			Minor infiltration noted throughout section.
Far East Interceptor	MH07-416	MH07-313	8,436	42	1970	RCP	2006	25.21	29			
Northeast Interceptor (FEI to SEI)	MH07-313	MH07-215	5,591	48	1964	RCP	2000	28.19	34			Some infiltration noted.
Southeast Interceptor (PS9 to SEI - Blooming Grove Ext)	MH07-823	MH07-218	8,473	8-15	1961 & 1992	AC & DI	2003	25.00	25			
Southeast Interceptor (SEI - Blooming Grove Ext to NEI)	MH07-218	MH07-215	1,606	36	1961	RCP	2001	33.00	33			Numerous cracks and mineral deposits.
Southeast Interceptor (NEI to PS7)	MH07-215	PS7	7,810	60	1961	RCP	2001	28.05	33			Many leaking joints.
Northeast Interceptor - Pflaum Road Replacement	MH07-955	MH07-932	7,323	36-54	2001 & 2005	DI, FRP	2005	25.00	25			Section replaced in 2005.
Southeast Interceptor - Blooming Grove Extension	MH07-249	MH07-218	13,413	18-36	1963 & 1967	RCP	2004	25.64	27			
Southeast Interceptor - McFarland Relief	MH07-517	MH07-228	6,667	20 & 30	1987	RCP & DI	2004	25.79	29			
Southeast Interceptor - Siggelkow Extension	MH07-618	MH07-512	5,078	8 & 12	1993 & 1996	PVC	2004	25.00	25			
East Interceptor	MH07-129	PS7	11,420	36 & 42	1948, 1985, 1986 & 1990	RCPWT, DI, RCP	2006	25.20	33			
WEIGHTED COMPOSITE SCORE									28.9	11		
Pump Station No. 8 Service Area												
West Interceptor Relief	MH02-547	MH02-014A	16,588	21-36	1959	RCP	2007	26.26	39			Hanging gaskets, mineral deposits, and roots from Shorewood Blvd. to west.
West Interceptor - Midvale Relief	MH02-708	MH02-531A	2,653	21	1971	RCP	2006	25.25	27			
West Interceptor - Campus Relief	MH08-228	MH08-201	5,682	36 & 48	1999, 2000, & 2005	DI & FRP	2007	25.00	25			
West Interceptor (State Crime Lab to Paunack Place)	MH02-542	MH02-513	12,023	12-21	1916, 1932 & 1961	VP	1999/2009	28.21	35			Some minor chips and cracks noted in 1999 televising. MH02-055 to MH02-049 televised in 2009 and found to be in good condition.
West Interceptor (Babcock Drive to Dayton Street)	MH02-021	MH02-014A	2,153	24	1916	CI	1999	33.00	33			Moderate iron buildup throughout section.
West Interceptor - Randall Relief	MH02-014A	PS8	10,020	30-48	1964	CI & RCP	2007	25.76	29			Some infiltration and mineral deposits noted.

Scoring : 25 (best) to 100 (worst)
Ranking: 1 (worst) to 17 (best)

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Table 5.2
Televising History for Gravity Interceptors

				Pipe Characteristics				Condition Score				
Segment Description	From	To	Segment Length (ft)	Pipe Dia. (in)	Year Installed	Pipe Material	Year Last Televised	Average Score per Segment	Worst Score per Segment	Ordinal Ranking	Significant Defects?	Comments and/or Defects
Southwest Interceptor - North Leg	MH02-189	MH02-174	5,539	15 & 18	1955	RCP(L) & AC(L)	2007	25.00	25			Ownership of SWI to be transferred to City of Madison in 2010.
Southwest Interceptor - South Leg	MH02-218	MH02-173A	5,456	12-16	1955, 1994 & 2000	PVC, RCP(L) & AC(L)	2007	25.00	25			Ownership of SWI to be transferred to City of Madison in 2010.
Southwest Interceptor	MH02-173A	MH08-106	17,229	15-30	1932, 1936, 1955, & 1994	AC, RCP, CI, VP & PVC	2007	27.27	27			Ownership of SWI to be transferred to City of Madison in 2010.
WEIGHTED COMPOSITE SCORE									30.9	9		
Pump Station No. 9 Service Area												
Southeast Interceptor	MH09-108	PS9	3,336	24 & 27	1961	RCP	2003	26.60	33			Moderate mineral deposits noted.
WEIGHTED COMPOSITE SCORE									33.0	5		
Pump Station No. 10 Service Area												
Northeast Interceptor - Truax Extension	MH10-145	MH10-426	10,948	48	1969	RCP	2005	27.50	35			
Northeast Interceptor Replacement	MH10-426	PS 10	9,222	48-63	2010	FRP	NR	NR	NR			Under construction in 2010.
Northeast Interceptor (Nakoosa Trail to PS 10)	MH10-112	PS 10	5,181	48	1964	RCP	2005	27.06	28			Insignificant to moderate corrosion.
Northeast Interceptor - Lien Extension	MH10-220	MH10-419	7,710	24 & 27	1970, 1973 & 1995	RCP & PVC	2009	26.00	29			Infiltration and minor mineral deposition noted.
Northeast Interceptor - Highway 30 Extension	MH10-305	MH10-104A	1,728	12 & 16	1966	AC & DI	2005	25.00	25			
WEIGHTED COMPOSITE SCORE									31.1	7		
Pump Station No. 11 Service Area												
NSVI - Mineral Point Extension	MH11-171	MH11-168	1,184	42	1968	RCP	2003	25.00	25			
Nine Springs Valley Interceptor (CTH PD to Certco)	MH11-168	MH11-161E	1,529	42	1965	RCP	2003	26.60	33			
NSVI - 2001 Relocation behind Certco	MH11-161E	MH11-161A	1,156	18 & 30	2001	PVC	2003	25.00	25			
Nine Springs Valley Interceptor (Certco to PS 11)	MH11-161A	PS11	30,275	30-54	1965	RCP	2003	27.31	33			Minor root and infiltration defects.
NSVI - Syene Extension	MH11-306	MH11-116A	1,822	12 & 16	1975	RCP	2003	26.00	29			
NSVI - Highway 14 Extension	MH11-423	MH11-106A	6,714	10-15	1977	PVC	2003	25.00	25			
NSVI - Highway 14 Extension (Granda Way Leg)	MH11-414C	MH11-414	834	10	1977	PVC	NR	NR	NR			
NSVI - Highway 14 Extension (Ski Lane Leg)	MH11-416A	MH11-416	236	8	1977	PVC	NR	NR	NR			
NSVI - Waubesa Extension	MH11-226	PS11	9,511	15-27	1971	RCP	2003	25.76	31			
WEIGHTED COMPOSITE SCORE									31.1	7		
Pump Station No. 12 Service Area												
NSVI - Mineral Point Extension	MH12-177	PS12	33,155	30-48	1968	RCP	2004	25.35	29			
NSVI - Midtown Extension	MH12-220	MH12-133	8,326	24 & 30	1999	PVC	2004	25.00	25			
WEIGHTED COMPOSITE SCORE									28.2	13		

Scoring : 25 (best) to 100 (worst)
Ranking: 1 (worst) to 17 (best)

Defect Codes: X -= Segment in poor condition
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Table 5.2
Televising History for Gravity Interceptors

Segment Description	From	To	Segment Length (ft)	Pipe Characteristics			Year Last Televised	Condition Score		Ordinal Ranking	Significant Defects?	Comments and/or Defects
				Pipe Dia. (in)	Year Installed	Pipe Material		Average Score per Segment	Worst Score per Segment			
Pump Station No. 13 Service Area												
NEI - Waunakee/DeForest Extension (PS 14 FM to Airport)	MH13-137	MH13-116H	6,609	48	1971	RCP	2006	28.71	34			Corrosion in pipe and manhole at City of Madison's Truax Interceptor junction.
NEI - Waunakee/DeForest Extension (Aiport to NEI/Truax Ext)	MH13-116H	MH13-116A	1,989	48	2006 & 2007	FRP	2007	25.00	25			New sewer relocated in 2007.
Northeast Interceptor - Truax Extension	MH13-116A	PS13	7,051	48	1969	RCP(L) & RCP	2008	25.00	25			RCP from MH13-116A to MH13-105 re-lined in 2008.
WEIGHTED COMPOSITE SCORE									28.8	12		
Pump Station No. 14 Service Area												
NEI - Waunakee/DeForest Extension (DeForest Leg)	MH14-209	MH14-102	49,465	21 - 36	1971	RCP	2004	25.64	37			Moderate infiltration noted.
Northeast Interceptor - Highway 19 Extension	MH14-417	MH14-134	6,334	12, 15 & 18	1971	RCP	2004	25.56	31			
NEI - Waunakee/DeForest Extension (Waunakee Union HS Leg)	MH14-362	MH14-358	775	10	1971	VP	2007	25.67	27			Moderate infiltration noted.
NEI - Waunakee/DeForest Extension (Waunakee Leg)	MH14-359	MH14-102	25,239	21-30	1971	RCP	2007	27.84	30		W	Moderate corrosion noted along entire length.
NEI - Waunakee/DeForeset Extension	MH14-102	PS14	1,907	42	1971	RCP	NR	NR	NR			
WEIGHTED COMPOSITE SCORE									34.3	3		
Pump Station No. 15 Service Area												
West Interceptor - West Point Extension	MH05-119	PB05-06607	1,955	14 & 18	1966	AC	1999	39.00	43			Moderate corrosion noted in AC pipe. Siphon under Pheasant Branch Creek not included in score.
West Interceptor Extension	PB05-06607	MH05-112A	1,832	14-30	1957	RCP	1999	29.50	30			Siphon under Pheasant Branch Creek not televised.
West Interceptor Extension Replacement	MH05-112A	MH15-101	3,842	8-10 & 30-42	2007	PVC	2007	25.00	25			
West Interceptor Extension	MH05-106	PS 15	1,645	30	1957 & 1999	PVC, RCP	1999	28.00	31			
West Interceptor	MH05-025A	MH05-103	880	12	1931	CI	1999	35.00	35			Mineral deposits and joint buildup.
WEIGHTED COMPOSITE SCORE									31.2	6		
Pump Station No. 16 Service Area												
West Interceptor - Esser Pond Extension	MH05-317	MH05-236	4,235	18-24	1978 & 1986	RCP	2003	25.00	25			
West Interceptor - Gammon Extension (Middleton Street)	MH05-240	MH16-211	1,264	24	1966	RCP	2003	25.00	25			
West Interceptor - Fortune Drive Replacement	MH16-211	MH16-202	2,016	36	2002	PVC	2007	25.00	25			
West Interceptor - Gammon Extension (Fortune Dr to PS 16)	MH16-202	PS16	228	36	1981	DI	2003	25.00	25			
Interceptor to PS 16 (via PS 15 force main)	MH16-105	PS 16	863	30 & 36	1981-82	PCCP & DI	2003	25.00	25			
WEIGHTED COMPOSITE SCORE									25.0	15		
Pump Station No. 17 Service Area												
Lower Badger Mill Creek Interceptor - Phase II	MH17-146	MH17-128	5,456	27 & 30	2008	PVCPW	NR	NR	NR			
Lower Badger Mill Creek Interceptor - Phase I	MH17-128	PS17	7,831	27-36	2006	PVCPW, PVC & DI	2007	25.00	25			New sewer constructed in 2006.
WEIGHTED COMPOSITE SCORE									25.0	15		

Scoring : 25 (best) to 100 (worst)
Ranking: 1 (worst) to 17 (best)

Defect Codes: X -= Segment in poor condition
W = Segment in mediocre condition

NR = No record for sewer inspection available from 1999-2010

Chapter 6
Special Projects and Diversions

Chapter Outline

This chapter is organized into the following sections:
- Introduction
- The Future of PS7 and a New PS18
- PS15 Diversion to PS8 or PS16
- Future MMSD Satellite Treatment Plants
- Inter-Station Diversions
- Lower Badger Mill Creek Interceptor
- East Verona Interceptor
- Headworks Equilization

Introduction

The operation of MMSD’s pumping stations, forcemains and interceptors can significantly impact one another. This chapter will examine a number of key projects and diversion concepts that may impact multiple stations or interceptors.

The Future of PS7 and a New PS18

PS7, located at Bridge Road in Monona, is MMSD’s largest and most critical station. The flows conveyed through three MMSD pumping stations (PS6, PS10 and PS9) and four interceptor systems (East, Southeast, Northeast, Far East) ultimately converge at PS7. With an average daily flowrate of 16.8 mgd in 2010, PS7 conveyed nearly 40% of MMSD’s total flow. The average wastewater volume at PS7 is projected to increase about 43% in the next 20 years according to CARPC’s 2030 UF estimates (from 16.8 mgd in 2010 to about 24 mgd in 2030).

The maximum PS7 pumping capacity is currently about 45 mgd. According to CARPC’s *Collection System Evaluation* (2009), a design capacity of 72 mgd will be needed for PS7 to handle peak flows from MMSD’s east side by 2060. These large future flows appear to be beyond the practical ability for PS7 to handle alone. A major capacity upgrade would require a new 42” or 48” forcemain to replace the old 36” line (1948) and would likely require a new set of 24” pumps. The PS7 pump room is already crowded with four horizontal 20” pumping units and associated 20” to 36” piping and valves. The larger equipment and the new forcemain connection would not be efficiently accommodated within the existing pump room and header geometry. Further, PS7 is already by far MMSD’s highest flow and most critical station (although not the largest station physically), and no diversion provisions exist. It is not prudent for MMSD to place such large future flow increases through this single station.

In view of the above, the concept of a future PS18 and PS18 forcemain, to work in tandem with PS7, was identified in MMSD's *Crosstown Forcemain Diversion Study* (November 2001). The driving force for the new PS18 and forcemain would be providing the needed capacity for MMSD's east side. A significant benefit, as a byproduct, would be the added protection and reliability provided by dual stations, each of which could serve as an emergency diversion for the other. A feasibility study for PS18 is included as Appendix A9 of this Facilities Plan.

PS15 Diversion to PS8 or PS16

MMSD's PS15 is located at Marshall Park on Allen Boulevard on the west side of the Madison metropolitan area. PS15 serves the far northwest side of the MMSD service area, including much of the City of Middleton.

PS15 is equipped to pump its flow either to PS16 (and ultimately down the Nine Springs Valley Interceptor system to PS12 and PS11) or to the West Interceptor system and PS8. When originally constructed in 1974, PS15 and its forcemain conveyed its flow to the West Interceptor. In 1983, a diversion forcemain was constructed to allow the PS15 flow to be diverted to the newly constructed PS16 and then on to the Nine Springs Valley Interceptor system. This diversion to PS16 remained the main operating scenario from 1983 until 1996. Starting in September 1996, the PS15 flow was directed back to the West Interceptor and PS8. This operating change was made in an attempt to reduce odor complaints occurring in the PS16 area, and also to reduce energy costs.

As discussed in Chapter 4, the direction of the PS15 discharge has significant implications for the downstream MMSD collection system. Average daily flows in 2010 were 1.34 mgd and peak flows were 5.12 mgd, as calculated by the Madison Design Curve. In 2030, the PS15 average daily and peak hourly flows are projected to be approximately 1.83 mgd and 6.65 mgd, respectively.

Upstream of Walnut Street, the need for capacity relief for the West Interceptor system depends significantly on PS15. If PS15 continues to be discharged to the West Interceptor system, approximately 10,100 feet of the West Interceptor Relief sewer from Whitney Way to Walnut Street will require relief by the year 2020. This is particularly significant since the PS15 service area has considerable potential for growth and the timing of this growth is uncertain. If flows from PS15 are redirected back to PS16 and the NSVI, the 2-mile gravity system from Whitney Way to Walnut Street would be better positioned with regards to anticipated flows and capacity relief would not be needed over the next fifty years. Without flows from PS15, the West Interceptor service area is projected to have little future growth. The physical condition of the interceptors, however, may still be of concern, particularly the segments of the original West Interceptor dating from 1916 and 1932.

The hydraulic adequacy of the Nine Springs Valley Interceptor system is also affected by the direction of PS15. If PS15 continues to be discharged to the West Interceptor system, approximately 32,000 feet of sewer in the NSVI gravity system will require capacity relief prior to the year 2030 (see Chapter 4). If PS15 is redirected back to PS16, the scope of capacity relief projects for the NSVI system will increase and all projects will be required much sooner, on the order of five to ten years (see Appendix 4-1 in Chapter 4 for details). Approximately 47,000 feet of sewer in the NSVI gravity sewer will require relief prior to 2030 under this scenario (i.e. with PS15 flows diverted to PS16 and the NSVI system).

Increases to maximum pumping capacity at PS12 and PS11 would also be needed in 2010 if flow were to be diverted from PS15 to PS16. A Sugar River Treatment Plant (discussed later in this Chapter) could significantly change the future needs at PS12 and PS11 and could reduce or eliminate the need for capacity relief in the NSVI system. However, the costs and regulatory constraints associated with a satellite treatment plant in the Sugar River basin are prohibitive at this time and do not support its construction.

The present worth analysis performed in Chapter 4 demonstrates that the preferred alternative for operation of PS15 is to continue the practice of routing flows from PS15 to PS8. While construction costs are similar between the two alternatives, energy costs associated with pumping from PS15 to PS16 are excessive relative to PS8. In addition, pumping to PS16 would exacerbate the odor problems that are currently observed at PS16. While it may be possible to mitigate the odor problems with more sophisticated equipment and intensive maintenance, the costs for doing so would not be practical or cost efficient. It is recommended that flows from PS15 continue to be directed to PS8 and that capacity relief projects for the West Interceptor system are scheduled accordingly.

Future MMSD Satellite Treatment Plants

The 2002 *Collection System Facilities Plan* discussed the concept of constructing satellite treatment plants in the collection system. The primary purpose of these plants would be to return treated effluent to the watersheds in which the water was originally withdrawn. A secondary benefit of these plants would be to reduce average daily and peak flowrates in downstream conveyance facilities, thus postponing the need for capacity relief projects.

In December of 2009, work was completed on MMSD's *50-Year Master Plan Report.* This report included a comprehensive analysis of all District operations and facilities at the treatment plant and within the collection system in order to identify capacity and condition related projects over the next fifty years. The report also investigated a number of master planning alternatives in the near term (2010 to 2030) and long term (2030 to 2060) involving satellite treatment plants.

Near-Term Alternatives

Wastewater flows in the Sugar River watershed are currently pumped to the Nine Springs Treatment Plant (NSWTP) via Pumping Stations 17, 12 and 11 and approximately 3.6 mgd of treated effluent is returned to this watershed. In order to continue this mode of operation significant improvements will need to be made in the collection system prior to 2020, including capacity relief for portions of the Nine Springs Valley intercepting system and firm capacity improvements at all three of the aforementioned pumping stations. Constructing a satellite treatment plant in the Sugar River watershed would postpone the need for all of these projects, while at the same time helping to promote the concept of watershed balance.

The Master Planning Report identified a number of alternatives for conveying and treating flows in the Sugar River watershed and advanced the following two alternatives for further analysis:

1. Alternative 1: Westside Conveyance System Expansion. This alternative included capacity improvement projects in the NSVI and at Pumping Stations 11, 12 and 17 to continue centralized treatment at NSWTP. Four options under this alternative were included to allow for increased flowrates of highly treated effluent back to the Lower Badger Mill Creek (LBMC)/Sugar River watershed.

2. Alternative 2: Sugar River WWTP. Under this alternative a new high quality effluent treatment plant would be built in the Sugar River watershed to treat wastewater generated in the PS17 and PS12 service areas, with the effluent discharged to the Sugar River.

A life cycle cost analysis for each alternative was performed and each alternative was scored based on a set of ranking criteria that included factors such as cost, regulatory constraints, and environmental impacts. From this analysis it was determined that the District's current mode of operation of centralized treatment and return of 3.6 mgd of treated effluent to the LBMC was the most cost effective option of serving the Sugar River basin and produced the highest total score. However, the report also noted that this option does not allow for future increases in inter-basin water transfers in order to achieve watershed balance.

Despite the higher life cycle costs and lower score associated with Alternative 2, the report recommended further evaluation of this alternative in an effort to obtain a more detailed cost estimate for construction of a new plant relative to the cost of NSVI facility improvements. Prior to performing this evaluation the District contacted the Wisconsin Department of Natural Resources (WDNR) and requested that the Department calculate effluent limits for both the LBMC and the Sugar River. This was done in an effort to verify the cost estimate of the new treatment plant, which assumes that a high quality effluent will be produced.

In their response, the WDNR classified a portion of the LBMC and the Sugar River as cold water fisheries that will have stringent effluent limits. The limits proposed by the Department for chloride, and possibly phosphorus, cannot be met with conventional processes at a new Sugar River WWTP. Thus, the cost estimates for a new plant, as detailed in the Master Plan, appear to be accurate and there is no need to study this option further at this time. The District will continue to convey wastewater flows from the Sugar River basin through the NSVI system to the NSWTP, while preserving the option for increasing the return flow of treated effluent to the LBMC and the Sugar River basin from 3.6 mgd up to a maximum of 8.0 mgd.

Long-Term Alternatives

Long-term alternatives are described as those which will provide relief in the conveyance system and will aid in mitigating inter-basin water transfers, but cannot be implemented prior to the year 2030. The *50-Year Master Plan* evaluated costs for the following two alternatives in providing long-term effluent reuse options:

1. Alternative 1: Centralized High Quality Effluent Treatment and Distribution. Under this alternative facilities at the NSWTP would be constructed to produce a high quality effluent that would be suitable for reuse such as augmenting stream flow, infiltration, industrial reuse, or irrigation.

2. Alternative 2: Decentralized High Quality Effluent Treatment Facilities. This alternative would include the construction of a satellite treatment plant near Pumping Station 13 that would receive flows tributary to PS13 or both PS13 and PS14 and provide reuse options similar to those listed in Alternative 1.

Life cycle costs were evaluated for both alternatives for effluent return flows of 4 mgd and 10 mgd. Alternative 1 scored higher than Alternative 2 for both flowrate scenarios due primarily to lower life cycle costs, greater public acceptance, and more flexibility in effluent reuse options. From this analysis it would appear that construction of a satellite treatment plant near PS 13 is not a cost effective or viable option at this point in time.

Conclusions

The District's *50-Year Master Plan* investigated the construction of satellite treatment plants in the southwest and northeast areas of the collection system. Life cycle costs and other ranking criteria such as regulatory constraints and public acceptance do not support the construction of these satellite plants at this time. In general, the most cost effective means of conveying, treating, and returning wastewater to its original basin is through centralized treatment at NSWTP.

Satellite treatment plants may become more viable as groundwater supplies become scarce, advanced treatment processes improve, and as the demand for a high quality effluent increases. While the District will continue to support and promote projects that mitigate inter-basin transfers of water and use treated effluent as a resource, where appropriate, the construction of satellite treatment plants will not be considered a viable alternative during the planning horizon for this facilities plan.

Inter-Station Diversions

In addition to the PS15 forcemain diversion, discussed earlier in this chapter, MMSD's collection system includes several inter-station connections that can allow a limited amount of flow diversion between specific stations. These diversions were not typically designed as such, but were generally by-products inherited from ongoing growth and expansion of the collection system into new station basins. Still, the availability of these diversions has been very beneficial for MMSD, and has been crucial in allowing MMSD the flexibility to take some major stations or forcemains out of service during emergency repairs or for major planned maintenance events.

Existing and potential MMSD inter-station diversion capabilities include the following:

Existing Diversions

- CTFM Diversion between PS1 and PS2
- PS15 forcemain diversion to PS16 or to PS8
- Gravity diversion of PS2 to PS8 via Southwest Interceptor
- Gravity diversion of PS8 to PS2 via Southwest Interceptor
- Gravity diversion of PS15 to PS5 via original West Interceptor
- Gravity diversion of PS16 to PS5 via West Interceptor Gammon Extension

Potential Diversion Projects

- Potential forcemain link between PS4 and PS8
- Potential for gravity diversion of PS13 to PS 1 via City of Madison Sanitorium Sewer and MMSD North Basin Interceptor.
- Potential for a gravity (or pressurized) link between PS6 and PS10
- Potential for a gravity link between PS7 and PS18

The inter-station diversions are detailed in Appendix A3, *Connector Lines Between Stations*, June 1999 (updated April 2010). In reviewing the list of inter-station diversions, it becomes apparent that all of the existing diversions are located in the western or central portions of the collection system. Other than the diversion capabilities of the Crosstown Forcemain, there is little to no redundancy or flexibility in the east side collection system. Three potential projects to improve this situation have been proposed in the memorandum in Appendix A3 and are briefly summarized in this chapter.

Diversion from PS13 to PS1

Prior to the construction of PS13 in 1970, flows in the PS13 service area were conveyed to PS1. With the extension of MMSD's Northeast interceptor to the Villages of DeForest and Waunakee in the early 1970's and capacity concerns in the PS1 service area, the City of Madison constructed the Truax Interceptor in 1971 to divert flows to PS13. With the rehabilitation of PS1 and PS2 in 2005, there is now ample capacity at these stations to convey a portion of the flow from the PS13 service area and much of the interceptor infrastructure in Packers Avenue and Pennsylvania Avenue exists to convey it. Approximately 2,700 feet of new sewer along the Packers Avenue frontage road and Commercial Avenue would need to be built to complete the diversion route to PS1. It is estimated that 1.2 mgd of average daily flow and 3.1 mgd of peak hourly flow could be diverted away from PS13 for 2030 TAZ flows.

Besides providing redundancy in the collection system, the PS13 diversion offers additional benefits by postponing the need for firm capacity improvements at the pumping station and for capacity relief in the Northeast Interceptor (Truax Extension) downstream of PS13. It is likely that a rehabilitation of PS13 will occur prior to 2030 to replace outdated equipment. Thus, the diversion of flow from PS13 will not, by itself, alleviate the need for significant work at PS13. This diversion should be considered, however, as an alternative to providing capacity relief of the Northeast Interceptor (Truax Extension) in the near term.

Diversion between PS6 and PS10

Pumping Station 10 handled the third largest average daily flow of the District's 17 pumping stations in 2010, yet there are few, if any, reasonable options for diverting this flow if PS10 or its forcemain becomes disabled. An overflow structure upstream of PS10 that discharged to Starkweather Creek was removed in 2009 as part of the replacement of the Northeast Interceptor. Due to the similarity of the wet well elevations of PS6 and PS10, it is possible to construct a gravity connector line between these stations. This gravity sewer line would be approximately 6,300 feet in length and could divert flows between the stations in the event of an emergency at either station.

It is estimated that approximately 5.6 mgd could be diverted in a 48" connector from PS6 to PS10 in an emergency, which is slightly less than the 2030 peak hourly flow for PS6 of 6.37 mgd. The estimated diversion capacity from PS10 to PS 6 is 25.9 mgd, which is less than the 2030 peak hourly flow for PS10 of 35.26 mgd, but well above the 2030 average daily flow of 13.3 mgd. Thus, while this connector line would not be able to fully convey peak flows from either station, it would have ample capacity to divert average daily flows from either station as well as a substantial portion of the peak flows.

Diversion between PS7 and PS18

Similar to PS10, PS7 is a high flow station with no available redundancy at this time. The average daily flow at PS7 in 2010 was 16.8 mgd, or approximately 39% of the total flow received at the Nine Springs Wastewater Treatment Plant. As such, it is deemed critical that the District reduce its reliance on this critical station to convey flows from the east side of the collection system. The District will begin preliminary planning in 2011 to construct a new PS18 approximately 6,300 feet to the southeast of PS7. Additional details regarding this diversion can be found in Appendix A9.

Lower Badger Mill Creek Interceptor

The Lower Badger Mill Creek (LBMC) watershed is located on the far westerly edge of the District's service area and extends roughly from PS17 in the City of Verona northerly to Old Sauk Road. Each of the four municipal entities comprising this watershed (Town of Middleton, Town of Verona, City of Madison, and City of Verona) have different development plans and thus needs for public sanitary sewerage service. In response to these needs, a sewer service report was prepared by the District in December 2004 that outlined various development scenarios and service options for this rapidly developing watershed. A copy of this report can be found in Appendix A6.

One of the recommendations presented in this report was that the District should work cooperatively with the City of Verona and City of Madison to design a new interceptor such that capacity exists to serve all lands within the watershed. To that end, the District entered into a Memorandum of Understanding with the City of Verona in 2006 for the first phase of the interceptor's construction from PS17 to Edwards Street (see Appendix A6 for copy of MOU). This interceptor segment was completed in 2006, at which time the District assumed ownership responsibilities.

The District also entered into an agreement with the City of Madison in 2008 for service to lands in the LBMC watershed that are located north of Midtown Road (see Appendix A6 for copy of agreement). This portion of the LBMC interceptor is to be owned and maintained by the City of Madison, with the provision that the District shall assume ownership responsibilities if lands in the Town of Middleton require future service. In 2010 the City constructed a pumping station at Midtown Road, approximately 1,000 feet to the west of the Hawks Landing development, to convey flows from this future interceptor to the District's NSVI-Midtown Extension sewer. A portion of this interceptor is scheduled for construction in 2011.

In 2008 the District extended the LBMC interceptor from Edwards Street in the City of Verona to Cross Country Road. The District is planning to construct the remaining portion of the LBMC interceptor from Cross County Road to Midtown Road as required by development needs in the basin. It is expected that this stretch of interceptor will be installed between 2015 and 2020. Upon completion of the LBMC Interceptor to Midtown Road, the City of Madison's Midtown Road Pumping Station will no longer be required.

East Verona Interceptor

The City of Verona has a need to reinforce a portion of its East Side Interceptor in the near term. This interceptor runs generally parallel to the Lower Badger Mill Creek

(LBMC) from PS17 to the Military Ridge Recreational Trail. The City intends to perform flow monitoring in this interceptor in 2010 or 2011. The schedule for capacity relief will depend on the results of this flow monitoring as well as the pace of new development in the sewer basin.

The District's PS17 forcemain travels through the same corridor as the City's East Side Interceptor. This forcemain is expected to reach capacity prior to 2020. Since a new Sugar River Treatment Plant will not be built in this watershed in the foreseeable future, capacity relief for the forcemain will need to be provided within the next ten years. This project should be coordinated with the City's interceptor project to the extent possible.

It is also possible that an extension of the District's LBMC Effluent Return pipeline could be located in this corridor in the future. Currently the District discharges approximately 3.6 mgd of treated effluent into the LBMC at the current outfall located south of USH 151 and east of CTH PB. It is likely that effluent return flowrates in excess of 3.6 mgd will need to bypass the LBMC and be returned further downstream to the Sugar River. A pipeline to convey this excess flow would likely follow the same general alignment as the City of Verona's East Side Interceptor and the District's PS17 relief forcemain.

Headworks Flow Equalization

The District's Headworks Facility at the Nine Springs Wastewater Treatment Plant currently receives raw wastewater flow from PS2, PS3, PS4, PS7, PS8, and PS11. If each of these stations were pumping at maximum capacity, the resulting peak flow would be approximately 150 mgd. Flows of this magnitude are at the upper limits of the plant's hydraulic capacity. The forcemain from PS18 will introduce another direct flow source to the Headworks Facility which may cause the plant's hydraulic capacity to be exceeded during large storm events if all of the stations are pumping at maximum capacity for extended periods of time.

For this reason the District should consider the construction of an equalization basin to temporarily store excess incoming flows during these large events. In order to properly analyze the need for such a system and to size it properly, it is recommended that the design of this project begin shortly after the design for the PS18 improvements are completed. An updated analysis of the plant's hydraulic capacity should be included as part of the Headworks Flow Equalization project.

Chapter 7
Collection System Maintenance

Chapter Outline

This chapter is organized into the following sections:

- Introduction
- General Discussion
- Pumping Station Maintenance
- Maintenance of Sewers and Force Mains
- Summary

Introduction

This chapter summarizes the practices used by MMSD to maintain its collection system of pumping stations, intercepting sewers, and force mains. The 17 regional pumping stations, 96 miles of intercepting sewers, and 29 miles of raw wastewater force-main sewers represent a significant investment by MMSD. The collection system is also an important part of the public-works infrastructure for the metropolitan area and is vital to protecting public health and the environment. To maintain such assets in good operating condition over a relatively long life requires a strong maintenance program and good maintenance practices.

General Discussion

MMSD has a long history of reliably maintaining its pumping stations and sewer systems. Although past maintenance practices kept MMSD's systems reasonably reliable, improvements in technology, better (modernized) maintenance methods, and better construction materials have allowed MMSD to improve on its maintenance practices over the years. MMSD's current maintenance practices are becoming more program-driven than in the past. Program driven maintenance (PDM) focuses labor resources on planned, preventive, and predictive activities to help reduce reactive maintenance to a small fraction of the maintenance performed. In addition, program driven maintenance relies on reliability centered maintenance practices to focus attention on those areas that are the highest priorities for sustaining a reliable system. A computer maintenance management system helps synchronize maintenance planning with inventory and tracks maintenance costs. Modern test equipment allows impending failures to be predicted with greater accuracy. Predictive testing permits repair or replacement of the failing parts to be proactively scheduled versus reacting when equipment fails. The proper balance of proactive and reactive work minimizes costs.

Pumping Station Maintenance

Overview

The purpose of MMSD's seventeen pumping stations is to receive incoming raw wastewater and pump it to another pumping station or to the Nine Springs Wastewater Treatment Plant. The stations operate continuously, 24 hours a day, seven days a week. The pumping units within the stations run as necessary to prevent sewer backups or overflows. To operate efficiently and effectively, the mechanical and electrical equipment must remain in good working condition, and the building structure must be kept sound and leak-proof. Additionally, the building and grounds should remain well maintained and aesthetically pleasing.

Mechanical Systems

The mechanical equipment in MMSD's pumping stations includes raw wastewater pumps, sump pumps, heating-ventilating and air-conditioning equipment, air compressors, valves, piping, gates, surge mitigating equipment, and solids handling equipment. This equipment is maintained by the Mechanical Maintenance Section. Each station is routinely visited at least once per week and inspected for proper operation. Additionally, each station is monitored via a radio telemetry system that provides information to computer screens on the process control system at the Nine Springs Wastewater Treatment Plant. Data displayed include pumping patterns, an indication of the pumps in service, the status of the electrical services, and in some cases, flow data. The telemetry system also signals the operator at the Nine Springs Wastewater Treatment Plant of any alarm conditions that occur. The operator will forward any alarm conditions to either the Mechanical or Electrical Maintenance Sections based upon the type of alarm received. If necessary a mechanic or electrician will be dispatched to the site.

During the routine site visit by the mechanic, the mechanic will look for any problems that need correction. If a problem cannot be corrected immediately, the mechanic will note the problem for follow-up work. A work order will be generated at the plant and planned and assigned for a later date. Other work orders are automatically generated for preventive and predictive maintenance of pumping station equipment. Lubrication of bearings and checking a pumping system for vibration or proper alignment are examples of preventive and predictive maintenance.

The most critical mechanical equipment at a pumping station is the raw wastewater pumping system. Therefore, it is very important that the pumps and ancillary equipment (valves, piping, surge arrestors, etc.) be well maintained to insure proper operation when needed. As part of the routine site visit, the mechanics visually inspect the pumps, listen for unusual sounds that may indicate wear or misalignment, feel the pumps to sense excess vibration or high temperature, check for plugged vent lines, and ensure that sump pumps are working properly. At recently rehabbed pumping stations, the raw wastewater pumping systems have been equipped with vibration sensors, and bearing and motor winding temperature sensors to continually monitor the pumps and the corresponding motors. During site visits, mechanics will also tighten packing on those pumps not using mechanical seals and will read any suction or discharge pressure gages as these could help identify problems with a pump.

Predictive and preventive pump maintenance that takes more time than is allowed during a routine site visit will be scheduled via periodic work orders. This maintenance includes checking pump/motor alignment, vibration testing of some of the larger pumps, exercising gates and valves, cleaning float tubes, testing backflow preventers, checking the HVAC systems, inspecting cranes, and preparing the stations for winter and summer operation. When major corrective action is necessary, the mechanics will remove a pump from service and transport it to MMSD's maintenance facilities. MMSD's maintenance facilities are equipped with full rebuild capabilities for pump repair.

Valves and gates play an integral role in keeping the pumping station operational. Pumping stations typical contain numerous types of valves and gates intended to divert and control the wastewater within the pumping station. Check valves allow water flow in only one direction, preventing an operating pump from pumping backwards though idle pumps and preventing the force main from draining back into the wetwell. Isolation valves on both the pump suction and discharge allow maintenance to be performed on a pump while other pumps remain in-service. If equipped, force main valves isolate the entire pumping station from the force-main, allowing work on any part of the piping system within the station.. Ball valves and sometimes gate valves are used for surge mitigation on start up and primarily on shut down of pumps. Gates are generally used to control the flow from the collection system into the pumping station wetwells or to isolate half of the wetwell. This is typically done for maintenance purposes, wetwell cleaning, and in some cases, for operational purposes.

The last paragraph discussed the importance of valves and gates within the pumping system and logically it follows that these are good reasons why valves and gates should be kept in good working condition. The best way to keep valves and gates maintained is to exercise them periodically. Oftentimes, valves and gates that are relied upon for isolation or operational procedures do not work when called upon, simply because they have not been operated for a significant amount of time. That being said, it is often difficult to operate some valves or gates without disrupting normal operation of the pumping station and/or because the valves or gates are difficult to close or open. Some valves or gates may require manual operation and take hundreds of turns to open. Therefore, the District has begun to include motorized operators on its valves and gates whenever possible, and where motorized operators are not installed, has attempted to come up with easier ways to operate them, e.g., using an electric drill with a socket to drive the operator rather than manually driving it. Eventually, it is hoped that all of the valves and gates will become part of a routine exercise program that periodically verifies proper operation.

Since the 2002 facilities plan, the District has made systematic changes in its' approach to solids handling at the pumping stations. With the Tenth Addition to the Plant, all screenings are now dealt with at the Nine Springs Wastewater Treatment Plant versus the pumping stations. The impacts of this change in operation are discussed in more detail in Chapter 3. In general, the change has shifted labor at the pumping stations from manual removal of screenings and maintenance of screening equipment to monitoring of pump performance and cleaning of pumps. Both of these maintenance activities are the result of a higher frequency of pump plugging. The only remaining piece of solids handling equipment within the District's collection system is a grinder at Pumping Station 17. This grinder remains in place because of concerns related to the large solids that Pumping Station 17 can potentially receive from the county mental hospital located within its service area. Typical mechanical problems with grinders include occasional jamming and periodic overhaul of the grinder mechanisms due to the maintenance intensive process of grinding non-organic (rocks, sand, etc.) solids.

Air compressors are installed in some of the pumping stations to provide air for level sensing instrumentation or for surge mitigation systems. For the level sensing systems, a small amount of air is bled into the wetwell via a pipe or plastic tube. The backpressure is measured and calibrated to correspond to the wastewater level in the wetwell. These air compressors use very little air, but because they are critical to sensing the proper level, it is very important to keep them well maintained. The surge mitigating systems use a great deal more air. These systems inject air into a storage vessel connected to the outgoing force main. The air stored in this vessel acts as a cushion or buffer for when the pumping units start or shut off. The air in the vessel compresses or expands, helping dissipate surge energy in the force main. It is also very important to keep the air compressors attached to these systems well maintained. At the present time, only Pumping Station 7 has a surge mitigating system of this type.

Other surge mitigating equipment includes surge arrestors that are a type of pressure release valve. Typically, these valves will open on high pressure (e.g., a pressure wave from a water hammer transient wave) releasing some wastewater, and consequently dissipating the high pressure, back into the wetwell. The amount of wastewater released in such an event is generally minimal. Since these surge mitigating devices protect the force main and the pumping station header, it is important that they remain in good working condition. In addition, another reason to keep them well maintained is that they could potentially stick in the open position and continue to release wastewater into the wetwell, causing excessive pump operation and possibly flooding the wetwell. Some of the force mains also include air release/vacuum intake valves, which provide another method of surge mitigation. Although not located within the pumping station, they can protect the pumping station's piping from excessive positive or negative pressures by releasing extreme pressures to the atmosphere, generally at the force main's high points. These are discussed in greater detail later in this chapter.

Ventilation and the air handling systems also provide an important function at MMSD pumping stations. Many of the older stations have little or no forced ventilation. This can lead to poor air quality within the stations, including foul and corrosive air in the dry well area. This, in turn, can lead to corrosion of sensitive electrical equipment, an unhealthy air quality, and rusting of the piping and equipment within the drywell.

To combat this, new regulations require air-handling systems that provide adequate amounts of fresh air to prevent the buildup of corrosive and/or toxic gases. All new or rehabilitated MMSD pumping stations are equipped with heating, ventilating and air conditioning (HVAC) equipment to meet these requirements. This provides a better environment for the pumping station equipment and a safer environment for personnel during site visits.

HVAC systems are maintained by the Mechanical and Electrical Maintenance Sections of the District, each taking care of their respective areas of the systems and equipment. Older controls are often manual while newer controls are typically integrated into the station's control system and may be monitored or operated from the system's station control center, e.g., a graphic display (operator interface terminal).

Electrical, Controls, and Instrumentation

The electrical equipment in MMSD's pumping stations includes power entrance, transfer, and distribution equipment, motors and motor controls, pump and auxiliary control systems, instrumentation (including telemetry equipment), and lighting systems. MMSD's electrical systems are maintained by the Electrical Maintenance Section with significant support from the Electrical Engineering Group. The District's electrical staff responds to problems in a manner similar to the mechanical staff. When an alarm signals the operator of a problem at one of the pumping stations, it is determined who will respond and either an electrician or mechanic will be dispatched to the site. However, the vast majority of electrical work at the pumping stations is either planned maintenance, preemptive replacement of equipment, or new equipment installation.

The electrical staff does extensive preventive and predictive maintenance of the electrical equipment at the pumping stations. This work includes cleaning of electrical cabinets, inspection of electrical contacts, tightening of electrical terminations, thermal sensing of electrical equipment while in operation, cycling of equipment to determine proper operation (for example – power system auto transfer schemes), verification of proper signaling for alarms and other instrumentation, and verification of proper control operation for all control systems. In addition, roughly every three years an electrical testing firm is hired to test power system relays, circuit breakers, and oil testing of oil filled switches and transformers. The Electrical Engineering Group prepares specifications and provides project management services for the electrical maintenance testing process with field support provided by the Electrical Maintenance Section. Proper operation of the power systems, motors, motor controls, and pumping system controls at the pumping stations is critical.

MMSD's pumping stations typically have two redundant utility power services. The two exceptions include Pumping Stations 3 and 17. However, Pumping Station 17 does have a backup generator on-site to provide redundant power. Each redundant service or the backup generator, as in the case of PS 17, will automatically connect to provide power in the event of a normal power outage. Since the pumping stations operate continuously, it is important that these automatic transfer systems are well maintained and function properly when required. To insure this, the transfer schemes are inspected and tested at least semi-annually and the generator at Pumping Station 17 is tested monthly by the mechanics. The mechanics start the generator manually and verify that it is providing power to the station. The generator runs for two hours and then automatically shuts off and the station is switched back to utility power. The Metrogro mechanics perform an annual inspection of the generator, which includes an oil change. If the generator would run more than normal, another oil change would be scheduled at other times during the year as needed. The Metrogro mechanics are familiar with large diesel engines and therefore, familiar with the engine that drives the generator at Pumping Station 17 as well as the portable generators that the District owns.

The motor control systems, starters, and or adjustable frequency drives (AFDs), especially for the wastewater pumps, are routinely inspected for bad components, loose connections, and worn contacts. Components in poor condition are repaired or replaced prior to failure. Although it is sometimes difficult to assess the condition of solid-state equipment such as solid-state starters and adjustable speed drives, these enclosures are also cleaned and the equipment inspected for signs of overheating or other damage. The equipment is checked for proper operation prior to returning it to service.

Most of the control systems, such as the pump control system, are now controlled via programmable logic controllers or another programmable device. Since these generally either work or they do not work, it is important to have a backup control system or backup plan in the event of equipment failure. It is generally difficult to predict when this type of equipment will fail. Although older control systems have more individual components, it is generally not any easier to predict failures. After proper operation of the control and alarm systems is initially verified, keeping the instrumentation components calibrated and working well, and testing alarm functionality periodically is probably as much as can be done. The periodic testing of alarms should include testing of the telemetry system to verify that all alarms show up properly on the operator’s screen at the plant.

The lighting systems, although important from the standpoint of allowing maintenance personnel to see what they are working on, probably receive less attention than most of the other systems, simply because they require little maintenance and they play a supporting role versus a critical role to the mission of the pumping station. Burnt out lamps are generally replaced by the Building and Grounds Crew. If there is something wrong with the fixture, e.g., bad ballast, a work order is generated for the electricians to take corrective action.

<u>Buildings and Grounds</u>

The pumping station structure, building exterior, roof, and site maintenance are taken care of by MMSD’s Building and Grounds Crew.

The Building and Grounds Crew annually inspects each pumping station’s roof and exterior for structural damage and leaks. Any leaks or damages that are reparable by the crew are fixed, while those that are not are either contracted for repair or budgeted for repair during the next year. Leaks or damage that require immediate attention are repaired while those that can wait are budgeted for.

The interiors of wetwells and drywells typically require little maintenance. However, occasional repairs to damaged concrete are required. If these are not too extensive, the Buildings and Grounds Crew may make these small structural repairs. If extensive rehabilitation is required, it is generally dealt with as a contracted service managed by the Engineering Department. Painting of piping, equipment, and sometimes walls, is done as necessary, usually on a rotating basis, and may be done internally or contracted out depending upon the size of the project and the pending workload. A good fresh coat of paint adds significantly to the neat and tidy appearance of the pumping station.

The Building and Grounds Crew keeps the pumping stations aesthetically pleasing externally and internally. Trash within the building is removed and floors swept and cleaned periodically. The lawn and landscaping are well cared for. MMSD's pumping station sites are often located near neighborhoods or parks, and it is important that the site be kept clean, well landscaped, and well groomed. A good appearance is less likely to bring negative attention to the pumping station. A good internal appearance also provides for a better working environment for the mechanics and electricians.

To minimize the build up of grease and solids in Pumping Station wetwells, some stations have an automatic well cleaning sequence programmed into the station control system. This sequence runs during the nighttime hours and results in the station pumps lowering the well level to a lower than normal level. The pumping station's pumps then pump most of the floating and settled material from the well under these conditions. Unfortunately, some wetwells are more susceptible to solids and grease build-up than others and therefore need more cleaning than can be provided using the pumping systems. To deal with this issue, the Buildings and Grounds Crew periodically hires the City of Madison to provide a vactor truck to assist in cleaning these wetwells. Typical solids include grease, rags, and other non-organic materials. The method of removal is to high-pressure spray the wells while pumping the wash water into the vactor truck.

Maintenance of Intercepting Sewers and Force Mains

MMSD's wastewater collection system currently includes 96 miles of gravity intercepting sewers, 29 miles of raw wastewater force mains, and 1,551 manholes. These pipelines and manholes are responsible for collecting and transmitting the wastewater from the various communities to and between MMSD's 17 pumping stations, and ultimately to the Nine Springs Wastewater Treatment Plant. MMSD staff follows a written interceptor maintenance guideline that has been used and revised since 1992. This section presents a summary of MMSD's *Interceptor Maintenance Program Guidelines* (latest (3rd) revision – Nov. 2009), which is included as Appendix A4. The interceptor maintenance program defines seven areas that are each addressed with a separate plan. The seven areas and their separate plans are summarized in turn:

Interceptor Evaluations

MMSD has developed a formalized interceptor evaluation program that keeps staff members informed about the physical condition and hydraulic adequacy of its individual gravity interceptors, and allows informed decisions regarding the need for rehabilitation or replacement projects. The program includes televising, cleaning, manhole inspection, flow documentation, and various other work. Interceptor evaluations are performed on roughly 10% of MMSD's gravity sewers each year. The program includes systematic recordkeeping and organization of the work. The program has been successful in identifying system needs prior to their becoming emergencies, and has allowed MMSD to more efficiently plan, budget and carry out the necessary repairs and rehabilitation projects

As noted above, approximately 10%, or nine miles, of MMSD interceptors are evaluated each year. During this process, the interceptors are cleaned (e.g., grit and roots are removed) and televised. Following televising of the interceptors, MMSD receives video documentation of the televising. MMSD personnel then view the results in detail and enter any defects noted into a database. The database assigns a score to the interceptor based on the condition observed during the televising results. The scores are used to rank the overall condition of the interceptor and prioritize the need for any repairs. As interceptors are re-inspected every 10 years or so, new scores will be assigned and condition of the interceptor can be compared to the previous inspection.

Force Main Isolation Valve Exercising

Eighteen exterior isolation valves presently exist on MMSD’s force main sewers (an up-to-date listing of the actual number and status of these valves is maintained in MMSD’s *Interceptor Maintenance Program Guidelines* – the most recent version is included in Appendix A4). Some of these valves are located immediately outside of pumping stations and were designed to limit possible pumproom flooding in the event of a burst header inside the pumping station. Several others were added at specific forcemain junction points to allow diversion of flow as part of a construction project. Most of MMSD’s older isolation valves are double-disc gate valves. Newer valves are resilient-wedge gate valves or plug valves. Since the seating area can become filled with grit and solids that can prevent full seating of any type of valve, each valve is regularly exercised and inspected by MMSD twice per year. Valve exercising verifies that the valve is operational and in working order, but does not automatically verify that the valve is fully sealing off the flow. Some valves may leak even though their valve stem exercises freely to closure, and may require additional rehab work when needed. The valve exercising program is intended to maintain the valves in good working condition and to help insure, but not guarantee, that the valves will work and seal properly when they are needed.

Air Valve Inspection and Maintenance

There are twenty-eight air release valve installations on MMSD's raw wastewater forcemains (an up-to-date listing of the actual number and status of these valves is maintained in MMSD's *Interceptor Maintenance Program Guidelines* – the most recent version is included in Appendix A4). Most of MMSD's air valves are "combination" valves, i.e. they perform both a vacuum breaking function and an air release function. The vacuum breaking function admits air into the forcemain during low pressure conditions (such as during pump shutdowns), thus preventing possible vapor cavity formation & water column separations which could lead to waterhammer failures. The air release function prevents air pockets from accumulating and potentially restricting the flow at forcemain high points. To ensure that each valve remains in working order, each air valve is inspected and cleaned twice each year, or more frequently when the valves are prone to plugging. If possible the valves are cleaned and repaired in the field. In most cases, the valve must be removed and returned to the shop where it can be inspected and cleaned prior to reinstallation at the site.

Siphon Cleaning

Eleven active inverted siphons currently exist in MMSD's collection system (an up-to-date listing of the actual number and status of the siphons is maintained in MMSD's *Interceptor Maintenance Program Guidelines* – the most recent version is included in Appendix A4). The purpose of a siphon is to carry the wastewater flow beneath an obstacle (such as a streambed or a major utility line) that would otherwise block the interceptor's gravity profile. Unfortunately, a siphon typically carries a lower velocity (since it always flows full) and thus creates greater potential for solids deposition. Newer siphons with multiple barrels are designed to minimize the potential for solids deposition. MMSD began contracting out the regular annual cleaning of its siphons in 1998. Prior to 1998, siphons were cleaned only if specific problems occurred. Annual contracted siphon cleaning helps to catch any problems before they become serious. The contractor's cleaning operations are closely observed, and the adjacent siphon manholes are visually inspected at the time of cleaning to determine if any additional work is needed.

Stoplog & Gate Structures

There are eight stoplog and gate structures on MMSD interceptors (an up-to-date listing of the actual number and status of these structures is maintained in MMSD's *Interceptor Maintenance Program Guidelines* – the most recent version is included in Appendix A4). Some of these structures were constructed at junction points between adjacent interceptor projects. Others were originally constructed as flushing manholes (no longer used) for the purpose of periodic flushing of the interceptor with adjacent surface water. To ensure that the stoplog and flapgate structures remain in good repair, MMSD inspects each structure annually and provides any stoplog or gate replacements or repairs that are needed.

Special Projects, Events, and Repairs

In addition to the regular planned maintenance activities, there are numerous specific projects, repairs and events that occur every year in the operation and maintenance of interceptors and force mains. Examples include high flow events, emergency repairs, connection inspections, odor complaints, backup events, I/I work, specific manhole repairs, surface route inspections, and other events. These specific events are an important aspect of the interceptor maintenance program. Therefore, specific records of these events are kept for future decisions and management of the MMSD program.

Program Coordination and Management

Coordination and management of the interceptor maintenance program includes numerous functions needed to make the program successful. Examples include the following:

- Preparing annual program budget and tracking it during the year
- Tracking of work performed and work outstanding
- Updating interceptor GIS database and maps
- Managing inventory
- Managing contractors
- Managing Diggers' Hotline membership and locating services
- Organization of emergency preparedness
- Screening outside projects via UTILITY log.
- Organizing cross-training activities
- Recommending periodic improvements to the program

The interceptor and forcemain maintenance program is carried out as a joint effort of MMSD's Operations and Maintenance Department, MMSD's Engineering Department, and outside contractors. MMSD's Collection System Supervisor currently handles oversight of the entire program. MMSD's Monitoring Services/Sewer Maintenance Crew carries out most of the field activities, including inspection and maintenance of valves and stop logs, manhole repairs, and response to odor or backup complaints. Locates and field marking are handled as a contracted service, presently provided by United States Infrastructure Corporation (USIC). Televising and cleaning work is annually bid and contracted. MMSD's Engineering Department provides engineering and assistance for major projects and special events, and maintains system maps and the Geographical Information System (GIS). Major repairs, excavation, heavy construction and specialty services are contracted out to private construction firms.

Summary

MMSD's collection system represents a significant investment and an important asset for the protection of public health and the environment. To preserve that investment requires a diligent and thorough maintenance program. MMSD uses a program driven approach to maintenance intended to reduce the number of emergency maintenance events. All components of MMSD's collection system are inspected and maintained to insure that proper operation of MMSD’s system continues. Components that are found in poor condition are repaired or replaced prior to failure. Detailed records of maintenance, high flow events, and failures are kept for future reference and decision-making. MMSD’s program will not prevent all failures; however, a sound maintenance program has and will continue to maximize the life and usefulness of MMSD’s collection system components.

Chapter 8
Addressing I/I Issues and High Flows

Chapter Outline

This chapter is organized into the following sections:
- Introduction
- Background
- Estimation of Infiltration Volume
- Conveyance Costs
- Effect of Climate Change
- Peaking Factors
- I/I Mitigation Strategies

Introduction

The inflow and infiltration (I/I) of clear water into the sanitary sewer collection system is a concern for several reasons. It can result in environmental damage through sanitary sewer overflows, damage the property of system users, and lead to increased costs for conveyance and treatment. This chapter will evaluate the impact of I/I on the District's collection system, examine how the collection system is designed to accommodate these flows, what factors contribute to increased levels of I/I, and what measures can be undertaken to mitigate the impact of I/I.

Background

All sanitary sewer collection systems infiltrate clear water to some degree. Properly designed sewers employ the use of a peaking factor to account for these extraneous flows. Since 1961 MMSD has used the "Madison Design Curve" as a guide in determining the appropriate peaking factor for its wastewater conveyance facilities (see Chapter 4). In general the District's use of this peaking factor in design has proven adequate for conveying wet weather flows over the past 50 years.

MMSD's collection system has experienced a number of high flow events in the last twenty years. Several of these events have resulted in peak flows greater than those predicted by the Madison Design Curve. As such, it is reasonable to question if the Madison Design Curve is an adequate design standard for future conveyance projects.

Estimation of I/I Volume

Attempting to quantify the amount of I/I in a collection system is challenging. The term infiltration is generally used to account for clear water that enters the collection system directly from groundwater through cracks and joints in the piping network. Since this type of flow is relatively constant over time, it is easier to estimate than inflow through flow metering and pump records. Inflow generally describes storm water that directly flows into the sewer system through defects in manhole covers and cross-connections with storm water conveyance facilities (i.e. residential roof drains, sump pumps, municipal storm sewers, etc.). Inflow volumes and rates are responsive to a number of rainfall characteristics such as amount, intensity and duration and thus are difficult to quantify.

Table 8.1 includes an estimate of total daily infiltration volumes for Year 2010 in the District's collection system by pump station service area. These volumes were derived by CARPC in their C*ollection System Evaluation (2009)* by subtracting estimated dry weather wastewater flows from MMSD's metered flow data and pumping records.

Table 8.1 also includes an estimate of infiltration volumes directly into MMSD's interceptor sewers based on television inspection results. MMSD maintains a database that estimates infiltration rates in each segment of MMSD sewer based on closed circuit television inspection and/or industry standards. Infiltration of clear water into District sewers accounts for approximately 29% of the total infiltration that is conveyed to the treatment plant. The remaining 71% of the total infiltration is attributable to the conveyance systems of the District's satellite communities. A schematic of the total daily infiltration rates throughout the District's collection system is presented in Figure 8.1.

Infiltration per unit length of MMSD's interceptor sewers are also calculated for 2010 for each pump station service area in Table 8.1. The results show that the service areas for PS1, PS15, PS16, and PS17 are relatively tight systems. In the cases of PS15, PS16, and PS17 this is most easily explained by the fact that these service areas have been developed more recently and employ the use of better construction materials. PS1 had historically been a very problematic area with regard to infiltration and inflow. However, the 2002 replacement of the North Basin Interceptor has reduced infiltration dramatically in this basin.

Infiltration into MMSD's Rimrock Interceptor upstream of PS3 is estimated to be moderate, although the overall infiltration rate in the PS3 basin is significant. More investigation of I/I in this basin is recommended (see Appendix 5 for further details).

Service areas that exhibited high infiltration rates per unit length of interceptor in 2010 include those for PS6, PS9, PS12, PS13, and PS14. The PS6 basin is unique in that the length of MMSD interceptors is small relative to the overall service area since a significant portion of the flow is conveyed to PS6 through City of Madison interceptors. The City of Madison plans to perform a study of its conveyance facilities in a portion of the PS6 service area in 2011.

Table 8.1
MMSD Infiltration by Pumping Station Basin (2010)
Madison Metropolitan Sewerage District

Pump Station Basin	Estimate of Total Infiltration by Pump Station Basin[1] (gpd)	MMSD Infiltration by Pump Station Basin[2] (gpd)	MMSD Infiltration as Percentage of Total Infiltration (%)	Total MMSD Interceptor Mileage in Service Area (miles)	MMSD Infiltration per Unit Length of Interceptor (gpd/mile)
1[3]	480,000	505	0%	1.71	296
2	270,048	44,255	16%	2.73	16,202
3	84,000	5,309	6%	0.72	7,377
4	81,000	12,055	15%	1.55	7,789
5	204,450	44,655	22%	3.00	14,880
6	140,000	67,414	48%	1.91	35,312
7	360,000	218,130	61%	19.76	11,040
8	640,000	322,715	50%	14.64	22,042
9	136,000	16,735	12%	0.63	26,487
10	206,000	75,645	37%	6.59	11,481
11	525,000	225,956	43%	10.04	22,504
12[4]	535,000	238,075	45%	7.86	30,304
13	990,000	76,040	8%	2.96	25,656
14	1,170,000	411,255	35%	15.84	25,958
15	130,000	6,804	5%	1.97	3,447
16	177,000	6,617	4%	1.63	4,060
17	31,000	0	0%	2.52	0
TOTAL	6,159,498	1,772,165	29%	96.06	18,448

(1). Source: CARPC's *MMSD Collection System Evaluation (January 2009).* Includes all infiltration into sewers owned by MMSD and satellite communities in Year 2000.

(2). Includes only infiltration into MMSD sewers. MMSD infiltration derived from inspection records and/or industry standards. Values reflect Year 2010 conditions.

(3). PS1 infiltration based on CARPC's Year 2030 estimate to reflect MMSD's North Basin Interceptor Replacement in 2005.

(4). CARPC's estimate for infiltration in PS 12 basin is 208,000 gallons. Estimated infiltration in this basin was revised upwards based on 2009-2010 flow metering data and MMSD infiltration calculations for this basin.

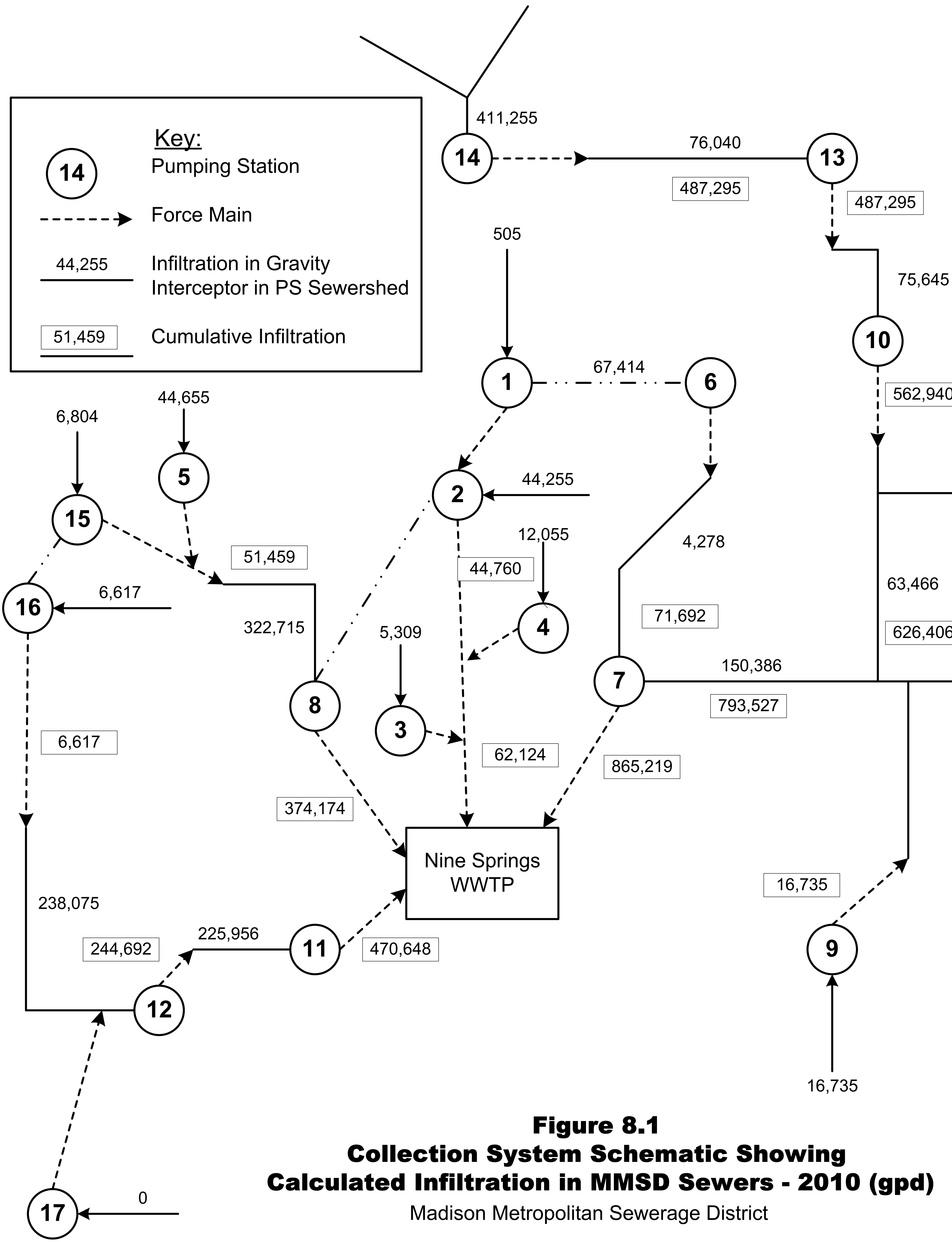


Figure 8.1
Collection System Schematic Showing Calculated Infiltration in MMSD Sewers - 2010 (gpd)
Madison Metropolitan Sewerage District

CARPC's estimated 2010 wastewater flows for each pump station service area are compared to actual pumping records in Table 8.2. The infiltration rates computed from this data in the PS1, PS2, PS8, and PS15 basins result in negative values, indicating either inaccurate flow data or overestimates of wastewater flows. Whatever the cause, the negative values suggest that these basins are relatively tight with regard to the infiltration of clear water. More importantly, Table 8.2 shows that infiltration is significant in the PS3, PS5, PS7, PS13, and PS14 basins (i.e. infiltration rate >25% of average daily flow).

I/I studies were recommended for the PS9, PS12, PS13 and PS14 service areas in the District's *2002 Collection System Facilities Plan.* Flow monitoring and I/I investigations in the PS9 and PS12 basins were performed by Strand Associates in 1999. While flow monitoring did confirm high peaking factors in some of the PS12 subbasins, no definitive sources of inflow or infiltration were discovered.

The City of Madison completed an I/I investigation in a portion of the PS13 basin in 2005. The City relined approximately 3,000 feet of its 24" Anderson Street Interceptor in 2008 in response to a recommendation from this study. The City also relined approximately 8,000 feet of smaller diameter sewer and 40 manholes as part of this project from 2008 to 2010.

Additional rehabilitation work in basins with significant infiltration was completed by MMSD in 2011. Approximately 2,800 feet of MMSD's West Interceptor in the PS5 basin was rehabilitated with a cured-in-place liner (MH05-011 to MH05-021).

It is recommended that additional studies be performed in the PS3, PS7, and PS14 basins, with PS14 receiving the highest priority. No formal I/I study of the PS14 service area has been performed and it is recommended that one be conducted in the next one to two years based on the recommendation of the *2002 Collection System Facilities Plan*, the sewer system overflows observed in this basin during the June 8, 2008 rain event, and the data presented in Tables 8.1 and 8.2.

Conveyance Costs

As mentioned at the beginning of this chapter, one of the primary reasons to identify and remove I/I in a collection system is to reduce conveyance costs. The District has a large number of pumping stations relative to its service area and the pumping of clear water can result in correspondingly large and unnecessary pumping costs. Given the layout of the District's collection system, some clear water flows are pumped as many as five times. Figure 8.2 shows the unit costs to pump clear water flows from each of the District's 17 pump station service areas in 2010.

In looking at Figure 8.2, it can be seen that PS16 has the highest unit pumping rate at $0.22/1000 gallons. This rate is attributed primarily to the high system head of 182 feet for PS16. Another reason for the elevated rate for PS16 is that flows from this station

Table 8.2
Total Infiltration by Pumping Station Basin (2010)
Madison Metropolitan Sewerage District

Pump Station Basin	Actual Average Daily Flow by Pump Station Basin[1] (mgd)	CARPC Estimated Wastewater Flows[2] (mgd)	Estimated Total Infiltration by Pump Station Basin (gpd)	Infiltration Rate as Percentage of Average Daily Flow (%)
1	4.16	4.59	-425,000	N/A
2	3.52	4.08	-559,000	N/A
3	0.32	0.23	90,000	28%
4	1.02	0.90	125,000	12%
5	0.70	0.42	280,000	40%
6	1.55	1.47	82,000	5%
7	5.41	3.09	2,323,000	43%
8	5.20	5.84	-642,000	N/A
9	0.83	0.76	72,000	9%
10	2.53	2.51	19,000	1%
11	3.21	2.85	356,000	11%
12	2.85	2.53	320,000	11%
13	2.07	1.43	643,000	31%
14	4.23	2.60	1,635,000	39%
15	1.33	1.34	-11,000	N/A
16	1.81	1.65	159,000	9%
17	0.89	0.82	72,000	8%
TOTAL	41.63	37.09	4,538,738	11%

(1). Year 2010 actual average daily flows are based on metered data for PS1, 2, 3, 5, 7, 8, 10, 11, 16 and 17. Pump run-time records are used at all other stations.
(2). Source: CARPC's *MMSD Collection System Evaluation (January 2009).* Estimate includes only projected wastewater flows for 2010. Infiltration not included.

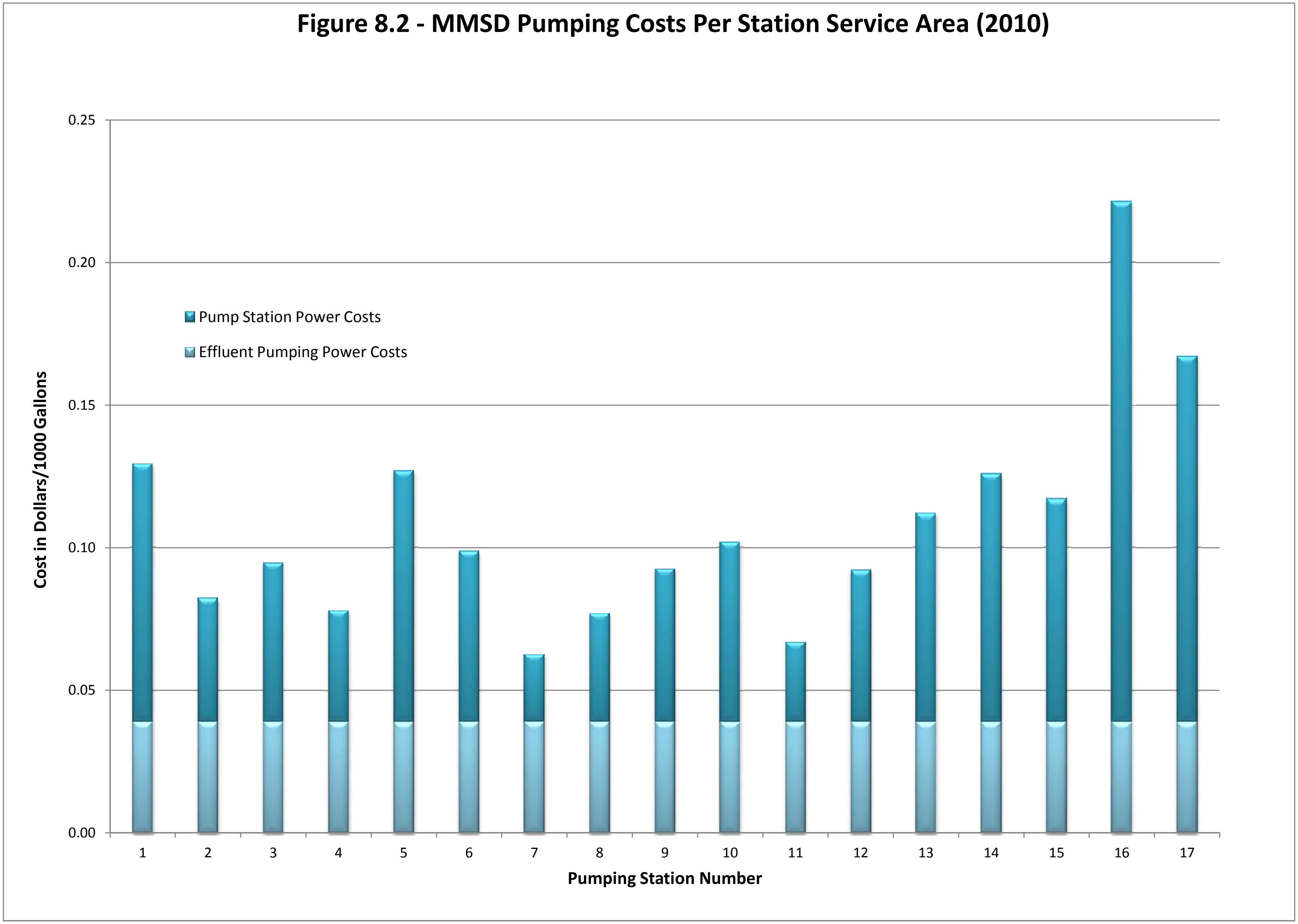


Figure 8.2 - MMSD Pumping Costs Per Station Service Area (2010)
Pump Station Power Costs
Effluent Pumping Power Costs
Cost in Dollars/1000 Gallons
0.25
0.20
0.15
0.10
0.05
0.00
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
Pumping Station Number

also pass through PS12 and PS11. A similar situation occurs for PS13 and PS14. Even though both of these pump stations have relatively low system heads, their unit pumping rates are higher than average since their flows also pass through PS10 and PS7. Thus, it makes sense that a gallon of clear water removed in upstream service areas such as PS13, PS14, or PS16 would result in greater energy savings than a gallon of clear water which is removed in the PS7 service area.

Table 8.3 contains a summary of the costs to pump all infiltration to the treatment plant and from the treatment plant through the effluent force mains in 2010. The estimated cost to pump infiltration in this year was approximately $235,000, with approximately $87,000 of that total representing effluent pumping. The average annual cost to pump infiltration at each station in 2010 was $8,700. PS7 and PS10 had infiltration pumping costs over three times the average value, which is partially a result of these stations conveying infiltration from the PS13 and PS14 service areas.

Table 8.4 shows the costs to pump various rates of infiltration in each pump station service area in 2010. It also calculates the 50-year present worth costs of pumping these same infiltration rates. The present worth analysis takes into consideration that energy rates increase on an annual basis and must be accounted for in determining pumping costs as they are a significant factor. In looking at Table 8.4, the costs to pump infiltration in a "leaky" basin such as PS14 can be significant. Infiltration rates in this service area are estimated to be 750 gpm (1.08 mgd) or more, resulting in a 50-year present worth pumping cost of $5.3 million.

Given these costs, it is reasonable to question whether it is more cost efficient to continue to pump these extraneous flows or provide methods of sewer rehabilitation which mitigate the infiltration. A rudimentary analysis is presented in Table 8.5 to assess the cost effectiveness of rehabilitation relative to infiltration conveyance. In this analysis the pumping costs associated with infiltration into MMSD interceptors is calculated over a 50-year period. This present worth cost is then used to calculate the length of MMSD's sewers that could be rehabilitated through a cured-in-place liner to mitigate infiltration. It is important to note that only the pumping costs associated with infiltration into MMSD sewers are accounted for in this analysis. As discussed earlier, approximately 70% of the total infiltration amount in the District's collection system comes from the District's satellite communities.

As shown in Table 8.5, the money saved from reductions in pumping of infiltration does not provide for much sewer rehabilitation in each service area. On average only 9% of the District's interceptors could be rehabilitated with a cured-in-place liner with the money saved from reduced pumping costs. There are several problems with approaching sewer rehabilitation in this fashion. Most importantly, it is extremely difficult to identify definitive infiltration sources. Further, it is very unlikely that all or even a majority of the infiltration is occurring in only 9% of the interceptor length. A meaningful reduction in infiltration for any service area may require that 25%-50% of the interceptors be rehabilitated. Finally, as mentioned previously, even if the District's entire sewer

Table 8.3
Infiltration Pumping Costs by MMSD Pumping Station (2010)
Madison Metropolitan Sewerage District

	Cumulative Infiltration at MMSD Pump Station[1] (gpd)	2010 Unit Pumping Cost ($/MGal)	2010 Annual Cost to Pump Infiltration ($/yr)
1	480,000	$47.27	$8,283
2	750,048	$43.26	$11,844
3	84,000	$55.70	$1,708
4	81,000	$39.17	$1,158
5	204,450	$49.97	$3,729
6	140,000	$36.09	$1,844
7	3,002,000	$23.71	$25,975
8	974,450	$38.04	$13,531
9	136,000	$29.93	$1,486
10	2,366,000	$39.37	$33,999
11	1,268,000	$27.85	$12,887
12[3]	743,000	$25.26	$6,850
13	2,160,000	$10.14	$7,991
14	1,170,000	$13.87	$5,924
15	130,000	$40.10	$1,903
16	177,000	$129.28	$8,352
17	31,000	$75.13	$850
		Average PS Cost	$8,724
Infiltration to WWTP[2]	6,159,498	-	-
Effluent Pumping	6,159,498	$38.91	$87,477
TOTAL PUMPING COSTS (2010)			**$235,000**

Notes:

(1). Source: CARPC's *MMSD Collection System Evaluation (January 2009).* Includes MMSD's collection system and satellite community systems.

(2). Includes cumulative infiltration from PS 2, 3, 4, 7, 8, & 11.

(3). CARPC's estimate for infiltration in PS 12 basin is 208,000 gallons. Estimated infiltration in this basin was revised upwards based on 2009-2010 flow metering data and MMSD infiltration calculations for this basin.

Table 8.4
Pumping Costs for MMSD Pumping Stations for Various Rates of Infiltration (2010)
Madison Metropolitan Sewerage District

Pumping Station Service Area	Pumping Station Costs ($/1000 gal)	Effluent Pumping Costs ($/1000 gal)	Total Pumping Costs (Fig. 8-2) ($/1000 gal)	Annual Cost to Pump Various Infiltration Rates (2010 $/yr) (Infiltration rates in gpm)									
				1	5	10	25	50	100	250	500	750	1,000
1	0.0905	0.0389	0.1294	68	340	680	1,701	3,402	6,804	17,010	34,019	51,029	68,038
2	0.0433	0.0389	0.0822	43	216	432	1,080	2,160	4,319	10,798	21,595	32,393	43,191
3	0.0557	0.0389	0.0946	50	249	497	1,243	2,486	4,973	12,432	24,864	37,296	49,729
4	0.0392	0.0389	0.0781	41	205	410	1,026	2,052	4,104	10,259	20,519	30,778	41,038
5	0.0880	0.0389	0.1269	67	334	667	1,668	3,335	6,671	16,677	33,354	50,031	66,709
6	0.0598	0.0389	0.0987	52	259	519	1,297	2,594	5,188	12,969	25,938	38,908	51,877
7	0.0237	0.0389	0.0626	33	165	329	823	1,646	3,291	8,228	16,455	24,683	32,910
8	0.0380	0.0389	0.0770	40	202	404	1,011	2,022	4,045	10,112	20,223	30,335	40,447
9	0.0536	0.0389	0.0925	49	243	486	1,216	2,432	4,864	12,160	24,321	36,481	48,642
10	0.0631	0.0389	0.1020	54	268	536	1,340	2,680	5,360	13,401	26,802	40,202	53,603
11	0.0278	0.0389	0.0668	35	175	351	877	1,754	3,509	8,772	17,543	26,315	35,086
12	0.0531	0.0389	0.0920	48	242	484	1,209	2,418	4,836	12,090	24,181	36,271	48,362
13	0.0732	0.0389	0.1121	59	295	589	1,473	2,947	5,893	14,733	29,465	44,198	58,931
14	0.0871	0.0389	0.1260	66	331	662	1,656	3,311	6,622	16,556	33,111	49,667	66,222
15	0.0781	0.0389	0.1171	62	308	615	1,538	3,076	6,152	15,381	30,761	46,142	61,523
16	0.1824	0.0389	0.2213	116	582	1,163	2,908	5,816	11,631	29,078	58,155	87,233	116,310
17	0.1282	0.0389	0.1671	88	439	878	2,196	4,392	8,785	21,962	43,925	65,887	87,850

Pumping Station Service Area	Pumping Station Costs ($/1000 gal)	Effluent Pumping Costs ($/$1000 gal)	Total Pumping Costs (Fig. 8-2) ($/1000 gal)	50-Year Present Worth Costs to Pump Various Infiltration Rates (2010 $) (Leakage rates in gpm)									
				1	5	10	25	50	100	250	500	750	1,000
1	0.0905	0.0389	0.1294	7,261	36,307	72,614	181,535	363,071	726,141	1,815,353	3,630,707	5,446,060	7,261,414
2	0.0433	0.0389	0.0822	4,610	23,048	46,095	115,239	230,477	460,954	1,152,386	2,304,771	3,457,157	4,609,542
3	0.0557	0.0389	0.0946	5,307	26,537	53,073	132,683	265,366	530,731	1,326,828	2,653,655	3,980,483	5,307,311
4	0.0392	0.0389	0.0781	4,380	21,899	43,798	109,495	218,989	437,979	1,094,947	2,189,895	3,284,842	4,379,789
5	0.0880	0.0389	0.1269	7,120	35,598	71,195	177,988	355,976	711,952	1,779,879	3,559,758	5,339,637	7,119,516
6	0.0598	0.0389	0.0987	5,537	27,683	55,366	138,415	276,830	553,660	1,384,149	2,768,299	4,152,448	5,536,598
7	0.0237	0.0389	0.0626	3,512	17,562	35,124	87,810	175,619	351,238	878,096	1,756,191	2,634,287	3,512,383
8	0.0380	0.0389	0.0770	4,317	21,583	43,167	107,917	215,835	431,670	1,079,174	2,158,349	3,237,523	4,316,697
9	0.0536	0.0389	0.0925	5,191	25,957	51,913	129,783	259,565	519,131	1,297,826	2,595,653	3,893,479	5,191,306
10	0.0631	0.0389	0.1020	5,721	28,604	57,208	143,021	286,041	572,083	1,430,207	2,860,413	4,290,620	5,720,826
11	0.0278	0.0389	0.0668	3,745	18,723	37,446	93,616	187,231	374,463	936,157	1,872,313	2,808,470	3,744,627
12	0.0531	0.0389	0.0920	5,161	25,807	51,614	129,036	258,071	516,143	1,290,357	2,580,715	3,871,072	5,161,430
13	0.0732	0.0389	0.1121	6,289	31,447	62,894	157,236	314,471	628,942	1,572,355	3,144,711	4,717,066	6,289,422
14	0.0871	0.0389	0.1260	7,068	35,338	70,676	176,691	353,381	706,762	1,766,905	3,533,811	5,300,716	7,067,622
15	0.0781	0.0389	0.1171	6,566	32,830	65,660	164,151	328,302	656,603	1,641,508	3,283,016	4,924,524	6,566,032
16	0.1824	0.0389	0.2213	12,413	62,066	124,133	310,332	620,663	1,241,326	3,103,315	6,206,630	9,309,945	12,413,260
17	0.1282	0.0389	0.1671	9,376	46,879	93,758	234,396	468,792	937,584	2,343,959	4,687,919	7,031,878	9,375,838

<u>Assumptions</u>

Interest rate =	3.00%
Energy escalation rate =	6.00%
Term (yrs) =	50

Table 8.5
Life Cycle Costs for Pumping of Infiltration (2010)
Madison Metropolitan Sewerage District

Goal of Analysis: Determine the length of MMSD interceptor sewers that could be rehabilitated in each service area for the 50-year present worth cost to pump infiltration from that service area.

	INFILTRATION PUMPING COSTS (2010 $)					PIPE REHABILITATION (SEWER LINING)				
Pumping Station Service Area	Pumping Station Costs ($/1000 gal)	Effluent Pumping Costs ($/1000 gal)	Total Pumping Costs ($/1000 gal)	MMSD Infiltration by PS Service Area (gpd)	50-year PW Cost to Pump MMSD Infiltration ($)	Average Pipe Diameter in Service Area (in)	Sewer Lining Unit Cost ($/ft)	Length of MMSD Interceptor to be Lined(5) (ft)	Total MMSD Interceptor Length in Service Area (ft)	Fraction of MMSD Interceptors to be Rehabilitated (%)
1	0.0905	0.0389	0.1294	505	$2,547	30	125	20	9,029	0%
2	0.0433	0.0389	0.0822	44,255	$141,663	27	115	1,232	13,464	9%
3	0.0557	0.0389	0.0946	5,309	$19,567	12	60	326	3,802	9%
4	0.0392	0.0389	0.0781	12,055	$36,666	21	90	407	8,184	5%
5	0.0880	0.0389	0.1269	44,655	$220,779	15	70	3,154	15,840	20%
6	0.0598	0.0389	0.0987	67,414	$259,197	33	150	1,728	10,085	17%
7	0.0237	0.0389	0.0626	218,130	$532,053	33	150	3,547	104,333	3%
8	0.0380	0.0389	0.0770	322,715	$967,405	27	115	8,412	77,299	11%
9	0.0536	0.0389	0.0925	16,735	$60,331	24	100	603	3,326	18%
10	0.0631	0.0389	0.1020	75,645	$300,522	48	225	1,336	34,795	4%
11	0.0278	0.0389	0.0668	225,956	$587,584	36	175	3,358	53,011	6%
12	0.0531	0.0389	0.0920	238,075	$853,338	33	150	5,689	41,501	14%
13	0.0732	0.0389	0.1121	76,040	$332,116	48	225	1,476	15,629	9%
14	0.0871	0.0389	0.1260	411,255	$2,018,469	27	115	17,552	83,635	21%
15	0.0781	0.0389	0.1171	6,804	$31,026	27	115	270	10,402	3%
16	0.1824	0.0389	0.2213	6,617	$57,041	27	115	496	8,606	6%
17	0.1282	0.0389	0.1671	0	$0	33	150	0	13,306	0%
Average				1,772,165						9%

Notes/Assumptions
(1). Interest rate = 3.00%
(2). Energy escalation rate = 6.00%
(3). Analysis Term (yrs) = 50
(4). Service life of rehabilitated (lined) sewer = 50 years.
(5). Length of MMSD interceptors to be lined is calculated using the 50-year present worth value for pumping of infiltration.

network were rehabilitated to eliminate infiltration sources, only 30% of the problem would be addressed.

In summary, it is apparent that it is more cost efficient for the District to convey infiltration to and from the treatment plant rather than to adopt an aggressive, regional sewer rehabilitation program across each service area to eliminate infiltration sources. It is also clear, however, that infiltration into the sanitary sewer network reduces overall conveyance capacities and can lead to premature replacement or reinforcement of certain sections. Excessive infiltration into the sewer network also depletes the groundwater supply and can disrupt watershed balances, if excessive. With these considerations in mind, a more systematic approach for dealing with the issue of infiltration is needed, as discussed in following sections of this chapter.

Effect of Climate Change

The Madison area has experienced a number of severe storms in the last twenty years. Both the volume and intensity of these storms has overwhelmed the collection system of the District and its satellite communities on occasion. Historical rainfall data over the past 50-60 years for the Madison area shows a noticeable rise in volume. From 1950 to 2006 the annual average precipitation in Dane County has increased approximately 5.5 inches (*Center for Climatic Research & Center for Sustainability and the Global Environment, Nelson Institute, UW-Madison).* Figure 8.3 shows the general rise in annual precipitation measured at various cities in the state of Wisconsin during this time period. These trends would seem to indicate that a significant change in climatic patterns in the Madison area is taking place with regard to rainfall.

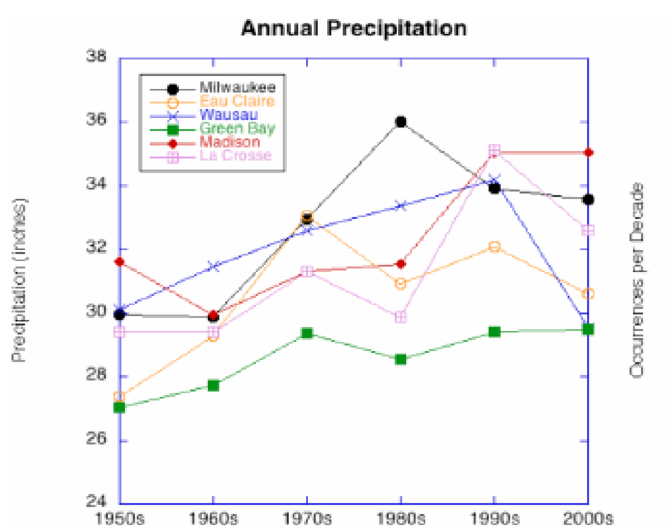


Figure 8.3 – Historical Annual Precipitation for Wisconsin Cities
Source: Kucharik, C.J. S.P. Serbin, E.J. Hopkins, S. Vavrus, and M.M. Motew, 2010: Patterns of climate change across Wisconsin from 1950 to 2006

A more in-depth analysis of the data suggests that these trends are not so clearly definable. In looking at a longer historical rainfall record for the Madison area, members of the *Wisconsin Initiative on Climate Change Impacts* (WICCI) determined that there does not appear to be a statistically significant increasing trend in annual precipitation from 1869 to 2008 (Figure 8.4).

Similarly, an analysis was performed to see if the magnitude and frequency of intense rainfall events has increased over this same time period. From this analysis it was determined that while an increase in the magnitude of storm events in the Madison area does not appear to be statistically significant, the occurrence of five 3" daily storm events from 2004-2008 does suggest an increase in intensity (Figure 8.5). For the purposes of this analysis an "event" was defined as any one-day precipitation total, while "an intense event" is defined as a daily precipitation total that exceeds a threshold of three inches.

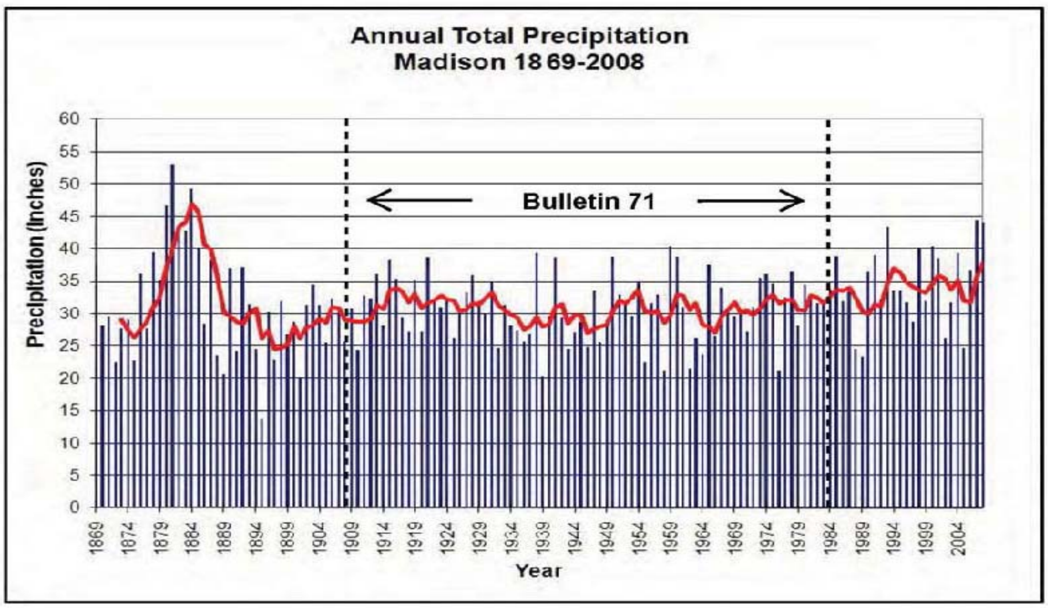


Figure 8.4 – Annual Total Precipitation in Madison (1869-2008)

Source: Stormwater Management in a Changing Climate: Managing High Flow and High Water Levels in Wisconsin, WICCI Stormwater Working Group, June 2010

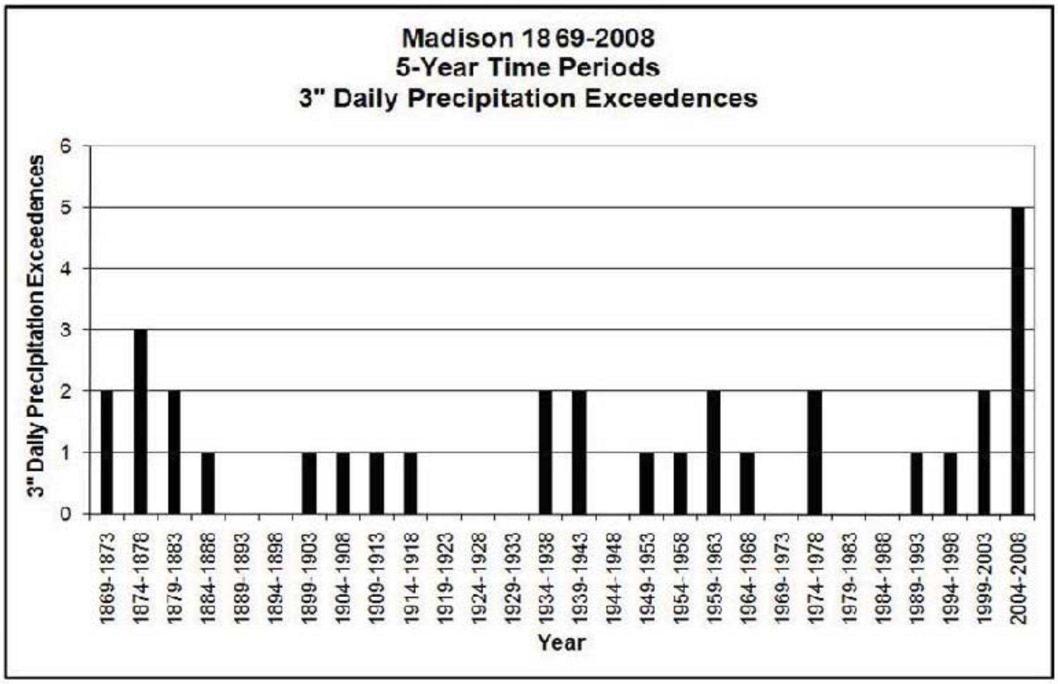


Figure 8.5 – 3" Daily Precipitation Exceedences in Madison (1869-2008)
Source: Stormwater Management in a Changing Climate: Managing High Flow and High Water Levels in Wisconsin, WICCI Stormwater Working Group, June 2010

WICCI has also attempted to project future rainfall amounts and frequencies from a number of Global Circulation Models (GCM). GCM's consider the increased emission of greenhouse gases in the atmosphere over time and their effect on global climatic patterns. The output from 15 of these models was used by WICCI to assess the effect of climate change on various hydrologic parameters in three distinct time periods: (1). 1961-2000; (2). 2046-2065; and (3). 2081-2100. The results of this analysis can be generally summarized as follows:

1. A modest increase in the magnitude of intense precipitation events can be expected during the next 90 years. The magnitude of the 100-year, 24-hour storm event is expected to increase by about 11% by the 2046-2065 time period.

2. Total precipitation and intense precipitation events are projected to increase significantly during the winter and spring months (December to April).

3. The amount of precipitation that occurs as rain from the months of December to March is projected to significantly increase.

Climate change is an emerging and constantly evolving topic. While its effects are not widely understood at this point and are not universally accepted by all members of the scientific community, local rainfall data does suggest that it has had an impact on rainfall volume and intensity during the last ten years. Even though an increase in rainfall volume and intensity may not be statistically significant for a long period of record in the Madison area, it would seem prudent to acknowledge these increases in the short term record when assessing the required capacity for both new and existing collection system facilities.

Peaking Factors

The required capacity of all intercepting sewers, force mains and pumping units in the collection system is determined by peak flowrates. As mentioned previously, the District uses the Madison Design Curve as a guide for calculating appropriate peaking factors for its facilities. This curve was developed by Greeley & Hansen Engineers in their "*Report on Sewerage & Sewage Treatment*" (1961) for the District and is represented by the following formulas:

$$\text{Peaking Factor} = 4/(Q_{avg})^{0.158} \qquad \text{(Q in mgd)}$$

or,

$$Q_{peak} = 4*(Q_{avg})^{0.842} \qquad \text{(Q in mgd)}$$

Note: 1. Peaking factor = 4.0 for $Q_{avg} \leq 1.0$ mgd
2. Peaking factor = 2.5 for $Q_{avg} \geq 20.0$ mgd

As discussed in the previous section, the Madison area has experienced a number of extreme rainfall events in the last twenty years, some of which have stressed portions of MMSD's collection system. District staff have analyzed data for several of the larger rainfalls and prepared reports for the following events:

- June 17, 1996
- August 18-19, 2007 (Two day rainfall total of 5.52 inches)
- June 7-8, 2008 (Two day rainfall total of 6.34 inches)

In response to the 2008 flow event the District prepared a memorandum to outline specific actions that the District would undertake to address the issue of high flows in its collection system. This memorandum and its updates can be found in Appendix 10. One of the action items from the memorandum was to review the District's use of the Madison Design Curve in sizing future conveyance facilities and in assessing capacity of the existing collection system. A file memo dated June 3, 2009 suggested that more conservative peaking factors may be needed in some areas of the collection system, such as increasing the peaking factor by 1.0 in wetter service areas.

In order to identify the most vulnerable service areas in the collection system, actual peaking factors for each of the aforementioned large rainfall events were calculated and compared to those predicted by the Madison Design Curve. As one might expect, some of the peaking factors for local pump station service areas fell below the value predicted by the Madison Design Curve and some were above the predicted value. Much of this variation can be attributed to the spatial variability and intensity that is associated with large rainfall events. For instance, the June 2008 event was particularly intense in the northwest portion of the District's collection system, leading to sewer overflows in the PS13 and PS14 service areas while other portions of the collection system were much less affected. After review of the data for each of the three large storm events, however, the following pump station service areas were identified as having peaking factors greater than those predicted by the Madison Design Curve for each event: PS2, PS6, PS7, PS8, PS11, PS12, PS13, and PS14. Most of these service areas also ranked high in estimation of infiltration volume as discussed in a preceding section of this chapter.

The peak flow in these wet service areas was increased by adding 1.0 to the Greeley & Hansen peak factor calculation to determine the additional length of gravity interceptors that would reach benchmark capacity by 2030. Table 8.6 compares the 2030 benchmark capacities for each service area with the conventional peaking factor developed by Greeley & Hansen and the revised peaking factor proposed in this section. As can be seen in Table 8.6, 25.3 miles of gravity interceptors will reach or exceed benchmark capacity by 2030 with utilization of the conventional peaking factor.

Applying the more conservative peaking factor to the wet service areas results in 41.0 miles of gravity interceptors reaching benchmark capacity by 2030, an increase of approximately 62%. Using an approximate unit replacement cost of $600 per foot for the additional 15.8 miles of sewer to be replaced, the cost to provide additional capacity in all of the wet service areas is estimated at $50 million over a 20-year period, or $2.5 million per year. This level of incremental funding for capital projects is not sustainable over the long term. Thus, revising the peaking factor to all wet service areas may not be the most efficient use of available funds for identifying and prioritizing capacity-related projects. A more systematic and detailed analysis of each "wet" service area identified in this facilities plan would likely generate the best results for the funds available.

It is interesting to note that a substantial portion (42%) of the additional interceptor length that would reach benchmark capacity with a revised peaking factor is in the PS14 service area. A more concentrated and thorough analysis of this service area, in particular, is recommended to verify existing rates of daily flow, to identify areas of suspected inflow and infiltration, and to determine excess capacity.

I/I Mitigation Strategies

The District outlined the following five steps to be taken following the June of 2008 storm to address the issue of high flows caused by excessive infiltration and inflow into the collection system (see Appendix 10):

Table 8.6
Gravity Interceptor Capacity Evaluation
Madison Metropolitan Sewerage District

Pumping Station Service Area	Total Gravity Intercveptor Mileage in Service Area (miles)	Mileage Predicted to Reach Benchmark Capacity By 2030				Additional Mileage Requiring Capacity Relief
		Greeley & Hansen Peaking Factor Gravity Interceptors		*Revised Peaking Factor* [(1)] Gravity Interceptors		
		(miles)	(%)	(miles)	(%)	(miles)
PS1	1.71	0.00	0%	0.00	0%	0.00
PS2	2.73	0.41	15%	0.41	15%	0.00
PS3	0.72	0.72	100%	0.72	100%	0.00
PS4	1.55	0.00	0%	0.00	0%	0.00
PS5	3.00	0.00	0%	0.00	0%	0.00
PS6	1.91	0.00	0%	0.02	1%	0.02
PS7	19.76	8.39	42%	10.68	54%	2.29
PS8	14.64	3.22	22%	6.94	47%	3.72
PS9	0.63	0.05	9%	0.06	10%	0.01
PS10	6.59	2.07	31%	2.07	31%	0.00
PS11	10.04	5.29	53%	6.25	62%	0.95
PS12	7.86	0.67	8%	2.85	36%	2.19
PS13	2.96	0.36	12%	0.36	12%	0.00
PS14	15.84	3.49	22%	10.11	64%	6.62
PS15	1.97	0.04	2%	0.04	2%	0.00
PS16	1.63	0.53	32%	0.53	32%	0.00
PS17	2.52	0.00	0%	0.00	0%	0.00
Totals	96.06	25.25	26%	41.04	43%	15.80

Notes:

(1). Revised peaking factor = Greeley & Hansen peaking factor + 1.0

(2). Revised peaking factor calculated only for service areas identified as "wet" basins (highlighted in grey).

1. Review design standards for sizing interceptor sewers and pump stations and adopt the use of higher peaking factors if deemed necessary and cost effective.

2. Review design standards for the materials used in the collection system to ensure that rainfall is less likely to leak into the system during heavy rains and floods.

3. Review flow data and inspect existing interceptor sewers to identify defects that allow excessive rainfall into the District's collection system.

4. Review flow data from the District's satellite communities that is collected during high flow events.

5. Increase public education efforts in the area of water conservation.

The remainder of this chapter summarizes the actions that the District has undertaken with regard to these steps since July of 2008.

Review of Peaking Factors

A previous section of this chapter discussed the District's current standards for sizing its conveyance facilities. This section also assessed the impact of adopting a higher peaking factor for facilities in low-lying or flood prone areas. In general it is cost prohibitive to adopt higher peaking factors for each existing facility in a "wet" area. The cost to rehabilitate or replace sewers in each of these areas would not be sustainable and may not be necessary in certain portions of the collection system. A more cost-effective approach would be to identify and prioritize those service areas with the highest susceptibility to I/I and perform detailed studies of each basin that include activities such as flow monitoring, television inspection, and smoke testing to locate specific I/I sources.

For new facilities it may be wise to consider more conservative peaking factors in the design, especially in areas of known I/I problems. An example where this approach might be applicable would be the design of PS18, which will receive flow from wet areas in the PS13 and PS14 basins.

Review of Design Standards for Construction Materials

Other than cross-connections with stormwater conveyance facilities or illicit discharges, the majority of I/I enters a collection system through manhole and pipe joints and manhole access covers. As demonstrated in Table 8.1, the pump station service areas with the greatest rates of I/I are those in which the pipes and manholes were installed prior to 1970. The majority of the pipes installed by the District prior to 1970 were of concrete and iron construction with poor sealing characteristics at joints and more joints per unit length than today's commonly used sewer materials.

The development and use of PVC and other flexible piping for sanitary sewers began in the 1970's and has led to significant improvements in the sealing of joints between pipes. Similar improvements have been made in the sealing of joints between manhole sections and in manhole access covers. The District installs chimney seals on all new manholes and on existing manholes prone to flooding to reduce the possibility of stormwater inflow through the access cover and adjusting rings. The District has also been proactive in replacing center-pull covers with sealed lids in areas prone to infiltration.

The use of improved materials of construction are reflected in the lower rates of I/I that are seen in the pump station service areas where these materials have been used in the last 30-40 years as part of new construction or rehabilitation projects (i.e. PS 1, PS15, PS16, and PS17). Going forward the District will continue to be proactive in replacing older access covers and installing chimney seals on existing manholes in flood prone areas and as part of road reconstruction projects.

Review of Flow Data and Television Inspection

The review of flow data by District staff for three of the largest storm events in the last fifteen years has been discussed in previous sections of this chapter. The analysis of this data has led to the identification of the following pump station service areas with peaking factors higher than predicted: PS2, PS6, PS7, PS8, PS11, PS12, PS13, and PS14. Some significant sources of inflow were identified by the District's Sewer Maintenance crew following the June 2008 high flow event. Several manholes in the PS12 service area were raised and flood-proofed upon inspection of the system after the storm event. The Sewer Maintenance crew will continue to inspect and rehabilitate those facilities that are susceptible to I/I as part of their routine inspection program.

Inspection of interceptor segments by closed-circuit television should be used as an additional tool to identify and prioritize I/I projects in suspected wet service areas. Table 5.2 in Chapter 5 should be used as a guide in this effort. Of the wet service areas previously identified, the District's television database has noted moderate infiltration in interceptors in the PS7, PS11, and PS14 basins.

Review of Flow Data from District Satellite Communities

The District maintains 61 pumping stations throughout its service area (44 are owned by satellite communities). As such, the District has access to flow data from these stations during high flow events and routinely analyzes the data to alert communities of areas of excessive I/I. The City of Madison, in particular, has used this information to identify and implement a number of I/I rehabilitation projects.

In recent years the District has been more proactive in communicating I/I problems to its satellite communities. After the June 2008 storm the District sent a memo to each community documenting the high flow conditions in the District's collection system and at the Nine Springs Wastewater Treatment Plant and the actions that the District would undertake to help mitigate the situation.

In August of 2009 the District took the additional step of analyzing flows from each of its satellite communities based on user charge (or flow monitoring) data and identifying those communities with higher than normal volumes of wastewater discharges. A letter was sent to each of the 17 communities that were identified as having high discharges, requesting that each community allocate funds to perform I/I studies and remediation efforts in their collection systems. Several communities have responded favorably to this request with documentation of their recent I/I investigations or future plans. The District will continue to monitor both average daily and peak flows from its satellite communities on a routine basis to identify problem areas and encourage rehabilitation programs. At this time the District has not directed any of its customers to take specific actions with regard to high flows.

Increase Public Education Efforts

The most effective strategy in reducing I/I in a collection system is to remove it at the source. To that end, in 2009 the District developed a series of radio advertisements which directed homeowners on efforts that could be undertaken to prevent rain water from entering their homes and ultimately the sanitary sewer system. The response to these advertisements has been favorable and more work in this area appears warranted and needed to help reduce I/I at the source level.

One specific area in which public education initiatives could be undertaken is with regard to rehabilitation of private sanitary sewer laterals. These private laterals contribute significant amounts of I/I into the sanitary sewer collection system and their condition is rarely inspected by the property owner. Lateral rehabilitation or replacement is usually only initiated by the owner due to root intrusion or damaged pipe that cause line blockages. With advances in sewer lining technology, the opportunity exists for the District to work with satellite communities on developing programs that encourage and offer incentives to property owners to inspect, maintain, and repair their sewer laterals.

Chapter 9
Recommended Projects & Initiatives

Based on the results and considerations presented in the preceding chapters, Table 9.1 is a summary of projects recommended for the MMSD collection system. The projects are organized by pumping station drainage basin. The driving needs for the individual projects (hydraulic capacity, physical condition, or both) are noted in the table and are discussed in the preceding chapters. The locations of the projects are highlighted in Figure 9.1 (see large map enclosed in the attached pocket).

Individual project costs shown in Table 9.1 are preliminary and may be subject to significant change as individual projects are examined in detail and refined in scope. All preliminary estimates shown in Table 9.1 are in terms of Year 2010 dollars.

The projects in Table 9.1 are organized into four time periods based on consideration of priority and needs (Period A: 2010-2015, Period B: 2016-2020, Period C: 2021-2030, and Period D: beyond 2030). An additional category, entitled 'Uncertain', has been included as a separate category for complex projects that do not fit into a specific time period based on capacity or condition but may be required as conditions and needs within the collection system evolve over time. Due to the long-range timeframe of Table 9.1, it is likely that the scope and priority of various projects will change as detailed studies are performed and as future developments occur. Table 9.1 and Figure 9.1 should be reviewed and updated annually to maintain a current picture of MMSD's collection system needs and to track the completion of major projects.

Funding for the projects will be provided from reserves and through general obligation debt placements. It is estimated that MMSD will borrow an average of $7 million each year over the next twenty years to fund these projects. Debt service on these borrowed funds will be recovered through MMSD's service charges. It is assumed that all borrowing will be made from the Wisconsin Clean Water Fund Program administered by the Wisconsin Department of Natural Resources. Terms of the loans are assumed to include interest at 4% and a twenty-year repayment period. The average household in MMSD could expect their annual service charge to increase by $4 each year for the next twenty years to fund these projects. This increase will be in addition to increases associated with inflation in wages, materials, energy, and services that will impact MMSD's operating budget from year to year.

In addition to the projects listed in Table 9.1, a number of other initiatives and recommended improvements have been discussed in this facilities plan to enhance the management, maintenance and operation of the District's collection system. These initiatives relate primarily to asset management and CMOM principles. A list of these major initiatives is shown in Table 9.2.

Table 9.1
MMSD Collection System Projects
Approximate Timetable and Costs

Project	Project Completed?	Cost Estimate (Year 2010 dollars) Period A 2010-2015	Period B 2016 - 2020	Period C 2021-2030	Period D Beyond 2030	Uncertain	Primary Need: Hydraulic Capacity	Physical Condition	Comments
System Wide Projects									
Telemetry System - Third Updgrade			$ 150,000						Upgrade with Process Control System.
Influent Storage and Equalization			$ 9,000,000				x		Influent storage required after PS 18 is built.
Update and Maintain CSFP									Ongoing process.
Interceptor Rehabilitation (Lining)			$ 2,500,000	$ 5,000,000				x	Allowance for annual lining of interceptors.
Pumping Station No. 1 Service Area									
Northend Interceptor Lining		$ 100,000						x	1,482' - 10" & 12" VP ~85 years old.
Pumping Station No. 2 Service Area									
Southwest Int - Haywood Replacement				$ 1,200,000			x	x	1,500' - 36" provides additional capacity for PS2-PS8 diversion.
Old West Int Lining (MH02-014A to MH02-101)		$ 660,000						x	5,000' - 24". 1916 cast iron sewer with mineral deposits.
WI - Spring Street Relief Lining		$ 600,000						x	4,580' - 24". 1940 cast iron sewer with mineral deposits.
Pumping Station No. 3 Service Area									
Rimrock Interceptor Replacement			$ 550,000				x	x	3,800' - 12" RCP (1959).
PS No. 3 Rehabilitation			$ 600,000					x	PS 3 ~50 years old (1958).
Pumping Station No. 4 Service Area									
South Interceptor - Baird Street Lining	x	$ 100,000						x	1,500' - VCP (1928). Lined with CIPP in 2010.
PS No. 4 Rehabilitation			$ 1,300,000					x	PS 4 ~45 years old (1967).
Pumping Station No. 5 Service Area									
West Int. Rehabilitation U/S of PS5		$ 300,000						x	3,600' - CIP (1931). PS5 to Gammon Ext. junction.
Pumping Station No. 6 Service Area									
East Monona Interceptor Lining		$ 120,000						x	Cracked pipe sections.
PS No. 6 Rehabilitation	x	$ 3,300,000						x	Four new pumps and related electrical and control work.
Gravity Tie from PS 6 to PS 10						$ 4,500,000	x		Intertie for diversion flexibility and system redundancy.

Project	Project Completed?	Cost Estimate (Year 2010 dollars) Period A 2010-2015	Period B 2016 - 2020	Period C 2021-2030	Period D Beyond 2030	Uncertain	Primary Need: Hydraulic Capacity	Physical Condition	Comments
Pumping Station No. 7 Service Area									
NEI Relief from FEI junction to PS18		$ 7,400,000					x		5,600' - 48" needs relief. To be completed with PS18 project.
NEI Lining from FEI junction to SEI junction			$ 1,300,000					x	Rehabilitate existing 48" NEI after relief sewer is constructed.
FEI - Cottage Grove Extension Lining	x	$ 190,000						x	5,500' - 18" rehabilitated with CIPP liner.
Far East Interceptor				$ 2,200,000	$ 6,900,000		x		3,900' - 30" may require relief in Period C.
PS 18 (to provide relief for PS 7)		$ 10,500,000					x		For future growth and reliability.
PS 18 Force Main		$ 11,600,000					x		From new PS 18 to NSWWTP.
Pumping Station 7 Improvements			$ 1,800,000					x	Construct after PS18 is operational.
FEI - Door Creek Extension					$ 8,800,000		x		High growth may require relief in Period C.
SEI - Blooming Grove Extension				$ 4,500,000	$ 1,200,000		x		High growth may require relief in Period B for segments east of I-90.
SEI - Dutch Mill Extension					$ 1,100,000		x		
Southeast Int (MH07-215 to MH07-218)					$ 1,100,000		x		
Pumping Station No. 8 Service Area									
West Int Lining on Old University Ave	x	$ 300,000						x	3,400-ft of 18"-21" from Farley to Forest (1916).
PS No. 8 Rehabilitation	x	$ 3,300,000						x	Four rebuilt pumps and related electrical work.
West Int Relief - Additional Capacity			$ 12,000,000				x		~12,000 ft of relief sewer from Whitney Way to Walnut St.
West Int/Midvale Relief - Additional Capacity				$ 900,000			x		2,600' - 21" may need relief. Could be provided with WI Relief project.
West Int/Randall Relief					$ 60,000		x		
Southwest Interceptor					$ 1,500,000		x		
Pumping Station No. 9 Service Area									
PS No. 9 Rehabilitation			$ 600,000					x	PS 9 ~50 years old (1961).
Southeast Interceptor					$ 1,800,000		x		
Pumping Station No. 10 Service Area									
NEI - Relief Upstream of PS No. 10	x	$ 8,700,000					x	x	9,200' of relief sewer from Lien Road to PS 10.
NEI - Truax Extension Replacement/Relief				$ 9,800,000			x		11,000-ft. of relief sewer from PS 13 FM to Lien Rd.
I/I Study						$ 150,000	x		Recommendation from 2002 CSFP.
PS10 Forcemain Relief					$ 6,400,000		x		PS6-PS10 gravity connection for system redundancy.
Pumping Station No. 11 Service Area									
PS No. 11 Rehabilitation		$ 3,700,000					x	x	PS 11 ~50 years old. Major electrical upgrades required.
NSVI Relief Projects			$ -	$ 11,700,000	$ 16,100,000		x		Assumes relief sewer of similar size to existing.
PS 11 Forcemain Relief				$ 1,900,000			x		4,200'-36" relief FM from PS 11 to NSWWTP.

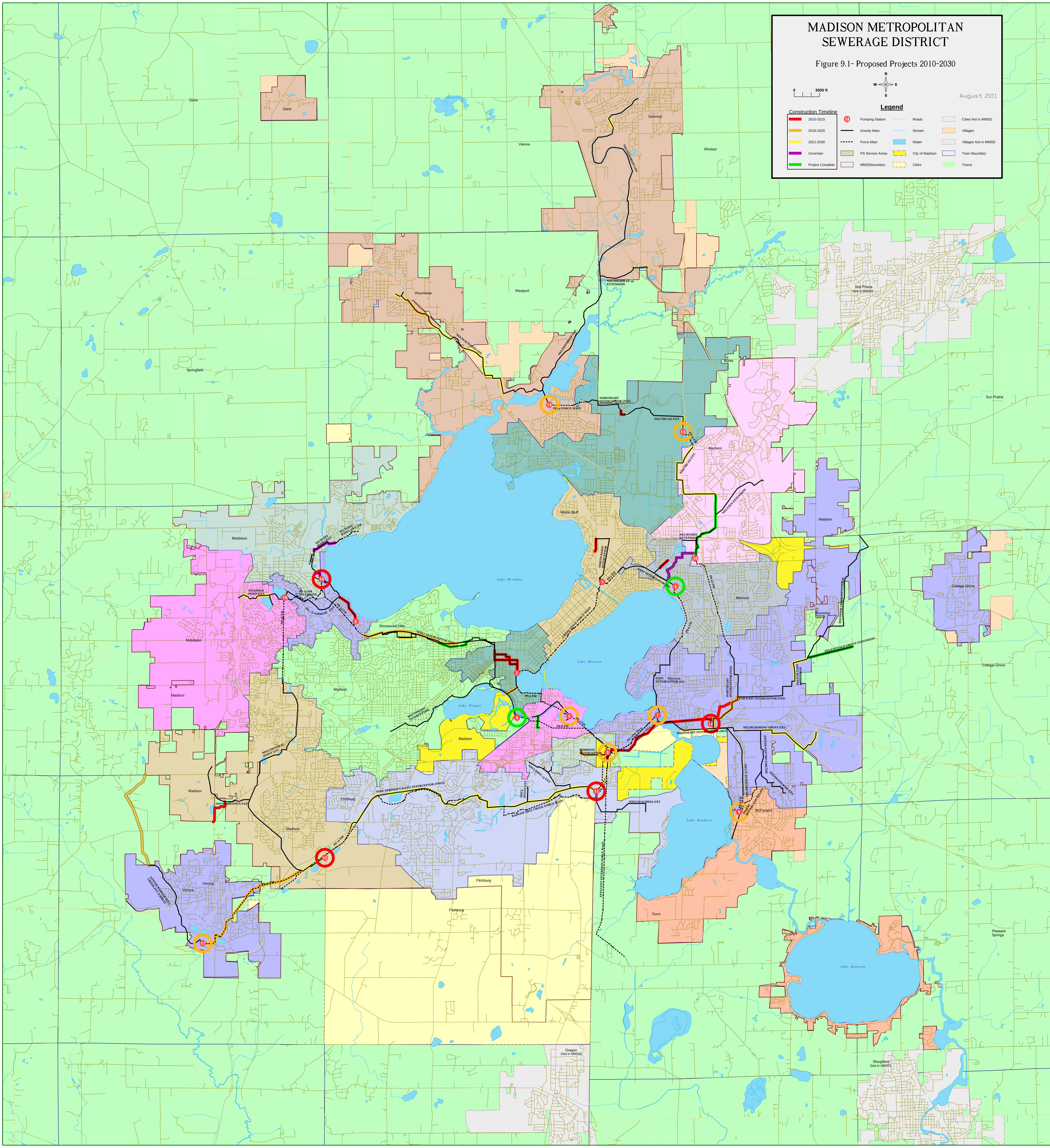
Project	Project Completed?	Cost Estimate (Year 2010 dollars) Period A 2010-2015	Period B 2016 - 2020	Period C 2021-2030	Period D Beyond 2030	Uncertain	Primary Need: Hydraulic Capacity	Primary Need: Physical Condition	Comments
Pumping Station No. 12 Service Area									
NSVI - Morse Pond Extension		$ 740,000					x		3,500-ft of new sewer to serve future development.
PS No. 12 Rehabilitation		$ 3,700,000						x	PS 12 ~40 years old. Major electrical upgrades required.
NSVI Relief Projects			$ -	$ 3,200,000	$ -		x		~3,500'-48" sewer U/S of PS 12 needs relief.
Pumping Station No. 13 Service Area									
PS No. 13 Rehabilitation			$ 3,400,000					x	PS13 ~40 years old and requires electrical upgrades.
NEI - Rehabilitation West of Airport		$ 600,000						x	Corrosion from MH13-116H to MH13-125 (~1,250').
NEI - PS 14 to PS 13				$ 1,800,000	$ 4,200,000		x		
Sanitarium Sewer						$ 1,890,000			Divert flows from PS13 service area to PS1.
Pumping Station No. 14 Service Area									
PS No. 14 Rehabilitation			$ 3,400,000					x	PS14 ~40 years old and requires electrical upgrades.
NEI - Waunakee Extension Relief			$ 2,500,000	$ 6,300,000	$ 4,100,000		x		
NEI - DeForest Extension Relief				$ 400,000	$ 16,600,000		x		
I/I Study		$ 150,000					x		Recommendation from 2002 CSFP.
Pumping Station No. 15 Service Area									
PS No. 15 Rehabilitation		$ 2,400,000	$ 1,300,000					x	PS 15 ~35 years old and needs firm capacity relief.
West Int Extension - Siphon Replacement						$ 500,000	x		Improve maintenance for siphon at Pheasant Branch.
West Int Extension					$ 100,000				
Pumping Station No. 16 Service Area									
West Interceptor - Gammon Extension Relief				$ 700,000			x		2,800' of 18" & 24" on Voss Parkway needs relief.
Pumping Station No. 17 Service Area									
Lower Badger Mill Creek Interceptor - Phase III			$ 2,800,000	$ 1,500,000					New sewer from Cross Country Rd to Midtown Rd.
PS No. 17 Rehabilitation			$ 1,900,000				x	x	To be completed with LBMC Interceptor (Phase III).
PS No. 17 Force Main			$ 2,600,000				x		To be completed with LBMC Interceptor (Phase III).
Total Projects		$ 58,460,000	$ 47,700,000	$ 51,100,000	$ 69,960,000	$ 7,040,000			
Total Projects (less completed projects as of 7/11)		$ 42,570,000	-	-	-	-			

Table 9.2 – MMSD Collection System Initiatives

Collection System Area	Initiative	Related Information
Capacity evaluation	Consider purchase of flow metering equipment for I/I studies.	Chapter 2
Design	Consider development and adoption of revised peaking factors for service areas subject to excessive inflow and infiltration.	Chapter 8
	Continue to evaluate alternate designs for air release valves on forcemains and eliminate these valves where possible to avoid SSO's due to valve plugging.	Chapter 7
Management	Continue improvements in asset registry and system of assessing condition of assets. Develop systematic procedures to capture all information relating to collection system assets and improved methods of storing and tracking this data.	Chapter 2
	Improve methods to estimate remaining asset life, life cycle costs, and replacement costs. Particular emphasis should be placed on methods for pipes rehabilitated with new technologies such as cured-in-place liners.	Chapter 2
	Assess and determine expected level of service to be provided based upon customers and regulators expectations.	Chapter 2
	Develop a risk-based condition assessment tool to aid in prioritizing maintenance, repair, renewal, and replacement projects.	Chapter 2
	Optimize and improve upon the District's maintenance program, repair and renewal methods, and capital improvement planning methods. Develop and formalize written methods for project justification as part of budgeting process.	Chapter 2
	Monitor funding strategies for District's asset management program.	Chapter 2
	Develop written procedures for sanitary sewer overflow events, including procedures for identification and clean-up of overflows and notification requirements.	Chapter 2
Operation	Develop written rules and procedures for wastewater monitoring program.	Chapter 2
	Develop a systematic program to assess hydrogen sulfide concentrations in susceptible areas of the collection system.	Chapter 2
	Develop a written safety program relating specifically to collection system work areas.	Chapter 2

Maintenance	Provide enhancements to District's televising program. Enhancements to include staff training for pipeline inspection, improvements to the scoring and ranking system, and improved tracking of cleaning and televising frequency throughout the collection system.	Chapter 5

Overall, the District believes that its collection system is operated in a cost-efficient manner and provides a high level of service to its customers. The initiatives described in Table 9.2 are steps that have been identified to: (1). Optimize the use of available funds in operation and maintenance of its collection system; (2). Assist the District in meeting future regulations regarding sanitary sewer overflows; and (3). Adopt advanced asset management principles to help manage and operate the District's expanding collection system assets.


MADISON METROPOLITAN
SEWERAGE DISTRICT
Figure 9.1- Proposed Projects 2010-2030
N
W
E
S
0
3000 ft
August 2011
Legend
Construction Timeline
2010-2015
2016-2020
2021-2030
Uncertain
Project Complete
Pumping Station
Gravity Main
Force Main
PS Service Areas
MMSDboundary
Roads
Stream
Water
City of Madison
Cities
Cities Not in MMSD
Villages
Villages Not in MMSD
Town Boundary
Towns
Dane
Vienna
Deforest
Windsor
Waunakee
Westport
Sun Prairie
Springfield
Burke
Madison
Maple Bluff
Middleton
Lake Mendota
Shorewood Hills
Lake Monona
Lake Wingra
Blooming Grove
Cottage Grove
Monona
Fitchburg
Verona
McFarland
Lake Waubesa
Dunn
Lake Kegonsa
Pleasant Springs
Oregon
(Not in MMSD)
Stoughton
(Not in MMSD)
NORTHEAST INTERCEPTOR (NEI)
NEI/TRUAX EXT.
FAR EAST INTERCEPTOR (FEI)
SEI/BLOOMING GROVE EXT
NINE SPRINGS VALLEY INTERCEPTOR (NSVI)
BADGER MILL CREEK FORCE MAIN
NSVI/MIDTOWN EXT
NSVI/NAUBESA EXT
EFFLUENT DIVERSION FORCE MAIN

Appendix A1
MMSD Collection System Evaluation (January 2009)

Appendix A1 contains excerpts from the *Madison Metropolitan Sewerage District Collection System Evaluation (January 2009)*, prepared by the Capital Area Regional Planning Commission (CARPC) in collaboration with MMSD. These excerpts serve as background for the methodologies employed to generate the population and wastewater flow forecasts that are used throughout this update to *MMSD's Collection System Facilities Plan*. CARPC's complete document is on file at MMSD's Administrative offices.

MMSD Collection System Evaluation

January 2009

Madison Metropolitan Sewerage District Collection System Evaluation 2008

Prepared by the staff of the Capital Area Regional Planning Commision
in collaboration with the staff of the Madison Metropolitan Sewerage District

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Chapter 1 Introduction and Background

Background and Overview

The Madison Metropolitan Sewerage District (MMSD) was formed in 1930 to provide area-wide wastewater collection and treatment for the communities around Lakes Mendota and Monona. The District initially served a 50 square mile area including Madison, Monona, Maple Bluff, Shorewood Hills, and surrounding towns. By 2007, the District's service area had grown to 178 square miles, including all of the communities that formerly discharged treated wastewater to the Yahara River lakes. A map of MMSD's service area is shown in Figure 1-1.

All of the wastewater generated in the MMSD service area is collected and transmitted to the Nine Springs Wastewater Treatment Plant. Most of the treated effluent is discharged to Badfish Creek to divert treated wastewater around the Yahara River lakes. Some treated effluent is returned to Badger Mill Creek to offset the effects of inter-basin transfer on the base flow of Badger Mill Creek. The Badger Mill Creek outfall has a design capacity of 3.6 million gallons per day (mgd).

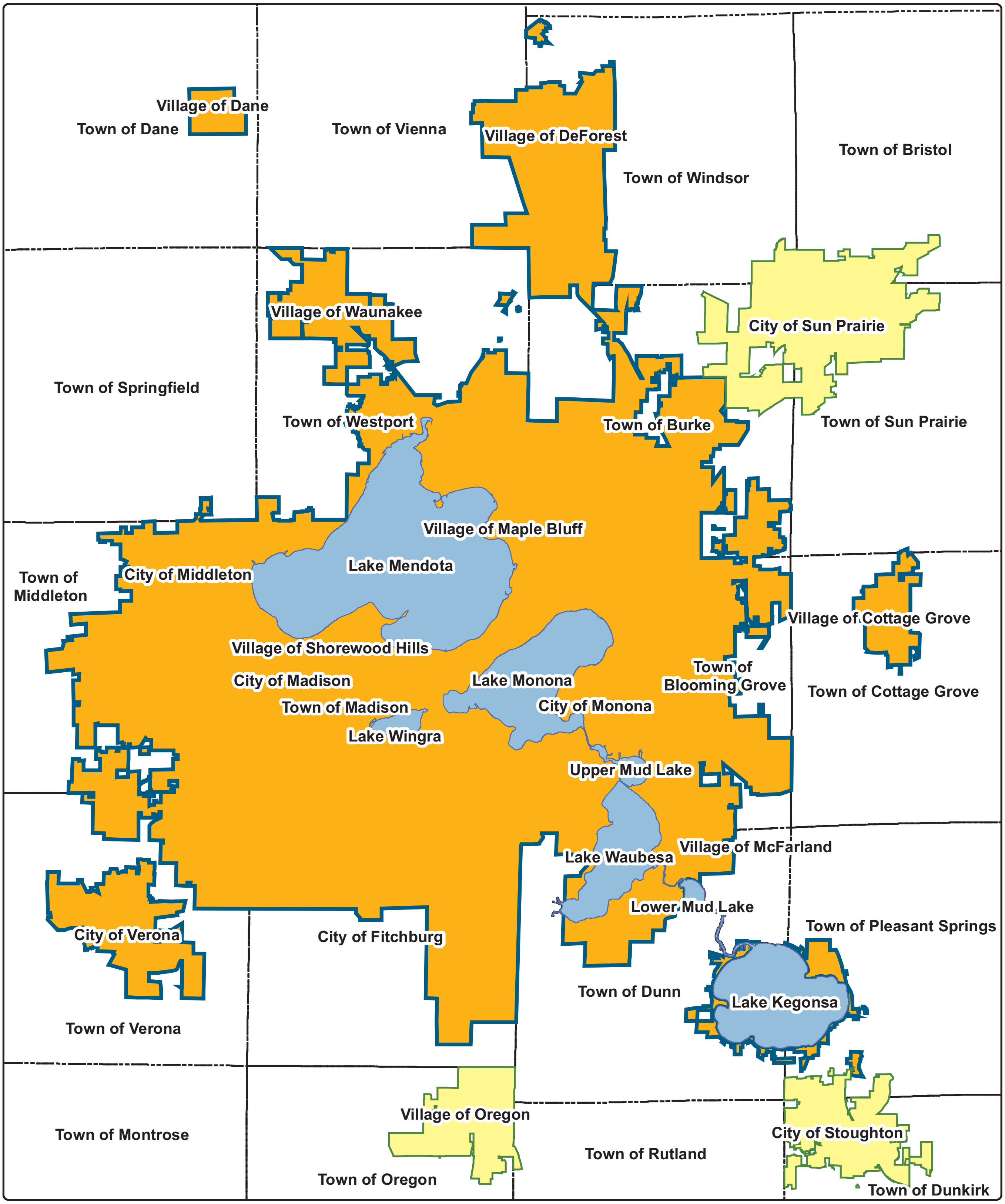
In 2007, the District's collection system included approximately 94.5 miles of gravity sewer, 29.3 miles of force main, and 17 major pumping stations. This collection system receives wastewater from the community sanitary sewer systems, and transmits the wastewater to the Nine Springs plant for treatment.

Previous Studies

Parts of the MMSD collection system date back to before 1900, and there have been numerous design studies of various sections or elements of the system over the years. The most significant system design studies and plans since 1960 are listed and described in Appendix A. These include:

- "Report on Sewerage and Sewage Treatment", 1961, Greeley and Hanson Engineers
- "Review of Project VII; West Side Collecting System", 1967, Mead & Hunt
- "Review of Project IV; Northeast Collecting System", 1969, Mead & Hunt
- "Report on Northeast Interceptor, Token Creek Extension", 1971, Mead & Hunt
- "Report on Sewage Treatment; Additions to the Nine Springs Sewage Treatment Works, 1971, Greeley and Hanson Engineers
- "Planning Report on the Fifth Addition to the Nine Springs Sewage Treatment Works", 1973, Dane County Regional Planning Commission (DCRPC)

1/06/2008



LEGEND

Outside District

Inside District

MMSD SERVICE AREA

MADISON METROPOLITAN SEWERAGE DISTRICT
COLLECTION SYSTEM EVALUATION
2008

FIGURE 1-1

In 1976, MMSD completed a major, comprehensive, facilities plan for the overall wastewater management needs for the entire district. As part of that facilities planning effort, the DCRPC and MMSD developed flow forecasts, evaluated the collection system, and considered regionalization or interconnection possibilities.

Several facilities plans, design studies, and reports concerning specific improvements and interceptor extensions were conducted between 1976 and 1986. These studies are summarized in Appendix A. They include design studies for:

- The Esser Pond Interceptor (1978)
- The Cottage Grove Extension of the Far East Interceptor (1978)
- The Mendota Extension of the Nine Springs Valley Interceptor (1979)
- The City of Middleton Sewer Plan (1982)
- The Facilities Plan for the Dunn-Kegonsa Sanitary District (1985)
- The Facilities Plan for the Town of Pleasant Springs portion of the Kegonsa service area (1986)
- The Design Study for the McFarland Relief Sewer (1986)

A comprehensive four-year study of the MMSD collection system was completed in 1993 with the publication of a report titled “MMSD Collection System Evaluation”. The study, a collaboration between the DCRPC and MMSD, utilized socioeconomic data generated by the DCRPC for transportation planning, to forecast flows for small geographic areas (sub-basins).

Several additional design studies and reports concerning specific improvements and interceptor extensions were conducted between 1993 and 1998. These studies are also summarized in Appendix A. They include design studies for:

- The City of Verona connection to MMSD (1993)
- The Badger Mill Creek effluent return project (1993)
- The Morrisonville Urban Service Area connection to MMSD (1995)
- The Lien Interceptor Extension (1995)
- The Village of Dane connection to MMSD (1997)
- The Far East Interceptor – Door Creek Extension (1997)

In 1999, the DCRPC and MMSD collaborated on an update to the 1993 collection system evaluation. The update also utilized socioeconomic data generated by the DCRPC for transportation planning, to forecast flows for small geographic areas (sub-basins).

Since 1999, there have been additional design studies and reports addressing specific improvements. These studies are summarized in Appendix A. They include:

- Summary Design Memo West Interceptor Replacement at UW Campus (1999)
- Collection System Facilities Plan (2002)
- The Lower Badger Mill Creek Sewer Service Report (2005)
- Predesign Memo for West Interceptor Extension (2006)
- Design Memo for Southwest Interceptor North & South Legs Rehabilitation (2006)
- Design Report for Rehabilitation of Pumping Stations No. 6 and 8 (2007)
- Final Design Report Pump Station 13 and 14 Firm Capacity Improvements (2007)

- Northeast Interceptor – PS10 to Lien Road Relief / Replacement Planning Report (2008)
- Northeast Interceptor Truax Liner Engineering Design Report (2008)

Purpose and Approach to the Evaluation

The basic purpose of this collection system evaluation is to update the 1999 collection system evaluation, in order to anticipate future capacity problems and identify needs for the expansion or improvement of sections of the MMSD collection system. This evaluation follows a similar approach to the 1999 evaluation. The approach to the evaluation includes the following steps:

1. Pumping station service areas and sub-basin boundaries are updated based on additions and changes to the community sanitary sewer systems.
2. Historic wastewater flows and flow distributions throughout the system are analyzed.
3. Characteristics and capacities of elements of the collection system (pumping stations, force mains, and interceptor sewers) are determined.
4. Future wastewater flows are forecast, and estimated for specific sections and elements of the collection system. These forecasts are developed from, and are consistent with, population, land use, and socioeconomic forecasts in adopted plans, as required by state statutes and administrative rules governing MMSD operations and facilities planning. Baseline and future flows are allocated to sub-areas (pumping station service areas and sub-basins) served by individual pumping stations or interceptor sewer sections.
5. The capacities of specific facilities are compared with baseline and future estimated wastewater flows to determine where there could be future capacity problems, and to assess the need for expansion or improvements to the collection system.
6. The evaluation includes the determination of long-term (2060) growth and development potential and flow forecasts, in order to provide guidance in selecting design flows and capacities for facility improvements.

The function of this report is to allow MMSD to adequately plan its collection system improvements to ensure pollution control into the future. This necessitates a conservative, yet reasonable, approach to estimating future development levels and wastewater generation rates. The identification of any area as a potential future growth area in this report is not intended to predict or promote growth in these areas, nor is it intended to be an indication of the likelihood that any specific area will be approved as an expansion of the urban service area in the future.

This collection system evaluation reflects the input and contribution from the staffs of both the Madison Metropolitan Sewerage District and the Capital Area Regional Planning Commission (CARPC). MMSD staff was primarily responsible for providing technical data and information regarding historic flows and distribution, characteristics and capacities of collection system components and evaluation of the results and implications of the evaluation. CARPC staff was primarily responsible for socioeconomic data and forecasts, development of future flow forecasts, allocation of flows into pumping station service areas and sub-basins, and developing long-range forecasts of flows and service areas.

Chapter 2 Plans and Socioeconomic Forecasts

Plan Consistency Requirements

The collection system evaluation is based on and consistent with adopted local and regional plans in order to satisfy the requirements in state statutes and administrative rules for plan consistency. The purpose of the plan consistency requirement is to ensure that decisions regarding sewerage are coordinated and consistent with other related planning decisions made by other agencies or units of government. The intent is to avoid conflict between plans and decisions of different agencies and units of government, and to coordinate the pursuit of common regional land use and development objectives. These consistency requirements are particularly important in the case of sanitary sewer systems, since the location and extension of sanitary sewers is often a major factor in the location of urban development. Coordinated and consistent planning allows the provision and extension of sanitary sewer service in a cost effective and efficient manner. Conversely, planned control over the timing and extension of sanitary sewer service is an important technique in guiding urban development.

State administrative rules governing water quality planning and wastewater facilities planning generally require that facilities planning, funding, and regulatory decisions be consistent with approved area-wide water quality management plans. The state also requires that all sanitary sewer extensions be consistent with the sewer service areas delineated in area-wide water quality management plans in designated areas, including Dane County. In addition to state water quality planning consistency requirements, state statutes governing metropolitan sewerage districts (Chapter 200, Wisconsin Statutes) require that plans of metropolitan sewerage districts be consistent with adopted regional plans.

Land Use Plans

The *Vision 2020 Dane County Land Use and Transportation Plan* is the overall comprehensive land use and development policy framework and guide for Dane County. Dane County and the Dane County Regional Planning Commission adopted this plan in 1997. The *Dane County Water Quality Plan*, the official area-wide water quality management plan for Dane County, is based on and incorporates the land use and transportation plan as the basic regional land use framework for the water quality plan. The water quality plan outlines the planned sewer service areas throughout Dane County, which reflect the urban service areas and limited service areas outlined in the land use and transportation plan. These plans also reflect the delineation of environmental corridors or environmentally sensitive areas that are to be protected from the impacts of urban development. Sections of the water quality plan are revised and updated on a periodic basis.

In 1999, the Wisconsin Legislature passed comprehensive planning legislation (§66.1001, Wisconsin Statutes) often referred to as the “Smart Growth” law. The law requires all Wisconsin communities that exercise land use authority to adopt a comprehensive plan by ordinance by 2010, and for land use decisions to be consistent with the adopted plan. Comprehensive plans are to serve as a guide for the future development and redevelopment of the local governmental unit over a 20-year planning period. Local comprehensive plans, as well as neighborhood development plans, provided information on the amount and location of future development in a community. The comprehensive plans of the following communities were reviewed as part of this study:

- City of Fitchburg (draft, October 2008)
- City of Madison (adopted, January 2006)
- City of Middleton (adopted, November 2006)
- City of Monona (adopted, April 2004)
- City of Verona (draft, April 2008)
- Village of Maple Bluff (draft, November 2002)
- Village of Cottage Grove (amended, July 2008)
- Village of DeForest (amended, April 2008)
- Village of McFarland (adopted, March 2006)
- Village of Waunakee (adopted, June 2003)
- Town of Vienna (adopted, June 2006)
- Town of Westport (adopted, March 2004)
- Town of Windsor (adopted, September 2006)

Socioeconomic Forecasts

Urban Service Area Data

Forecasts of future population and basic socioeconomic data are used to anticipate future growth and infrastructure needs. There are currently seven urban service areas (USA) and five limited service areas (LSA) within the MMSD service area. Urban service areas are those areas in and around existing communities that are most suitable for urban development and capable of being provided with a full range of urban services. Urban services are the public services normally provided or needed in urban areas, including public water supply and distribution systems, sanitary sewerage systems, higher levels of police and fire protection, solid waste collection, urban storm drainage systems, streets with curbs and gutters, street lighting, neighborhood facilities such as parks and schools, and urban transportation facilities such as sidewalks, taxi service and mass transit. Limited service areas are areas where only a few urban services, such as sanitary sewer service, are intended to be provided to special or unique areas (remote correctional facilities, sanitary landfills, etc.) or to areas of existing development experiencing sewage disposal problems. These areas are not intended to receive a full range of urban services or additional urban development.

Table 2-1 illustrates historic and forecasted population for the MMSD service area. Population forecasts for urban and limited service areas in 2030 are developed by the CARPC by allocating countywide population forecasts, developed by the Demographic Services Center of the Wisconsin Department of Administration (DOA), to smaller areas. These official population forecasts are required for use for facilities planning purposes. The CARPC population forecasts are based on the DOA countywide population forecasts prepared in January 2004 from 2000 US Census data. The DOA population forecasts project population 30 years into the future at the county level, and 25 years into the future at the municipal level. Population forecasts for Dane County and for each of the urban service areas are expected to be updated in 2014 from 2010 US Census data. The 2060 forecasts were developed from a least squares linear regression and are not official forecasts. The official 2060 population forecasts will not be developed until 2030.

Table 2-1: Population Trends and Forecasts for the MMSD

	1980	1990	2000	2030	*2060*
Central USA	218,344	245,390	268,850	339,222	*404,204*
Cottage Grove USA	901	1,131	4,059	9,372	*11,798*
Dane USA			799	1,351	*1,594*
Fox Bluff LSA			240	240	*240*
Kegonsa LSA			2,228	2,252	*2,252*
Morrisonville USA			352	428	*464*
Northern USA	5,393	7,160	9,901	16,883	*23,825*
Verona USA			7,306	15,685	*20,178*
Waubesa LSA			2,027	2,027	*2,027*
Waunakee USA	3,890	5,899	9,000	17,458	*23,367*
Windsor Prairie LSA			509	509	*509*
Westport LSA			377	377	*377*
MMSD	228,528	259,580	305,648	405,804	*490,835*

Historic and forecasted population figures for three urban service areas that are outside, but nearby, the current MMSD service area are shown in Table 2-2.

Table 2-2: Population Trends and Forecasts for Other USAs

	1980	1990	2000	2030	*2060*
Oregon USA	3,927	4,528	7,514	13,106	*17,275*
Stoughton USA	8,256	9,265	12,671	18,609	*23,064*
Sun Prairie USA	13,306	15,481	20,533	36,211	*45,188*

Traffic Analysis Zone Data

In addition to population forecasts at the urban service area level, socioeconomic data is available in smaller analysis units called traffic analysis zones (TAZ). The Madison Area Transportation Planning Board (MATPB) developed the most recent TAZ data in 2000 for transportation planning. This data divides Dane County into over 1,000 analysis zones, which range in size from 3.7 acres in the central urban area, to over 6,000 acres in rural areas. The socioeconomic data associated with each zone includes population, number of households, and total employment for the year 2000 as well as forecasts for the year 2030.

TAZ Data Sources

The TAZ allocation of year 2000 population and household data is based on US Census data and Census block boundaries. The MATPB developed the TAZ 2030 population and household data by allocating the DOA/CARPC population forecasts to TAZ regions based on community comprehensive plans and neighborhood development plans. They noted in their *Regional Transportation Plan 2030*, that the allocation of forecasted 2030 growth is far less than a build-out scenario of the planned growth identified in local plans.

The year 2000 employment data is generated from CLARITAS, which did a phone book survey of places of business in Dane County in 1999. The MATPB adjusted the CLARITAS data, in some cases, to be consistent with Census employment data. The data was geocoded to allocate it to the TAZ regions. The DCRPC developed a 2030 employment forecast based upon a labor supply forecast using the DOA 2030 population forecast by age group. The MATPB allocated the 2030 employment forecast to TAZ regions using the 2000 ratio of population to employment, the location of planned employment centers, and the change in the population/employment ratio for each urban service area from 1990 to 2000.

TAZ Data Adjustments

The geographic area associated with the TAZ data does not always coincide with the pumping station sub-basin areas. In these cases, the TAZ region was divided to correspond with the pumping station sub-basin areas and the TAZ data was allocated between the resulting areas.

Allocation of the year 2000 TAZ data is based on a review and analysis of other available data sources including the 2000 Census data, 2000 land use inventory, geocoding of the 1999 CLARITAS data, 2000 aerial photography, and municipal property information. TAZ household data includes households on septic systems. The 2000 TAZ household count are adjusted, where necessary, to remove households on septic systems.

Allocation of the 2030 household and employment forecasts is based on a review and analysis of 2005 aerial photography and current parcel data to identify areas that have been developed since 2000. Areas available for development were also identified. After accounting for development that has already occurred, 2030 household and employment forecasts were generally allocated based on the proportion of developable area remaining.

Comprehensive Plan Data

The TAZ data was developed in 2000. Most communities have completed their comprehensive plans since 2000, or are currently working on them. Thus, the TAZ data does not always reflect current development plans, which results in uncertainty with the accuracy of the data. Comprehensive plans and neighborhood development plans usually contain data on the amount of household and sometimes employment growth associated with new development. Municipal development plans were reviewed and summarized to develop another forecast of 2030 household and employment. In addition to reviewing the comprehensive plans, meetings were held with each community to discuss their comprehensive plan projections and to get their forecasts for long term growth through 2060.

Table 2-3: Comprehensive Plan Growth Projections

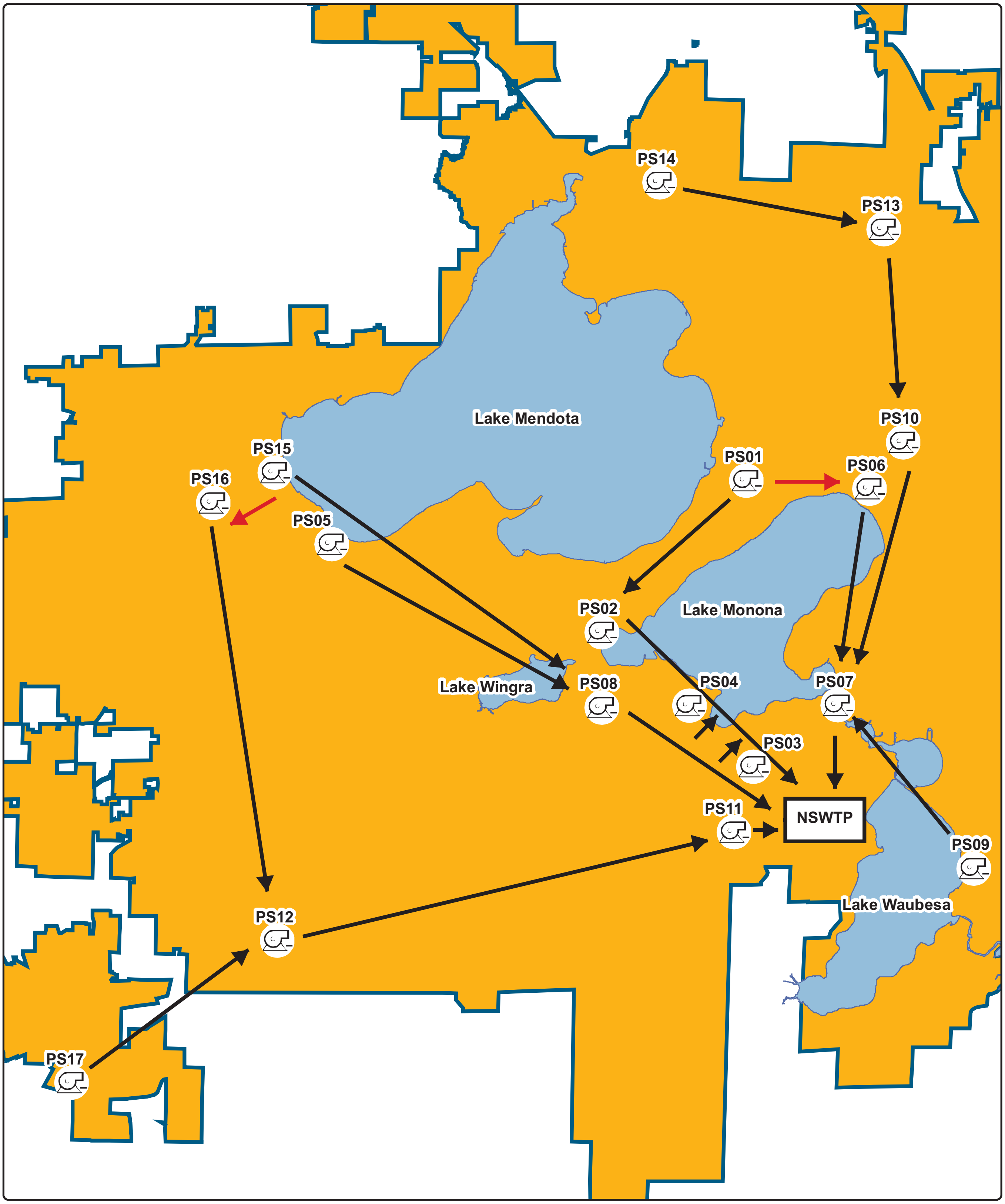
Municipality	2000		Comprehensive Plan Projection		
	Households	Population	Year	Households	Population
City of Fitchburg	8,262	20,501	2030	14,843	35,386
City of Madison	89,019	208,054	2030	117,900	264,850
City of Middleton	7,095	15,770	2025	9,173	19,608
City of Monona	3,768	8,018	2010		7,553
City of Verona	2,664	7,052	2030	12,798	31,099
Village of Cottage Grove	1,405	4,059	2025	3,476	9,560
Village of Dane				No plan available	
Village of DeForest	2,675	7,368	2025	4,479	11,865
Village of Maple Bluff	557	1,358			No information in plan
Village of McFarland	2,434	6,416	2025	3,910	9,776
Village of Shorewood Hills				No plan available	
Village of Waunakee	3,295	9,000	2025	5,513	14,855
Town of Vienna	461	1,294	2020	581	1,987
Town of Westport				No information in plan	
Town of Windsor	1,880	5,286	2025	2,412	7,101

Chapter 3 Wastewater Flows

Collection System Description

The MMSD collections system includes approximately 123.8 miles of interceptor sewer and force main, and 17 major pumping stations that transmit wastewater from municipal sewer systems in the MMSD service area to the Nine Springs Wastewater Treatment Plant. There are two points in the collection system where the wastewater can be routed in different directions. The flow from Pumping Station 15 can be routed to Pumping Station 8 or to Pumping Station 16. The flow from Pumping Station 1 can be routed to Pumping Station 2 or to Pumping Station 6. Figure 3-1 is a general flow diagram of the collection system. This schematic illustrates the current, normal operating mode of the system, in which the flow from Pumping Station 15 is routed to Pumping Station 8 and the flow from Pumping Station 1 is routed to Pumping Station 2. However, MMSD typically pumps an average of 150,000 gpd from Pumping Station 1 to Pumping Station 6 to flush the force main for maintenance.

10/23/2008



LEGEND

PumpStation

Flow Direction

Normal

Alternate

GENERAL FLOW DIAGRAM

MADISON METROPOLITAN SEWERAGE DISTRICT
COLLECTION SYSTEM EVALUATION
2008

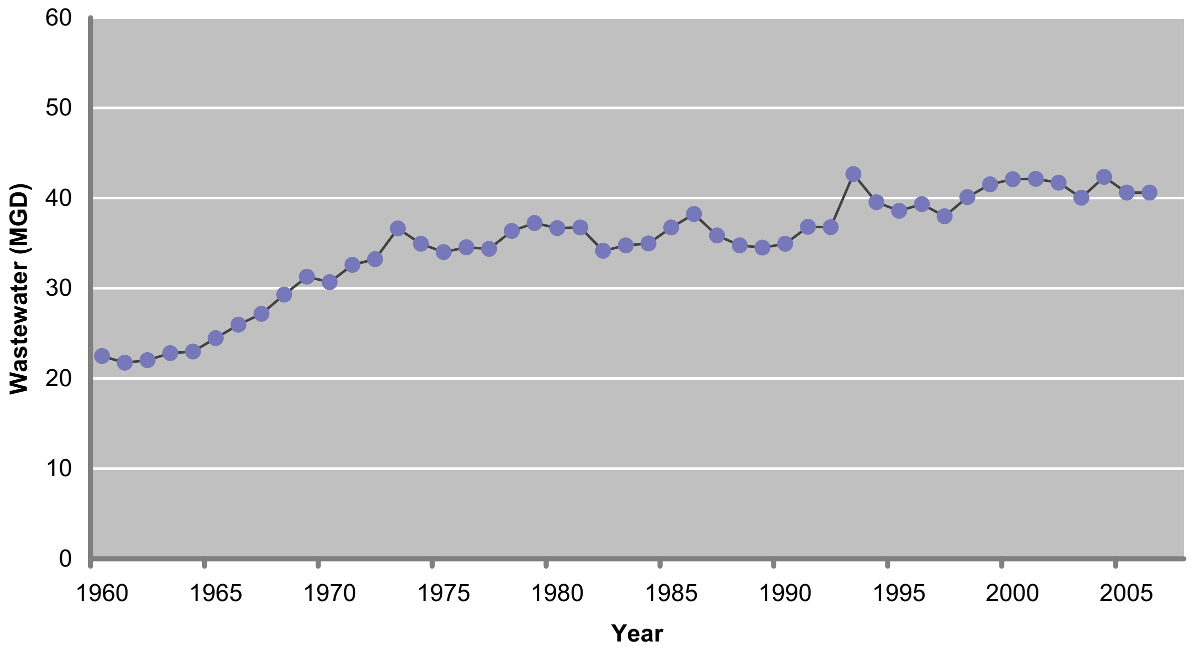
FIGURE 3-1

Historic Water Use and Wastewater Flows

MMSD Metering

Figure 3-2 illustrates the total average daily wastewater flow at the Nine Springs Wastewater Treatment Plant for the period from 1960 to 2007. The 50 mgd average daily flow design capacity currently used for the plant is based on the Seventh Addition design, completed in the early 1980's. A more specific measure of the design capacity of the treatment plant is determined by the design loading of each unit process used at the plant.

Figure 3-2: Average Daily Wastewater Flow at the Nine Springs Treatment Plant



MMSD measures the average daily flow of wastewater in the collection system with five venturi flow meters. They are located at the treatment plant, pumping stations 7, 8, 11, and downstream of the combined flow from pumping stations 2, 3, and 4. In addition, there are flow meters at pumping stations 1, 2, 3, 5, 6, 10, 16, and 17. MMSD calculates average daily flow at the remaining pumping stations from pump run time meters and pump capacities. These pumping station flow records are used as baseline flow data for each pumping station.

Water Utility Records

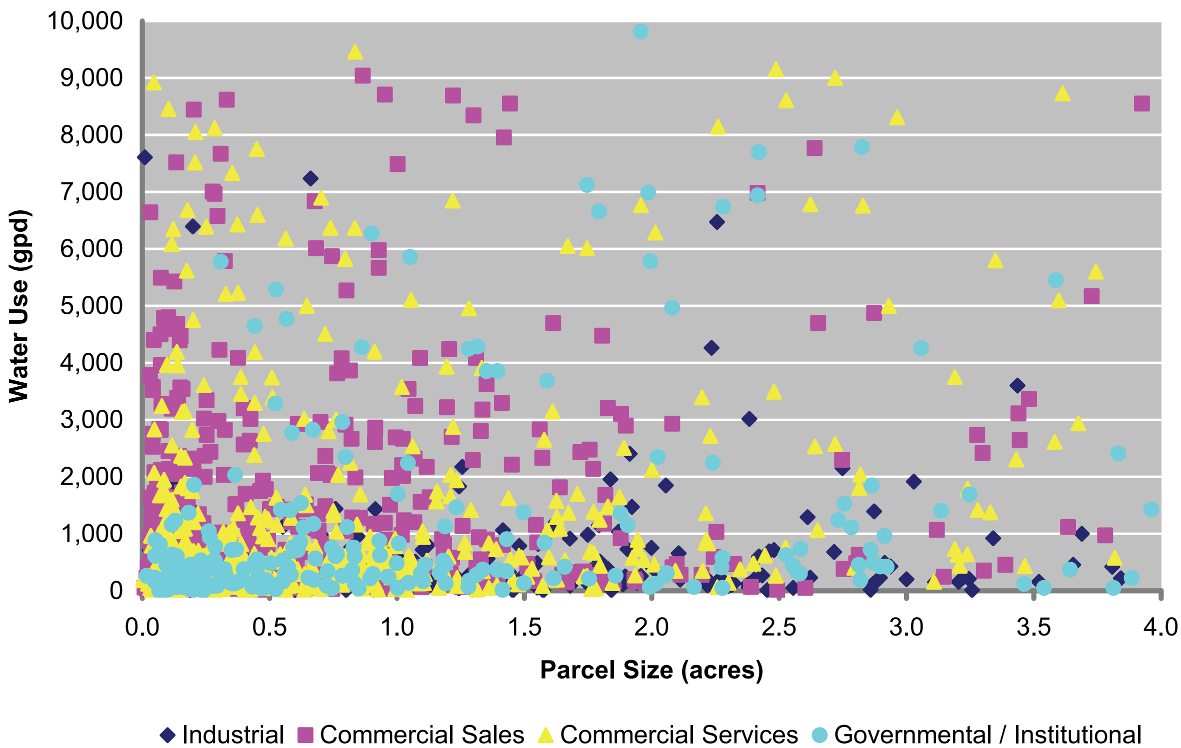
Annual water sales for every water utility are available from Public Service Commission reports. This data is used for estimating wastewater generation in municipalities served by the MMSD. The reports break down annual water sales data into the following categories:

- Residential (which includes single family and two family customers)
- Commercial (which includes commercial customers, multifamily apartments, and the UW)
- Industrial

- Public Authority Customers (which includes, local, state, and federal government customers)
- Sales for Resale (which includes sales to other water utilities)

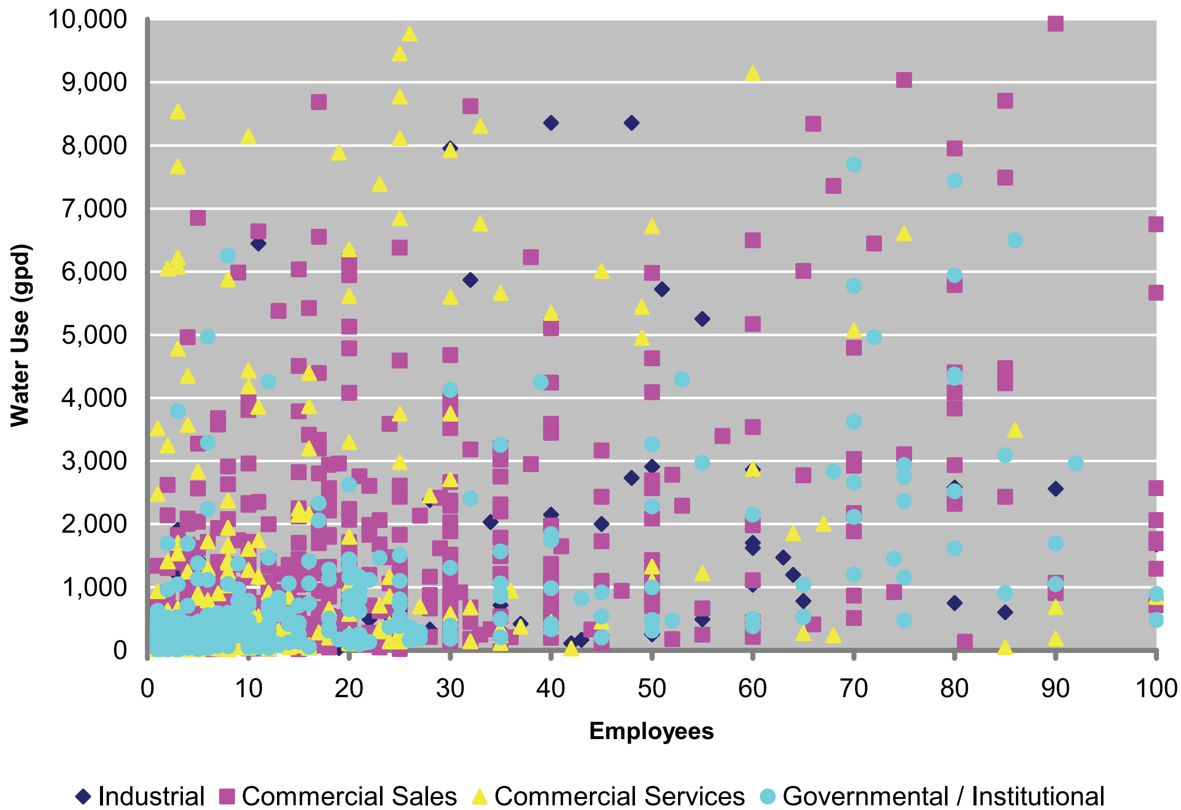
Year 2000 water use records were obtained for over 6,200 commercial, industrial, governmental, and multi-family accounts in the City of Madison. Over 2,000 non-residential accounts were matched to their parcel size and land use. The scatter plot in Figure 3-3 shows the generally weak correlation between water use and parcel size when grouped into industrial, commercial sales, commercial services, and governmental / institutional the land use categories. A linear regression of the data results in R^2 values ranging from 0.04 for governmental / institutional to 0.53 for industrial. A R^2 value near 1 indicates a strong correlation.

Figure 3-3: Water Use vs Parcel Size



The water use records of over 1,800 non-residential locations in the City of Madison were also matched to their number of employees and standard industrial classification in the CLARITAS data. The scatter plot in Figure 3-4 shows the correlation between water use and number of employees when grouped into industrial, commercial sales, commercial services, and governmental / institutional employment categories. A linear regression of the data results in R^2 values ranging from 0.18 for commercial sales to 0.81 for industrial. While this is a better correlation than water use per acre, it is still a weak correlation statistically.

Figure 3-4: Water Use vs Employees



Due to the weak correlation between wastewater generation and parcel size, as well as wastewater generation and number of employees, it was determined that the best methodology for estimating the non-residential component of wastewater generation is to estimate an average wastewater generation rate per employee for each pumping station service area based on actual water meter readings to the extent possible. GIS was used to identify industrial, commercial sales, commercial services, governmental, and institutional parcels based on their 2000 land use codes. Wastewater generation was determined from actual water meter readings for most parcels in the City of Madison. For other parcels, wastewater generation was estimated from CLARITAS employment data and median wastewater generation rates per employee by employment type. Water utility data on commercial, industrial, and public authority water sales were used as control totals for each community. Water meter data from golf courses, greenhouses, heating plants, and similar places were not used. These land uses consume large volumes of water that does not generate wastewater.

Some University of Wisconsin campus buildings have individual water meters. However most of the campus is provided water via the campus distribution system that is only metered in bulk at ten metering pits. The University of Wisconsin Facilities Planning and Management Department provided data from 1997 on the percentage distributed to each building. While this information is out-of-date, it is the best information available.

Pumping Station Service Area Baseline Wastewater Flows

The year 2000 is used as the baseline for the wastewater flow estimates, because actual Census and land use data are available for that year. The baseline wastewater flow estimates are composed of three main components: large sources (> 10,000 gpd), other (non-large) sources, and infiltration / inflow.

Pumping Station Sub-Basins

The collection system is divided into evaluation sections based on MMSD's July 2002 Gravity Interceptor Spreadsheet. Pumping station sub-basins boundaries are defined by which parcels contribute to each collection system section based on available municipal sewer data. There may be some inaccuracies in the pumping station sub-basin boundaries due to out of date or inconclusive municipal sewer data.

Large Wastewater Generators

Large wastewater generators are defined as those contributing greater than 10,000 gpd, based on metered water data. A list of water customers using 10,000 gpd or more in 2000 or 2005 was obtained from each water utility in the MMSD service area. Appendix B summarizes the large wastewater generators in 2000 by pumping station service area.

Other Wastewater Generators

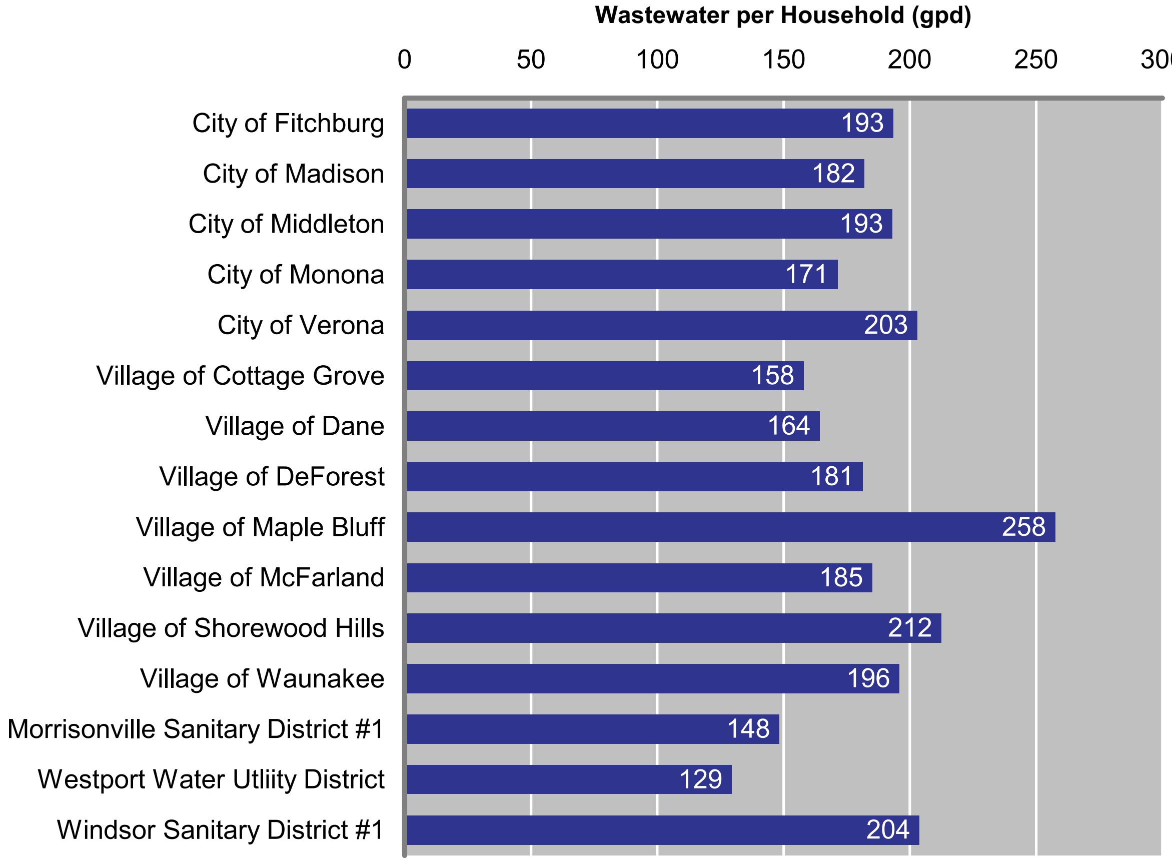
The non-large wastewater generation component is made up of household (single family, two-family, and multifamily) wastewater generation and employment wastewater generation.

Wastewater Generation per Household

Total annual residential (single family and two-family) water use, average number of residential customers, and monthly water pumping records were obtained from water utility reports for each municipality from 1997 to 2006. This data was used to estimate the water use per household per day for each month. To estimate the monthly wastewater generation per household it is necessary to estimate the amount of water used for lawn and garden watering.

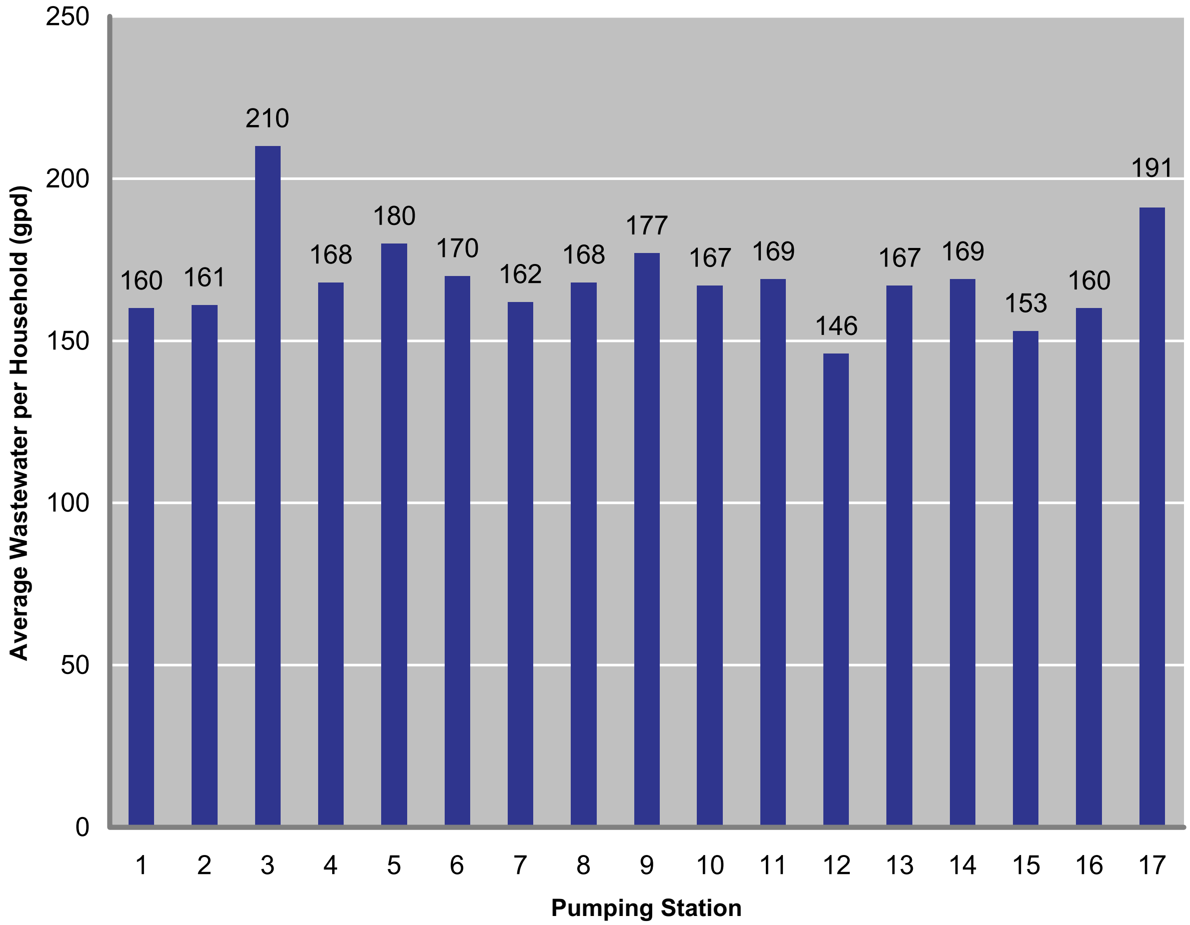
Appendix C contains the graphs of monthly residential water use plotted with monthly rainfall data and the Palmer Z drought index for each community. The assumption is that during periods of wet weather, there is little to no lawn or garden watering, thus residential wastewater generation can be approximated by water use during these wet periods. An average wastewater generation rate per household for 2000 was estimated for each municipality and water utility district as shown in Figure 3-5. The variation in the average household wastewater generation rate by municipality is likely due to differences in average household size, house size, and household water conservation (larger houses have larger housekeeping water use).

Figure 3-5: Residential Wastewater Generation



The amount of wastewater attributable to multifamily households is determined separately, since water utility reports classify multifamily customers as commercial rather than residential. A Geographic Information System (GIS) was used to identify single family, two-family, and multifamily parcels based on their 2000 land use codes. Single family and two family parcels are assigned the wastewater generation rate in Figure 3-5, based on their location. Multifamily wastewater generation was based on actual water meter readings for most parcels in the City of Madison. For other parcels, an average rate per multifamily unit was estimated from parcels with actual water meter readings. The number of multifamily units for each parcel was obtained from municipal property records, where available, or estimated from aerial photographs and census data where better information was not available. In general, the multifamily wastewater generation rates are lower than the single family / two-family wastewater generation rates, due to smaller units and the average household size being smaller. An average wastewater generation rate for all households (single family, two-family, and multifamily) was calculated for each pumping station service area by adding the wastewater allocation for the residential parcels and dividing by the number of households on those parcels. The results are shown in Figure 3-6.

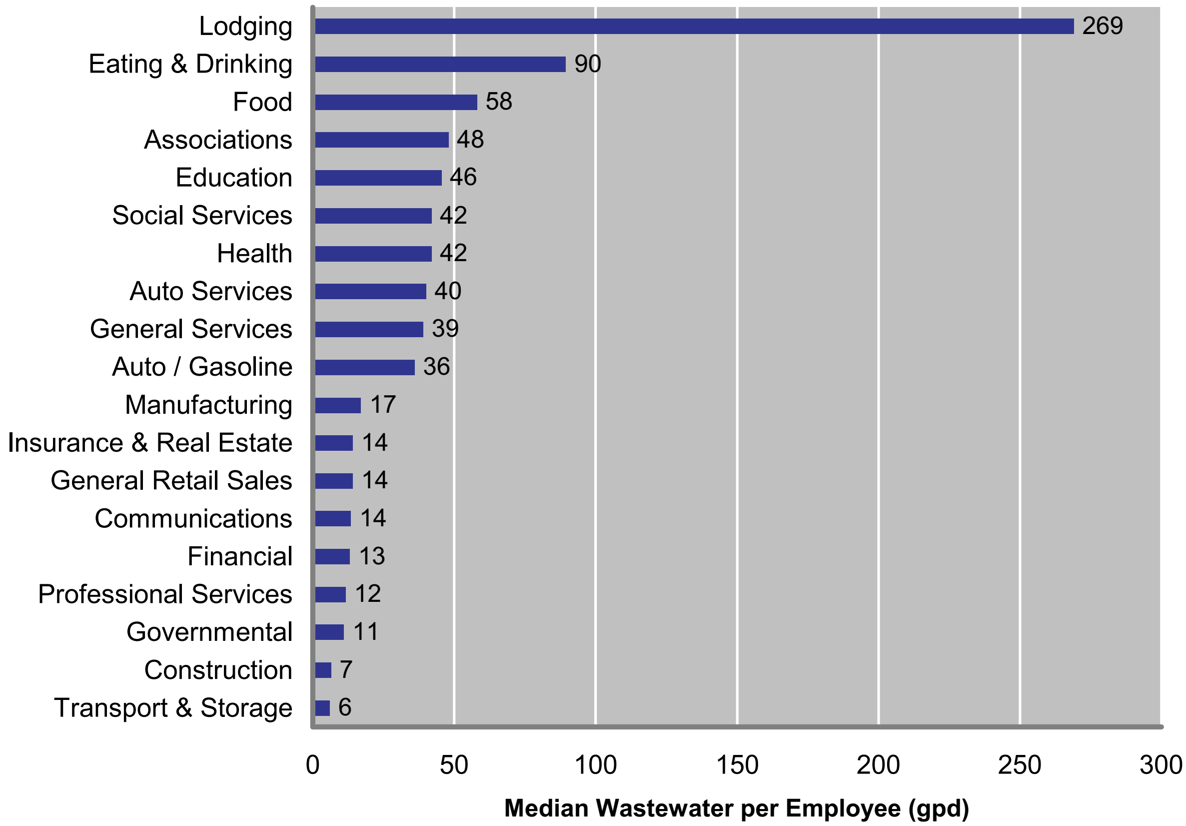
Figure 3-6: Household Wastewater Generation by Pumping Station



Wastewater Generation per Employee

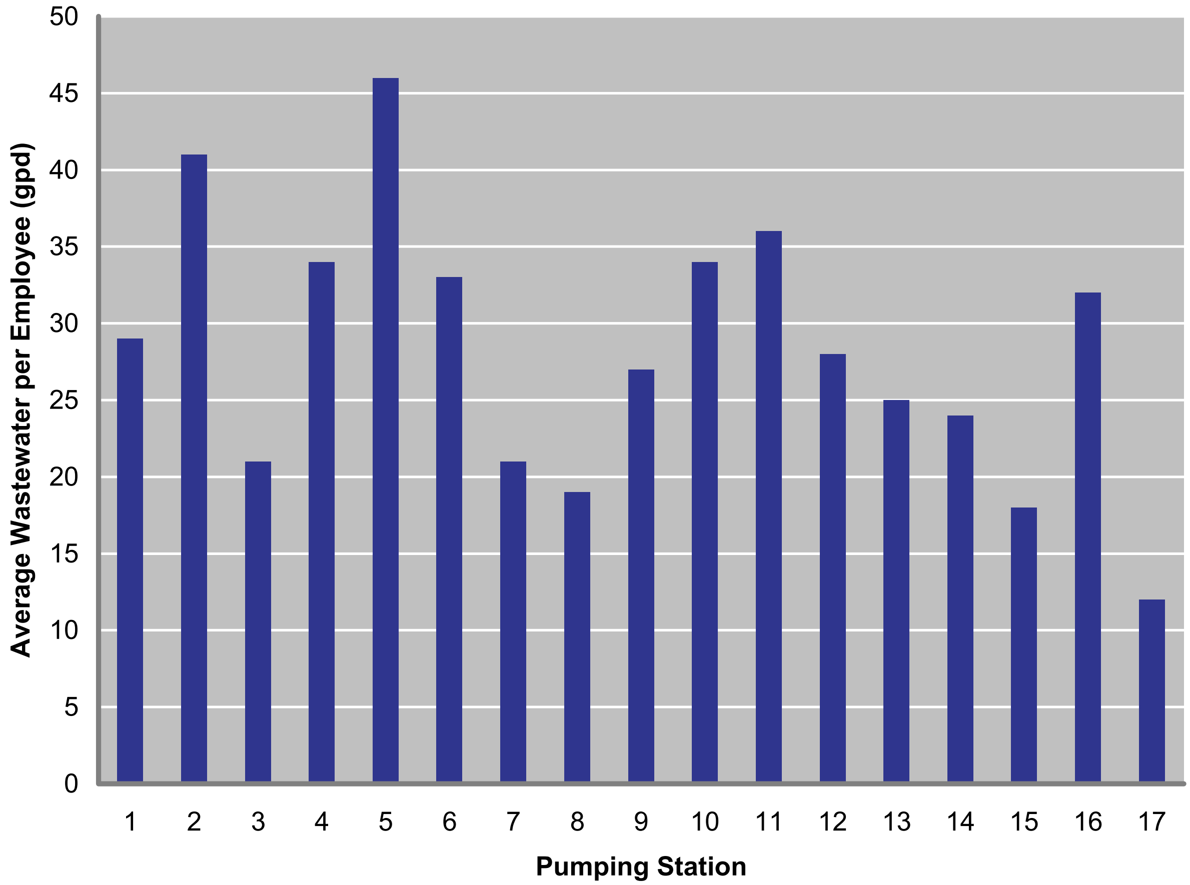
The median wastewater generation rates per employee were calculated for over 1,800 non-residential locations in the City of Madison by comparing their year 2000 water use records to their number of employees in the CLARITAS data. The results were grouped into 19 employment categories, based on their Standard Industrial Classification (SIC) codes. The median wastewater generation rate for each employment category is shown in Figure 3-7. There is a stronger correlation between wastewater generation and employment category, however there is still considerable variation within each category.

Figure 3-7: Median Wastewater Generation Rate by Employment Type



The total estimates of employment wastewater and employment were used to calculate an average wastewater generation rate per employee by pumping station as shown in Figure 3-8. These rates do not include large generators.

Figure 3-8: Average Wastewater Generation per Employee by Pumping Station



Infiltration / Inflow

The amount of infiltration and inflow (I/I) in each pumping station service area in 2000 is estimated by subtracting the estimate of the total wastewater flow for the pumping station service area from MMSD's pumping station flow records. In some cases, where the year 2000 meter data was suspect, the 2005 meter data was used or I/I was assumed to be 10%, which MMSD staff determined to be a reasonable average value. These instances are noted in the discussion of wastewater forecasts for each pumping station. I/I is distributed among the sub-basins proportional to sub-basin areas.

Pumping Station Service Area Wastewater Flow Forecasts

2030 Forecast Methodology

The basic approach to forecasting year 2030 wastewater flows is to use the estimated average household and employee wastewater generation rates in each pumping station for the baseline year and to multiply those rates by household and employment forecasts for each pumping station sub-basin. Two different 2030 forecast are generated, a TAZ forecast and an Uncertainty Factor (UF) forecast. This approach is based on several assumptions:

1. Future residential growth will have water use characteristics that are similar to current residential units (no dramatic housing type changes or substantial conservation measures). Because collection system studies are updated every 10 years, this assumption is expected to be valid in the context of other factors of safety used in operating and maintaining the collection system.
2. Future employment growth will be for businesses with characteristics that are similar to current businesses in each pumping station area. This assumption is expected to be valid in light of the fact that new wet industries and employment centers are screened by MMSD and CARPC (through the sewer extension review process) to ensure the availability of collection system capacity.

The use of uncertainty factors in flow forecasts (further discussed below) also accounts for some potential variability in future growth characteristics, so long as the variability is not dramatic.

TAZ Forecasts

In the TAZ forecasts, the TAZ regions are subdivided to coincide with pumping station sub-basins. The TAZ data within each sub-basin was added together to determine the household and employment forecast for the sub-basin.

Uncertainty Factor Forecasts

There is uncertainty about the accuracy of the TAZ data since it was developed before most of the current municipal comprehensive plans and neighborhood plans. The uncertainty factor (UF) forecast for households and employment looks at the development identified in these plans in addition to the TAZ data. Development plans were allocated to sub-basin areas based on the information contained in the plans and most current land use and aerial photography. In most cases future development was allocated proportionally to sub-basins based on available land area. Estimates of redevelopment are based on housing trends in each pumping station service area. To provide a conservative, upper end estimate, the higher projected value of households and employment between the TAZ data and the development plan data was used for the uncertainty factor forecasts.

Pumping Station Sub-Basins

The potential 2030 sub-basin boundaries are based on municipal development plans, meetings with municipal planners, and future municipal sewer location information where available. Contour data is used to estimate future sub-basin boundaries where more detailed information is not available. The pumping station sub-basin figures include the Wisconsin Department of Natural Resources data layer for wetlands larger than two acres. This information is included to illustrate areas where I/I may be a concern as well as areas that may not be developable. They cannot be used to delineate wetland boundaries.

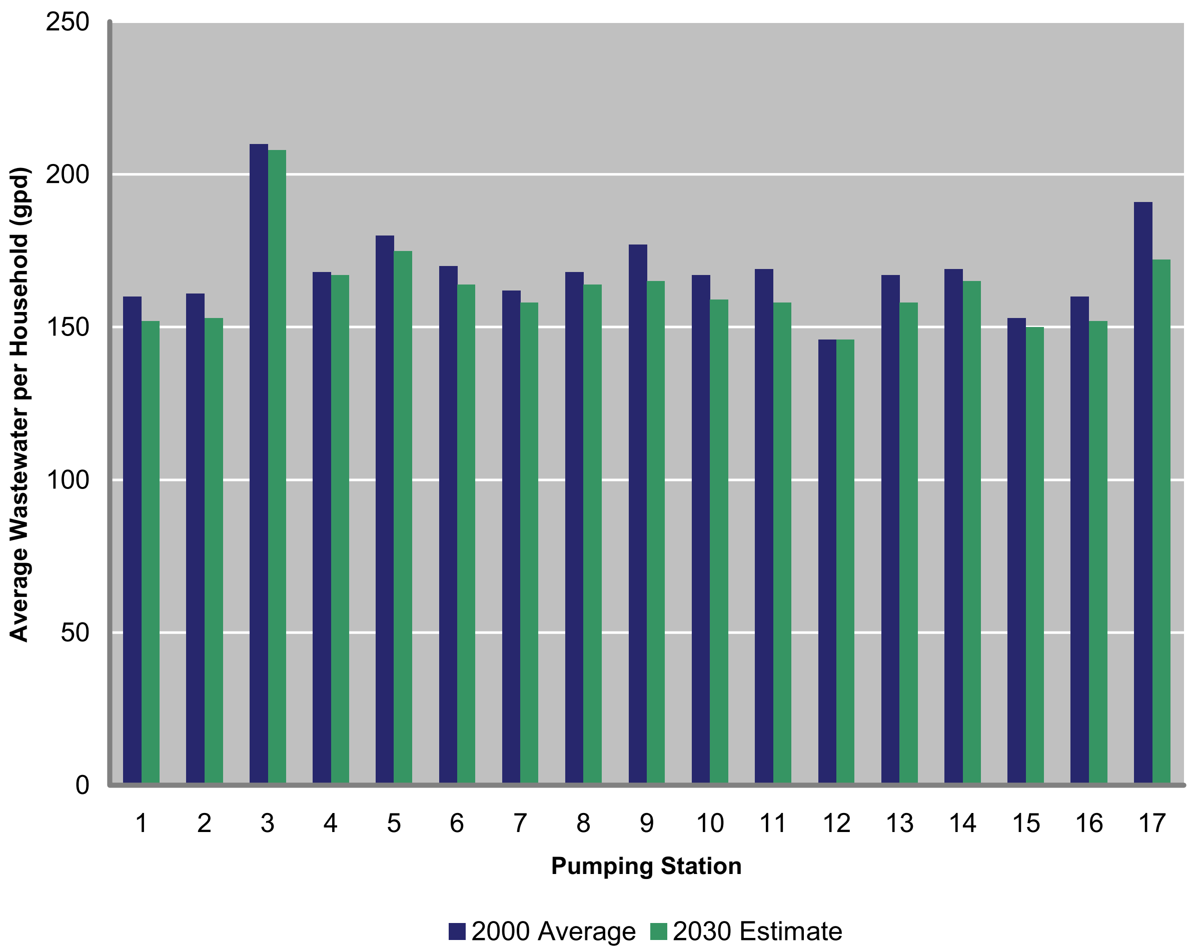
Large Wastewater Generators

The 2030 wastewater flow forecasts from large generators are assumed to remain at 2000 levels, except in those few cases where there was a significant decrease in water use from 2000 to 2005. These instances are noted in the discussion of wastewater forecasts for each pumping station.

Wastewater Generation per Household

The 2030 wastewater generation rate per household was assumed to decrease in most pump station service areas due to increased water conservation. The amount of the decrease was based on the trend of water use for each municipal water utility as shown in the graphs in Appendix C. The 2000 baseline and 2030 forecast of average household wastewater generation by pumping station is shown in Figure 3-9.

Figure 3-9: Household Wastewater Generation by Pumping Station



Wastewater Generation per Employee

The 2030 average wastewater generation rates per employee for each pumping station service area are assumed to remain at 2000 levels in the 2030 TAZ projections. There is some uncertainty that the average rates will not increase in the future for those pumping station service areas that are expected to have a large increase in employment by 2030. In the 2030 Uncertainty Factor (UF) projections the future wastewater generation rate per employee was increased in those areas where there will be a large increase in employment. A maximum rate of 46 gpd per employee was used, since this was the maximum average rate for any pumping station in 2000.

Infiltration and Inflow

Infiltration and inflow (I/I) is assumed to be the same in 2030 as it was in 2000, except where noted in the narrative for each pumping station.

2060 Forecast Methodology

The only purpose for deriving a 2060 flow forecast is to assist MMSD in sizing collection system pipes. While the treatment components of the wastewater system are designed with a 20-year planning horizon, pipes have an expected life of 50-70 years and need to be sized accordingly. There are no TAZ forecasts for households or employment in 2060. Therefore, a different methodology is used for the long-term, 2060 wastewater flow forecasts. The basic approach to forecasting year 2060 wastewater flows is to use an estimated average per capita wastewater generation rate for each pumping station (excluding large generators), and to multiply the rate by the 2060 population forecasts for each pumping station sub-basin. Wastewater from large generators is assumed to be the same in 2060 as it was in 2030. I/I is assumed to be the same in 2060 as it was in 2030. These components are added together to estimate the total 2060 wastewater forecast for each sub-basin.

Pumping Station Sub-Basins

The potential 2060 sub-basin boundaries are based on municipal development plans, meetings with municipal planners, and future municipal sewer location information where available. Contour data is used to estimate future sub-basin boundaries where more detailed information is unavailable.

2060 Population Forecast

The 2030 Uncertainty Factor forecast is used as the baseline number of households for the 2060 forecast. The increase in households for each sub-basin from 2030 to 2060 is estimated from the long-range development plans in each community. These are added together to estimate the number of households in each sub-basin in 2060. The 2060 population forecast for each sub-basin is then calculated. It assumes that the average household size (the number of persons per household) in each sub-basin forecast in the 2030 TAZ data will remain the same through 2060.

Wastewater Generation per Capita

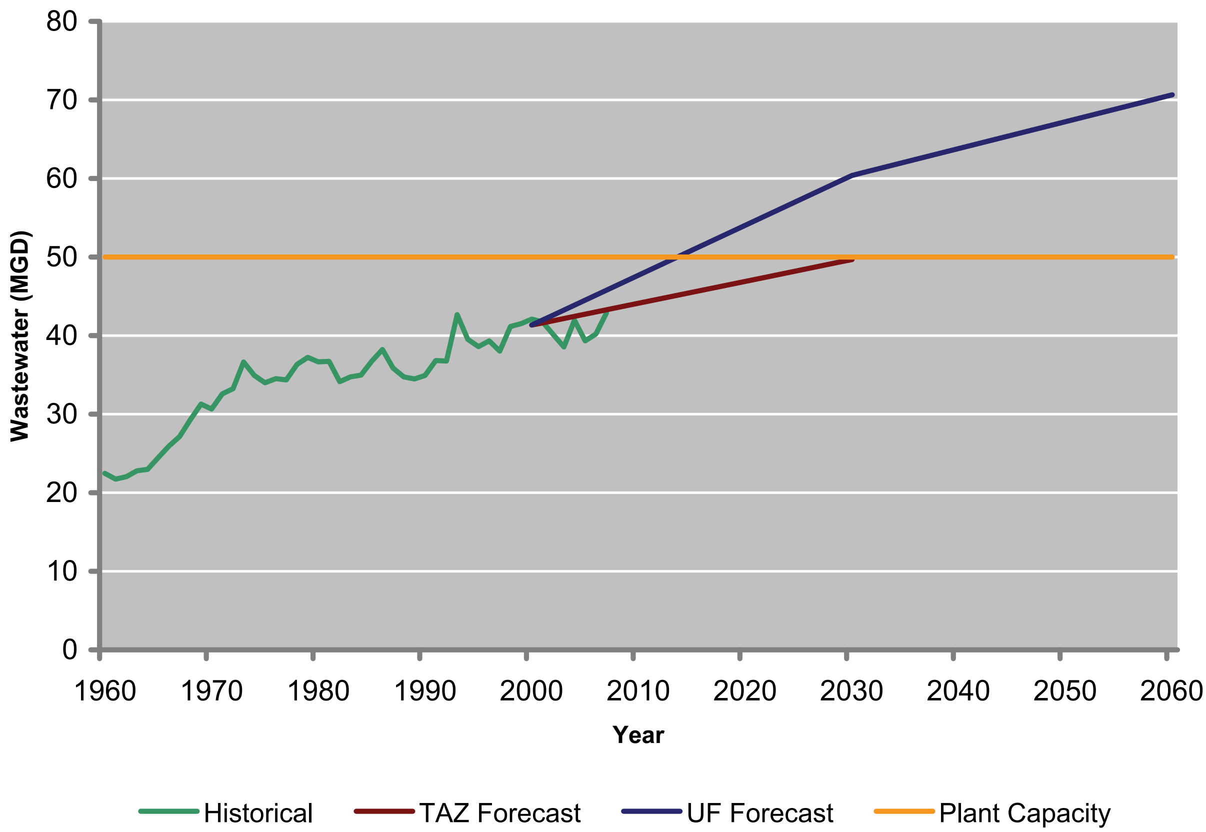
The wastewater generation rate per capita for other (non-large) sources in each sub-basin is calculated from the 2030 UF forecasts. The 2030 UF wastewater forecast for each sub-basin from non-large sources is divided by the sub-basin population to determine the per capita rate. The 2060 wastewater generation rate per capita is assumed to be the same as the 2030 UF wastewater generation rate per capita.

Cumulative Forecasts

The cumulative 2030 TAZ forecast projects the 2030 MMSD service area to contain 187,382 households and a population of 431,110. This is reasonably consistent with (within 7% of) the 2030 population estimate of 405,804 from Table 2-1, based on the CARPC / DOA population forecasts for the area. The cumulative average daily wastewater flow for the 2030 TAZ forecast is 49.68 mgd, near the current rated design capacity of the Nine Springs Treatment Plant of 50 mgd average daily flow.

The cumulative 2030 UF forecast projects the 2030 MMSD service area to contain 242,551 households and a population of 554,654. This is considerably higher (approximately 37% more) than the CARPC / DOA official population forecasts for the area. It is unlikely that all of the development projected by the 2030 UF forecast will occur by 2030. However it is probable that some of the sub-basin areas will develop to the levels projected in the 2030 UF forecast by 2030.

Figure 3-45: WWTP Meter Data vs Wastewater Forecasts



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Chapter 4 Collection System Capacity Evaluation

Overall Analysis Approach

The collection system is divided into evaluation sections based on MMSD's 2002 Collection System Facilities Plan. A section is defined as a distinct part of portion of the system that has similar hydraulic components, a generally larger division related by system capacity. The average wastewater flow for each pumping station sub-basin is added cumulatively as the flow from each sub basin enters the collection system. Peak flows were determined by applying the standard MMSD peaking factor formula, shown in Equation 1, to the cumulative average flows. A minimum peak factor of 2.5 and a maximum peak factor of 4.0 are used.

Equation 1: Peaking Factor

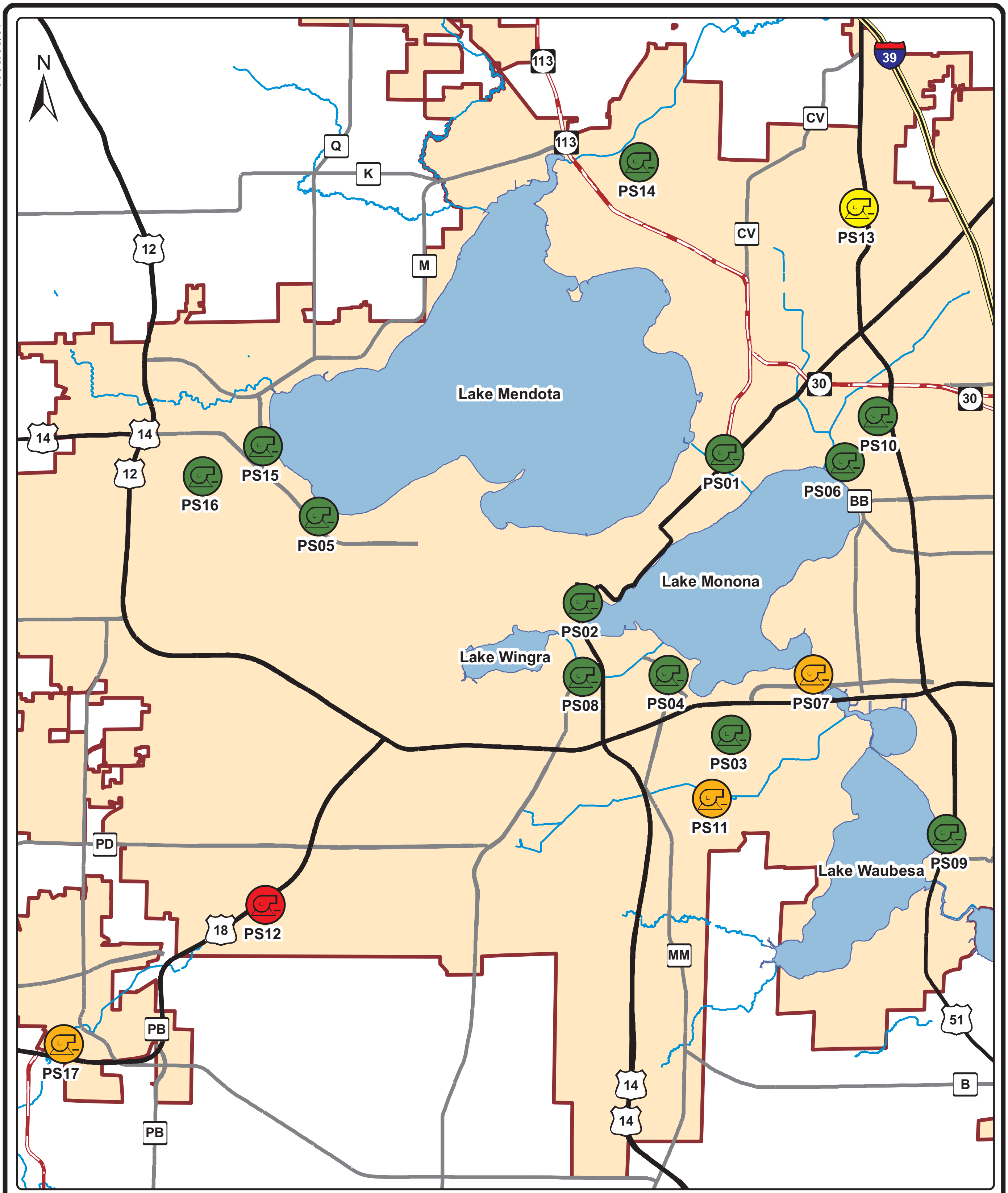
$$PeakFactor = \frac{4}{AverageFlow^{0.158}}$$

The peaking factors used for each sub-basin and the resulting cumulative peak flows are included in Appendix E. Detailed information on the hydraulic and pipe characteristics (i.e. invert elevation, size, slope, pipe material, friction factor and capacity) of each manhole segment from MMSD's collection system database is in Appendix G. A segment is defined as one run of sewer, the smallest part of the system, beginning at one manhole and ending at the next. Pumping station characteristics are described in Table 4-3. The capacities given for Pumping Station 6 and Pumping Station 8 are the planned capacities of these two stations after they are rehabbed in 2009 / 2010. Nominal force main capacities are based on a velocity of 8 feet per second, except for the force main from Pumping Station 7. The Pumping Station 7 force main capacity is 55 mgd based on transients.

Peak wastewater flows for 2010 and 2020 are interpolated from 2000 and 2030 UF wastewater flow projections. The 2030 UF projections are used for the capacity analysis rather than the 2030 TAZ projections because they are higher, and therefore more conservative. In cases where sub-basins were added or removed from the pumping station service area between 2000 and 2030, the peak wastewater flows for 2010 and 2020 are interpolated from 2000 to the wastewater flow projection in the year of the sub-basin change and from the year of the sub-basin change to the 2030 UF wastewater flow projection. Peak wastewater flows for 2060 are calculated from the 2060 UF wastewater flow projections as shown in Appendix E.

Figures 4-1 through 4-24 show various sections of the MMSD collection system. Each section of the collection system is color-coded based on the date range when that section is projected to reach capacity. Summary tables including the collection system sections, nominal capacity, and peak flow projections follow each figure.

12/22/2008



Capacity Reached

- 2000 - 2010
- 2010 - 2020
- 2020 - 2030
- > 2030

PUMPING STATION FIRM CAPACITY USING TAZ FLOWS

MADISON METROPOLITAN SEWERAGE DISTRICT COLLECTION SYSTEM EVALUATION

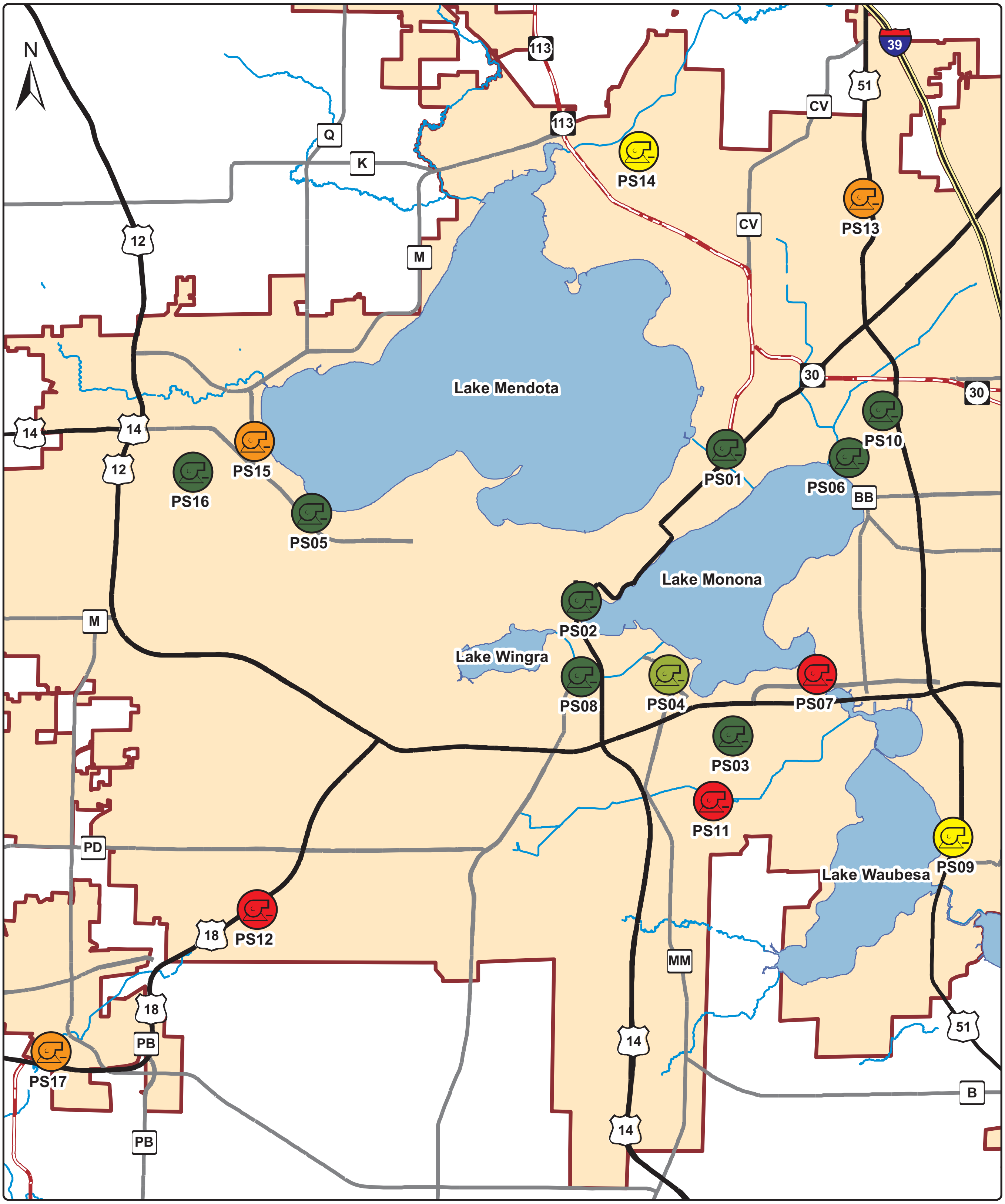
2008

0 1.25 2.5 5 Miles

FIGURE 4-1

Table 4-1: Pumping Station Capacity Evaluation – TAZ Flows

Pumping Station	Station Capacity		*Peak Flows (mgd)* / Percent Firm Capacity								Firm Capacity Reached
	Maximum	Firm	2000		2010 TAZ		2020 TAZ		2030 TAZ		
1	38.3	35.3	*19.1*	54%	*15.7*	44%	*15.9*	45%	*16.1*	46%	> 2030
2	41.0	41.0	*28.7*	70%	*26.4*	64%	*26.7*	65%	*27.1*	66%	> 2030
3	1.5	1.5	*1.2*	83%	*1.2*	83%	*1.3*	85%	*1.3*	86%	> 2030
4	4.2	4.2	*3.9*	93%	*3.9*	92%	*3.9*	93%	*3.9*	94%	> 2030
5	3.6	3.6	*2.6*	72%	*2.6*	71%	*2.4*	66%	*2.4*	67%	> 2030
6	24.2	24.2	*5.8*	24%	*6.0*	25%	*6.2*	25%	*6.4*	26%	> 2030
7	45.0	39.0	*35.1*	90%	*39.0*	100%	*42.5*	109%	*45.9*	118%	2010-2020
8	34.1	34.0	*25.1*	74%	*24.0*	71%	*24.1*	71%	*24.3*	71%	> 2030
9	4.5	4.5	*3.2*	72%	*3.6*	79%	*3.9*	87%	*4.2*	94%	> 2030
10	42.2	42.2	*23.1*	55%	*25.2*	60%	*27.2*	64%	*29.3*	69%	> 2030
11	31.2	25.5	*22.0*	86%	*25.6*	100%	*29.1*	114%	*32.5*	127%	2010-2020
12	23.5	16.6	*14.1*	85%	*17.3*	104%	*20.3*	122%	*23.2*	140%	2000-2010
13	20.2	20.0	*17.0*	85%	*18.5*	93%	*20.0*	100%	*21.6*	108%	2020-2030
14	15.6	15.0	*11.0*	73%	*12.2*	82%	*13.4*	89%	*14.6*	97%	> 2030
15	8.8	5.8	*5.4*	93%	*5.0*	86%	*5.3*	92%	*5.6*	97%	> 2030
16	18.7	18.7	*5.7*	30%	*7.4*	40%	*7.6*	41%	*8.5*	46%	> 2030
17	4.6	4.6	*2.7*	58%	*3.4*	74%	*6.3*	136%	*7.8*	170%	2010-2020



Capacity Reached
- > 2060
- 2030 - 2060
- 2020 - 2030
- 2010 - 2020
- 2000 - 2010

PUMPING STATION FIRM CAPACITY USING UNCERTAINTY FACTOR FLOWS

MADISON METROPOLITAN SEWERAGE DISTRICT
COLLECTION SYSTEM EVALUATION
2008

0 1.25 2.5 5 Miles

FIGURE 4-2

Table 4-2-: Pumping Station Capacity Evaluation – Uncertainty Factor Flows

Pumping Station	Station Capacity		*Peak Flows (mgd)* / Percent Firm Capacity										Firm Capacity Reached
	Maximum	Firm	2000		2010 UF		2020 UF		2030 UF		2060 UF		
1	38.3	35.3	*19.1*	54%	*16.0*	45%	*16.4*	47%	*16.9*	48%	*18.5*	52%	> 2060
2	41.0	41.0	*28.7*	70%	*27.3*	66%	*28.4*	69%	*29.5*	72%	*33.7*	82%	> 2060
3	1.5	1.5	*1.2*	83%	*1.3*	86%	*1.3*	89%	*1.4*	93%	*1.4*	93%	> 2060
4	4.2	4.2	*3.9*	93%	*4.0*	94%	*4.0*	96%	*4.1*	97%	*4.3*	102%	2030-2060
5	3.6	3.6	*2.6*	72%	*2.4*	68%	*2.5*	69%	*2.5*	70%	*2.7*	74%	> 2060
6	24.2	24.2	*5.8*	24%	*6.0*	25%	*6.2*	25%	*6.4*	26%	*7.1*	30%	> 2060
7	45.0	39.0	*35.1*	90%	*43.0*	110%	*50.6*	130%	*59.9*	153%	*72.3*	185%	2000-2010
8	34.1	34.0	*25.1*	74%	*25.0*	73%	*25.6*	75%	*26.2*	77%	*28.0*	82%	> 2060
9	4.5	4.5	*3.2*	72%	*3.9*	86%	*4.4*	98%	*4.9*	110%	*6.4*	142%	2020-2030
10	42.2	42.2	*23.1*	55%	*27.3*	65%	*31.3*	74%	*35.3*	84%	*38.7*	92%	> 2060
11	31.2	25.5	*22.0*	86%	*27.9*	109%	*33.6*	132%	*39.2*	154%	*44.8*	176%	2000-2010
12	23.5	16.6	*14.1*	85%	*19.3*	116%	*24.2*	146%	*28.9*	174%	*32.3*	195%	2000-2010
13	20.2	20.0	*17.0*	85%	*20.0*	100%	*22.9*	115%	*25.8*	129%	*29.4*	147%	2010-2020
14	15.6	15.0	*11.0*	73%	*12.8*	85%	*14.5*	97%	*16.2*	108%	*20.2*	134%	2020-2030
15	8.8	5.8	*5.4*	93%	*5.9*	102%	*6.3*	108%	*6.7*	115%	*7.6*	131%	2010-2020
16	18.7	18.7	*5.7*	30%	*8.3*	44%	*8.8*	47%	*10.2*	55%	*10.6*	56%	> 2060
17	4.6	4.6	*2.7*	58%	*3.9*	85%	*8.7*	188%	*11.3*	245%	*13.6*	295%	2010-2020

Table 4-3: Pumping Station Characteristics

Pumping Station No.	Station Location and Year Placed On-Line	Pumping Station Capacity		Individual Pump No.	Estimated Pump Performance at Turn-On Elevation		Nominal speed	Nominal Motor Size	Year Pump On-line	Comments
		Maximum	Firm		Q (gpm)	H (ft.)	(rpm)	(HP)		
1	104 N. First St. Madison 1950	1A (or 1B) + 1D 26,600 gpm 38.3 mgd	1A (or 1B) + 1C 24,475 gpm 35.3 mgd	1A	14,100	134	890	600	2005	1A & 1B are the new Crosstown pumps and pump to PS#2. 1C & 1D are the old pumps (with re-wound motors) and pump to PS#6. 1A or 1B can pump with 1C or 1D. Pump 1D rating per 6/96 venturi analysis.
				1B	14,100	134	890	600	2005	
				1C	10,375	31	580	150	1950	
				1D	12,500	41	585	150	1950	
2	833 W. Washington Brittingham Park Madison 1964	Any 3 pumps 9,500 gpm (ea) 28,500 gpm total 41.0 mgd total	Any 3 pumps 9,500 gpm (ea) 28,500 gpm total 41.0 mgd total	2A	16,500	108	890	600	2005	All pumps were replaced during station rehab in 2005. All 4 pumps are equal size. 2A & 2B are VFD and 2C & 2D are constant speed. Data reflects new 36" FM online in 2001.
				2B	16,500	108	890	600	2005	
				2C	16,500	108	890	600	2005	
				2D	16,500	108	890	600	2005	
3	Nine Springs WWTP 1959	3A or 3B 1050 gpm 1.51 mgd	3A or 3B 1050 gpm 1.51 mgd	3A	1,050	60	1175	30	1980	New 36" FM (Aug. 2001) has no significant impact on capacities. New Headworks (Aug. 2005) adds ~4' static. New impellers (13.0" vs 12.2") installed in 2004.
				3B	1,050	60	1175	30	1980	
4	620 John Nolen Drive, Madison 1967	4B or 4C 2,900 gpm 4.2 mgd	4B or 4C 2,900 gpm 4.2 mgd	4A	2,000	47	860	40	1967	Peak capacities include new 36" FM (8/2001), new Headworks (8/2005), WSEL=32, wetwell @ -7, PS3 @1,000gpm, PS2 @ 28,500 gpm. New impellers (17.0" vs 16.25") in 4B&4C-2004.
				4B	2,900	95	1160	100	1967	
				4C	2,900	95	1160	100	1967	
5	Spring Harbor Park Madison 1996	Any two pumps 2,480 gpm 3.6 mgd	Any two pumps 2,480 gpm 3.6 mgd	5A	1,800	75	1256	50	1996	Variable speed units. Ratings per 1996 startup testing at 106% speed.
				5B	1,800	75	1256	50	1996	
				5C	1,800	75	1256	50	1996	

Pumping Station No.	Station Location and Year Placed On-Line	Pumping Station Capacity		Individual Pump No.	Estimated Pump Performance at Turn-On Elevation		Nominal speed	Nominal Motor Size	Year Pump On-line	Comments
		Maximum	Firm		Q (gpm)	H (ft.)	(rpm)	(HP)		
6	402 Walter Street Madison 1950	Any 3 pumps 5,600 gpm (ea) 16,800 gpm total 24.2 mgd total	Any 3 pumps 5,600 gpm (ea) 16,800 gpm total 24.2 mgd total	6A	7,700	45	890	125	2009	All ratings shown are **after** station rehabilitation in 2009. All 4 pumps are equal size. 6A is variable speed and 6B-6D are constant speed.
				6B	7,700	45	890	125	2009	
				6C	7,700	45	890	125	2009	
				6D	7,700	45	890	125	2009	
7	6300 Metropolitan Lane, Monona 1950	7C + 7D 31,250 gpm 45.0 mgd	7B + 7C 27,100gpm 39.0 mgd	7A	11,500	47	695	60	1950	Dual pump ratings per 1996 high flow data. No major pump changes since station was rehabbed in 1992.
				7B	15,200	53	705	250	1992	
				7C	19,400	59	705	350	1992	
				7D	19,400	59	705	350	1992	
8	901 Plaenart Dr. Madison 1964	8C+8D+8A(or 8B) 7,900 gpm (ea) 23,700 gpm total 34.1 mgd total	8A+8B+8C(or 8D) 7,850 gpm (ea) 23,600 gpm total 34.0 mgd total	8A	12,800	58	585	250	2009	All ratings shown are **after** station rehabilitation in 2009. 8A&8B (formerly 8C&8D)are variable speed and equal size. 8C&8D (formerly 6C&6D) are constant speed and equal size.
				8B	12,800	58	585	250	2009	
				8C	13,900	60	705	300	2009	
				8D	13,900	60	705	300	2009	
9	4612 Larsen Beach Road, McFarland 1962	Any two pumps 3,150 gpm 4.5 mgd	Any two pumps 3,150 gpm 4.5 mgd	9A	2,300	51	1185	40	2003	All American Well Works pumps were replaced with Fairbanks Morse Built-Togethers (5434S) between 2002 & 2007. New pumps are same capacity as old.
				9B	2,300	51	1185	40	2007	
				9C	2,300	51	1185	40	2002	
10	192 Regas Road Madison 1965	Any 2 pumps 14,700 gpm (ea) 29,400 gpm total 42.2 mgd total	Any 2 pumps 14,700 gpm (ea) 29,400 gpm total 42.2 mgd total	10A	18,900	94	890	600	2005	All pumps were replaced during station rehab in 2005. All 3 pumps are equal size. 10A & 10B are VFD and 10C is constant speed. Pumps are currently not allowed to operate in parallel.
				10B	18,900	94	890	600	2005	
				10C	18,900	94	890	600	2005	
11	4760 E. Clayton Rd. Town of Dunn 1966	11C + 11D 21,700 gpm 31.2 mgd	11C or 11D + 11B 17,700gpm 25.5 mgd	11A	6,400	43	860	125	1950	11A relocated to PS11 from PS7. 11C & 11D individual capacities per testing in 2/2008. Firm capacity (11C or 11D in parallel with 11B) per testing in 2/2008.
				11B	9,100	49	880	150	1982	
				11C	13,300	57	705	250	1982	
				11D	13,300	57	705	250	1982	

Pumping Station No.	Station Location and Year Placed On-Line	Pumping Station Capacity		Individual Pump No.	Estimated Pump Performance at Turn-On Elevation		Nominal speed	Nominal Motor Size	Year Pump On-line	Comments
		Maximum	Firm		Q (gpm)	H (ft.)	(rpm)	(HP)		
12	2739 Fitchrona Rd. Town of Verona 1969	12C + 12D 16,300 gpm 23.5 mgd	12C or 12D + 12B 11,500 gpm 16.6 mgd	12A	3,400	44	700	50	1969	Firm capacity (12C or 12D in parallel with 12B) per estimate in 2/2008.
				12B	7,200	48	885	100	1969	
				12C	9,000	48	880	150	1982	
				12D	9,000	48	880	150	1982	
13	3634 Amelia Earhart Drive, Madison 1970	13C 14,000 gpm 20.2 mgd	13A + 13B 13,900 gpm 20.0 mgd	13A	8,200	16	585	50	2008	Pump 13A replaced in 2008. 13A matches 13B. Pump 13B re-built, including new impeller (same size). Pump 13C unchanged.
				13B	8,200	16	585	50	1970	
				13C	14,000	20	505	100	1970	
14	5000 School Rd. Madison 1971	14C 10,800 gpm 15.6 mgd	14A + 14B 10,400 gpm 15.0 mgd	14A	7,200	24	705	60	2008	Pump 14A replaced in 2008. 14A matches 14B. Pump 14B re-built, including larger impeller (17.375" vs. 16.5"). Pump 14C re-built with larger impeller (22.0" vs. 20.5").
				14B	7,200	24	695	60	1971	
				14C	10,800	29	585	100	1971	
15	2115 Allen Blvd. Madison 1975	15C 6,100 gpm 8.8 mgd	15A 4,000 gpm 5.8 mgd	15B	3,000	68	885	100	1975	Pump ratings shown are for pumping to the West Int. and PS8. See note (ii).
				15A	4,000	76	885	100	1975	
				15C	6,100	100	885	200	1982	
16	1303 Gammon Rd. Middleton 1982	Any two pumps 13,000 gpm 18.7 mgd	Any two pumps 13,000 gpm 18.7 mgd	16A	7,000	182	1185	500	1982	
				16B	7,000	182	1185	500	1982	
				16C	7,000	182	1185	500	1982	
17	405 Bruce Street Verona 1996	Any two pumps at 118% speed 3,200 gpm 4.6 mgd	Any two pumps at 118% speed 3,200 gpm 4.6 mgd	17A	2,300	115	1290	100	1996	Variable speed pumps. Nominal 100% speed=1190 rpm. Ratings shown are for 118% max speed. Incorporated dual pumping in 2007. Capacity based on 2008 testing
				17B	2,300	115	1290	100	1996	
				17C	2,300	115	1290	100	1996	

Notes:

i) Pump ratings are based on analysis of pump performance curves and system curves, and where available, flow meter data.

ii) For PS15 diversion to PS16, pump ratings are as follows: 15B) 1500 gpm @ 84' 15A) 3000 gpm @ 87' 15C) 6500 gpm @ 96'.

iii) Pump ratings are per pump turn-on level (high wetwell) and C=130.

iv) Due to limited downstream interceptor capacity, PS10 is currently limited to one pump operation (dual pumping is not allowed).

Chapter 5 Issues and Alternatives

There is the potential to postpone or avoid the projected need for capacity improvements if the projected flow increases can be offset by reducing infiltration and inflow, reducing per capita wastewater generation, or directing development to areas with excess capacity.

Infiltration and Inflow

Average daily infiltration and inflow in 2000 was estimated to be 7.2 mgd or approximately 17% of the total estimated wastewater flow.

Table 5-1 compares the municipal and sanitary district wastewater generation from MMSD records to their water sales from water utility reports to the Public Service Commission. It is expected that the ratio of wastewater to water sales would be less than 1, because some water uses do not contribute to wastewater, these include; lawn and garden watering, swimming pools, cooling towers, etc. A wastewater to water sales ratio of more than 1 indicates a problem with infiltration and inflow in that community, unless there are a large number of households with private water wells, but public sanitary sewer.

Table 5-1: Comparison of Wastewater Generation to Water Sales

Municipality / Sanitary District	2005 Wastewater (gpd)	2005 Water Sales (gpd)	Ratio Wastewater / Water Sales
City of Fitchburg	1,682,000	2,036,219	0.83
City of Madison	26,447,000	28,064,800	0.94
City of Middleton	1,694,000	2,203,589	0.77
City of Monona	898,000	925,299	0.97
City of Verona	727,000	988,315	0.74
Village of Cottage Grove	583,000	423,512	1.38
Village of Dane	55,000	57,485	0.96
Village of DeForest	623,000	640,414	0.97
Village of Maple Bluff	161,000	232,512	0.69
Village of McFarland	553,000	568,345	0.97
Village of Shorewood Hills	179,000	176,625	1.01
Village of Waunakee	1,243,000	1,240,414	1.00
Morrisonville Sanitary District	48,000	23,562	2.04
Token Creek Sanitary District	54,000	39,310	1.37
Windsor Sanitary District #1	192,000	221,690	0.87

The Village of Cottage Grove, Village of Shorewood Hills, Village of Waunakee, Morrisonville Sanitary District, and Token Creek Sanitary District have a wastewater to water sales ratio of 1 or greater. In the case of the Village of Cottage Grove, the difference is attributed to the Hydrite groundwater barrier project has pumped approximately 150,000 gpd of contaminated groundwater into the MMSD collection system since the fall of 2003. MMSD may wish to follow up with these communities regarding their municipal collection system televising and inspection programs to verify if infiltration is a problem and to encourage corrective measures to reduce clear water inputs into the collection system.

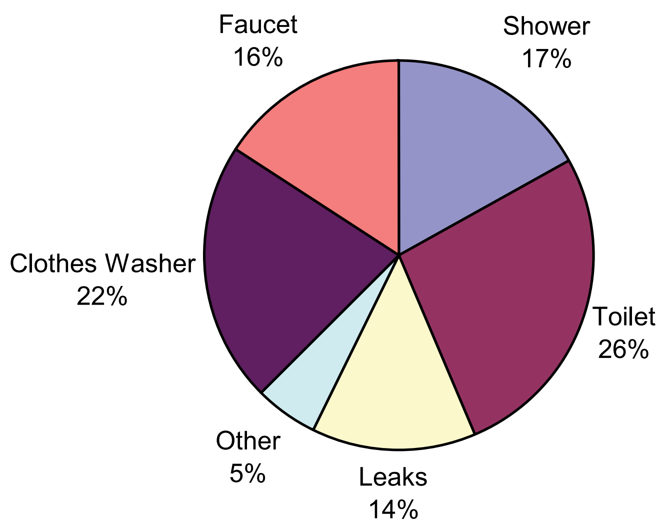
Demand-Side Management

A comprehensive evaluation and discussion of a demand side management program to reduce wastewater generation at the source could be the subject of an entire report alone. The information presented here is intended only to provide an introduction to the potential reductions in wastewater generation from a demand side management program.

Many power and water utilities have a demand-side management program to encourage conservation as a mechanism to help postpone or avoid the need for additional capacity. MMSD may wish to consider implementing a similar program. Implementation of a demand side management program could be either alone or in conjunction with local water utilities.

A breakdown of typical residential indoor water use in the United States is shown in Figure 5-1. The two largest water uses are for flushing toilets and washing clothes.

Figure 5-1: Breakdown of US Residential Indoor Water Use



Source: American Water Works Association Research Foundation, "Residential End Uses of Water", 1999

A study by the University of Arizona Water Resources Research Center documented the history of toilet water use. During the 20th century, the toilet was engineered to use progressively less water. Flush volumes declined over time in the U.S. from more than 7 gallons in early models, to five gallons per flush for much of the mid-20th century. By the 1980's, the standard in the U.S. was 3.5 gallons per flush. By 1992, 1.6 gallons per flush was the standard nationally. The study also reported that the life span of a toilet its typically 20 years. Based on this 20-year life span, it is likely that the majority of toilets within the MMSD service area that use 3.5 gallons per flush or more have already been replaced with 1.6 gallon per flush models, or are likely to be replace within the next 4 years.

Currently, dual flush toilets are not widely used in the MMSD area, but they are becoming more available. These toilets typically use only 0.8 gallons per flush for liquid waste. The potential wastewater reduction that can be achieved by installing a dual flush toilet is estimated to be 4.8 to 7.2 gallons per day per household.[6]

A study by the Oak Ridge National Laboratory determined that a front load washer reduces average water consumption by 15 gallons per load. The potential wastewater reduction that can be achieved by installing a front load washer is estimated to be 8.5 gallons per day per household[7].

The installation of dual flush toilets and front load washers together has the potential to reduce average daily household wastewater generation by 1.8 to 2.1 mgd, based on the number of households in the MMSD service area in 2000.

Targeting large wastewater generators or areas where the collection system is marginally close to capacity may further increase the cost to benefit ratio of a demand side management program.

Excess Capacity Areas

The portions of the collection system and corresponding sub-basins that are projected to have at least 25% of their capacity remaining by 2060 are classified as excess capacity areas. This does not include areas that have excess capacity upstream, but are capacity restricted further downstream. Therefore capacity in the collection system is ultimately restricted by the capacity of the force mains entering the wastewater treatment plant as shown in Table 5-2.

Table 5-2: Projected Capacity of Force Mains to NSWTP

Force Main	Projected Capacity
Pumping Station 11 Force Main	Capacity reached 2020 – 2030
Pumping Station 8 Force Main	75% of capacity in 2060
Pumping Station 2/3/4 Force Main	90-100% of capacity in 2060
Pumping Station 7 Force Main	Capacity reached 2020 – 2030

The only force main entering the wastewater treatment plant that is projected to have excess capacity in 2060 is from Pumping Station 8. The only sub-basin within the Pumping Station 8 service area that has excess capacity in 2060 and is not restricted further down stream is sub-basin 8-W.

If higher velocities and pressures were acceptable, resulting in a higher capacity rating for the Pumping Station 2/3/4 force main, then sub-basins 1-C, 1-D, 1-E, 2-D, 2-E, 2-F, and 2-H would also have excess capacity in 2060.

[6] Based on 3 people per household, 3-4 flushes per person per day, 1 flush per person per day @ 1.6 gallons and 2-3 flushes per person per day @ 0.8 gallons.
[7] Based on an average of 4 loads per week.

Appendix A2
Condition Assessment for Sewage Pumps at MMSD Stations

Appendix A2
Condition Assessment for Sewage Pumps at MMSD Stations
Madison Metropolitan Sewerage District
November, 2010

Outline

This document is organized into the following sections:

- Introduction
- Pump Information
  - Pump Data
- Condition Assessment
  - Evaluation Criteria
  - Observations
  - Availability of Spare Parts
  - Inspections of Pumps
- Pump Ratings
  - Qualitative Analysis
  - Maintenance Costs
- Conclusions and Recommendations
- Attachments

Introduction

The 2002 *Collection System Facilities Plan* contained an assessment of the condition of the sewage pumps at the District's 17 pumping stations. Since 2002 the District has replaced and rehabilitated a number of its pumping units through two major construction projects and through maintenance projects completed by District staff. This appendix serves to update the changes that have taken place since 2002, evaluate current maintenance practices for the pumping units, and provide recommendations for future operation and maintenance of pumps.

Pump Information

Pump Data

The District has 57 sewage pumps currently in service throughout its 17 pumping stations. A listing of the pumps and their attributes are shown in Table 1 (Pumps at District Stations) as an attachment to this document.

The District has seven brands of pumps at its stations as shown in Table 2. Of the 57 raw sewage pumps in the collection system, slightly less than half are Fairbanks Morse units (47%). The second most common brand is Allis Chalmers with 11 units (19%). The Fairbanks Morse and Allis Chalmers brands make up 67% of all District pumps. Since 2002 the District has added pumps manufactured by Cornell and Flygt to its collection system and removed pumps manufactured by American Well (PS9) and Dayton Dowd (PS1).

Table 2 - District Pumping Units by Manufacturer

Manufacturer	Number of Pumps	Locations of Pumps
Allis Chalmers	11	PS's 4, 11, 12, 14, 15
Cornell	9	PS1, PS2, PS10
Fairbanks Morse	27	PS's 1, 3, 6, 7, 8, 9, 11, 12, 13, 15
Flygt	1	PS14
Goulds	3	PS17
Patterson	3	PS 5
Worthington	3	PS16
Total	**57**	

Due to major pump replacement projects at Pumping Stations 1, 2, 6, 8, and 10 in the last five years the average age of the District's pumping units has decreased significantly. This can be seen in Table 3 and in Figures 1 and 2 (see attachments).

Table 3 - Pump Ages

	In Year 2000	In Year 2010
Average age (yrs)	29	21
Median age (yrs)	30	18
Minimum age (yrs)	4	0
Maximum age (yrs)	63	60

Pumps with 60 years of service at this time include Pumps 1C, 1D, 7A, and 11A. Pumps 1C and 1D are now used to transfer flow from PS1 to PS6 on a periodic basis and have limited run hours during normal operation. The motors for these pumps were rewound in 2005 as part of the PS1 rehabilitation project. Pumps 7A and 11A handle average daily flows and as a result have very high run hours. They are good examples of Fairbanks Morse pumps that provide excellent reliability and endurance despite their age. Pump 7A was rehabilitated in 2009.

Run times on pumps vary widely across the collection system. Table 4 provides a listing of age, runtime, and condition for all sewage pumps at District stations (see attachments). The higher capacity pumps at many of the stations have low run time hours as expected.

In addition, the new pumps at PS6 and PS8 have low run times due to their replacement in 2009 and 2010. Table 5 provides a summary of pump run times in year 2010 compared to year 2000. Figures 3 and 4 in the Appendix provide additional information on the distribution of pump run times.

Table 5 - Pump Run Times

	As of January 2000	As of October 2010
Average run time (hrs)	39,273	24,420
Median run time (hrs)	10,388	12,634
Minimum run time (hrs)	65 (Pump 15C)	118 (Pump 6B)
Maximum run time (hrs)	234,695 (Pump 1A)	192,931 (Pump 4A)

Like the average pump age, the average pump run time has decreased over the last ten years due to the major pump replacement projects at PS 1, 2, 6, 8, and 10.

Condition Assessment

Evaluation Criteria

Key components in developing and maintaining a pump condition assessment program include the evaluation of a pump's performance and subsequent determination of its service life. A number of criteria need to be considered in this evaluation, including (1). Age of the pump; (2). Pump run hours; (3). Availability of parts; (4). Evidence of volute/casing wear; and, (5). Maintenance history.

It should be noted that capacity is not an appropriate criterion for evaluation as the focus is on the integrity of the pumping units themselves and not on system concerns such as required capacity. Capacity is an overriding consideration and if it is inadequate and can't be increased sufficiently by installing a larger impeller, the pump will have to be replaced even if it is in excellent condition. Pump station capacity considerations are dealt with elsewhere in the collection systems facilities planning effort.

Observations

The criteria cited in the preceding section were discussed with the District's Mechanical Maintenance Department and the following points reflect the collective thoughts of the mechanical maintenance group:

- Sewage pumps are robust units and can have a very long service life if they are well maintained and if there are no particular problems with a pump.
- Age alone is not a good criterion of a pump's performance. The District has pumps that are 60 years old that are still performing satisfactorily. Parts are readily available for the District's Fairbanks Morse pumps, despite the fact that some of these pumps are 60 years old.

- Many parts on a pump are replaceable as they wear (bearings, shafts, impellers, wear rings, seals, etc.). Excluding the impeller, all these parts can be made or obtained without going through the manufacturer. Thus, even if the manufacturer goes out of business new parts can be obtained. Impellers deserve special consideration since there are fewer sources for these parts. If the pump and impeller are still being manufactured then there is a source for replacement impellers. If the original manufacturer has gone out of business there may still be replacement impellers available through another source. In the event that there is no source for the impeller, the pump would have to be replaced when a new impeller is needed.
- Wear on a volute or casing could make a pump unreliable or perhaps so inefficient that it should be replaced. The best method to check this wear is to inspect pumps for excessive wear and to check the pump capacities after determining that the impellers, wear rings, wear ring clearances, and other efficiency related components are in good order.
- Motors are long-lived, have few problems, and are repairable or replaceable when problems occur. Consequently, a motor in poor condition would not generally be a reason to replace an entire pumping unit. Efficiency and voltage issues may lead to a decision to change motors even if the motor is in good condition.
- Pumps driven by vertical, extended drive shafts require more maintenance than pumps with shorter drive shafts. In general the use of extended vertical drive shafts should be avoided in future designs.
- Pump plugging with rags and other stringy material has been a chronic problem since 2006 when the bar screens were removed at the four large stations pumping to the treatment plant. The problem has been particularly noteworthy at PS7 and PS11. This issue is discussed in more detail in Chapter 3 of this Facilities Plan.
- Technological improvements in pump control systems have led to greater operational flexibility and have extended the life of associated electrical equipment. These improvements include adjustable frequency drives, programmable logic controllers, motor soft starters, bearing temperature sensors, and vibration sensors. While these improvements are a net benefit to the overall performance of the pumping system, their complexity can make it more difficult to troubleshoot and correct problems with a pump or the operation of the overall pumping system.

Two points are worthy of further consideration and discussion: (1). Availability of spare parts, and (2). Internal inspections of pumps.

Availability of spare parts

Spare parts are readily available for most of the District's pumps since most of the pumps are still being manufactured. All of the Fairbanks Morse, Flygt, Cornell, Allis Chalmers, Goulds, Patterson, and Worthington pumps fall under this category. The full line of parts including impellers, bearing frames, casings and other cast parts are still available. This includes the Fairbanks Morse pumps installed in the 1950's. While some parts manufactured today may be slightly different than the original parts, the new parts generally still fit and work as replacement parts.

Table 6 is a summary of the primary vendors used by the District for replacement parts for each of the different pump manufacturers.

Table 6 - Suppliers of Spare Parts

Pump Manufacturer	Vendor(s)	Location
Allis Chalmers	ITT Flygt Corporation	Pewaukee, WI
	RDM Municipal Supply & Service, Inc.	Oak Creek, WI
Cornell	Cornell Pump Company	Clackamas, OR
	USEMCO	Tomah, WI
	Crane Engineering Sales, Inc.	Kimberly, WI
Fairbanks Morse	L.W. Allen, Inc.	Madison, WI
	ABBA Parts & Service	Burlington, ON (Canada)
Flygt	ITT Flygt Corporation	Pewaukee, WI
Goulds	Energenecs	Cedarburg, WI
	First Supply	Madison, WI
	Crane Engineering Sales, Inc.	Kimberly, WI
Patterson	Thomas Pump Company	Aurora, IL
Worthington	Furey Filter & Pump, Inc.	Germantown, WI

As can be seen in Table 6, most of the vendors supplying pump parts are located in southern Wisconsin. As a result, there are not significant shipping delays in most instances. Some problems have been experienced with the acquisition of parts for the Cornell pumps, whose parent company is located in the state of Oregon.

Another complicating factor in obtaining spare parts in some cases is the dissolution or consolidation of suppliers and/or manufacturers. ITT Industries purchased the Goulds and Allis Chalmers companies some time ago and replaced the brand name “Allis Chalmers” with the brand name “A-C Pump.” ITT subsequently sold the former Goulds (PS 17 pumps) dry pit sewage pump line to the Yeomans Chicago Corporation. Yeomans Chicago Corporation now markets the former Goulds sewage pump line as part of its Morris Pumps division. The local representative for Morris Pumps is Energenecs in Cedarburg, Wisconsin. As the Goulds brand has been sold on several occasions, it has been difficult to obtain timely and valuable technical support for the problematic pumps at PS17.

Parts for the District’s two most common pumps, Fairbanks Morse and Allis Chalmers, can be obtained through local suppliers in most instances. An alternative source of parts for these pumps is ABBA Pump Parts and Service headquartered in Ontario, Canada.

ABBA can make virtually any part needed for pumps including bearing frames, impellers, casings and other cast parts if an old sample part can be provided as a model.

For impellers, ABBA takes dimensions off an old worn one, and, if a performance curve is available for the original impeller, a computer program can be used to design a replacement impeller to match the performance of the original impeller. Maximum delivery time for almost any part is four months (much less than that for items they already have patterns for). These "special" parts will, of course, be costly, but the ability to have parts made is an alternative to installing an entirely new pump if capacity is not an issue.

In short, parts can be obtained for any of the pumps the District owns, even for those pumps that are no longer manufactured.

Inspections of Pumps

The District does not have a formal program for routine inspection of internal surfaces and components. Thus, it is difficult to predict how much wear or corrosion there may be on impellers, wear rings, and casings. Internal inspections have typically been done only when there is evidence of a problem or as part of other required maintenance such as bearing replacement or unplugging of pumps. In general, inspections are not scheduled due to staffing and workload issues.

As the District has implemented improved maintenance practices over the years such as the use of mechanical seals and better alignment of pumps, overhauls of pumps to replace worn shaft sleeves and worn bearings have become less frequent. Conversely, the removal of bar screens at the major pumping stations has resulted in increased pump plugging at these stations since 2006. Removal of rags from pumps has provided an opportunity to inspect pump internals, although not all pumps are inspected at the same frequency. In reviewing work orders for 2010, it was found that approximately 7% of the mechanical maintenance staff's time was spent unplugging pumps at District and non-District stations. Due to staffing constraints, the percentage of daily staff time spent unplugging pumps needs to decrease before a scheduled program for internal inspection of pumps can be implemented.

Even without the aid of formalized inspections, several technological advances in the last ten years have allowed District staff to better predict declining performance in pumps and/or mitigate pump wear. Vibration sensors have been installed on pumps at PS 1, 2, 6, 8, and 10 since 2005. These sensors have proved useful in developing trends to detect unusual vibrations.

Limit switches are being installed on check valves in conjunction with the rehabilitation or replacement of pumps. These switches monitor check valve status during pump start-up and run cycles. If the check valve doesn't open within a period of time or fails to stay open during the run cycle, the pump will fail, shut off, and alarm. This prevents unnecessary wear on the pump in cases that it is running but might not be pumping any liquid.

Finally, the installation of magnetic or venturi flowmeters on the discharge of new or rebuilt pumps has provided additional information on the performance of these units.

Recorded reductions in flow can be used to investigate pump problems before they may otherwise be noticed.

Pump Ratings

Qualitative Analysis

Pump condition ratings, as provided by the Mechanical Maintenance Department in 2010, are shown in Table 7. A three level rating system was used to qualitatively assess pump performance (Good, Fair, and Poor). A rating of "Good" implies that the pump, in general, performs as anticipated and does not require any unusual or unexpected maintenance. A rating of "Fair" suggests that the pump requires more maintenance than anticipated, although not extensive. A rating of "Poor" is used to describe those pumps that are not reliable and require frequent attention and/or rehabilitation.

Table 7 - Ratings of District Pumps

Rating	Number of Pumps	Pumps
Good	47	
Fair	6	12A, 15A, 15B, 16A, 16B, 16C
Poor	4	11B, 17A, 17B, 17C
Total	**57**	

It is important to note that the ratings provided in Table 7 reflect the current operating performance of the pumping units. Where some pumps may have been problematic in the past, rebuilding of their internal components has caused them to operate satisfactorily at this time and achieve a "Good" rating. Historical maintenance costs for each pump are discussed in the next section.

Eighty-two percent (47 of 57) of the pumps were rated in good condition overall. Six pumps were rated in fair condition. Pump 12A has nearly 150,000 operating hours and, not surprisingly, requires more maintenance than other pumping units. It was recently rebuilt in 2009 and is operating satisfactorily at this time. Pumps 15A and 15B are Fairbanks Morse pumps that have provided 35 years of service to date. Even though Fairbanks Morse pumps are generally very reliable, the model type for Pumps 15A and 15B is different from other Fairbanks Morse pumps at the District's stations. The pumps at PS16 have performed below expectations in recent years. All three pumps are scheduled to be rebuilt in 2011.

Pump 11B received a poor rating due to recurring problems with the pump shaft. Shafts on this pump were repaired in 2009 and 2010. The most problematic pumps in the District's collection system, however, are those at PS17. These pumps are manufactured by Goulds (Model # NCD 8x8-17). The pumps are driven by vertical shafts and vibrate excessively, causing premature wear and failure of several components. Bearing housings need to be machined frequently and impellers and shafts need to be refitted.

Mechanical seals, shaft sleeves, and wear rings also need frequent replacement due to the excessive wear on these pumps. The aforementioned components are replaced or rebuilt once every three years at present time.

A comparison of pump ratings between 2000 and 2010 is shown in Table 8. The District has been successful in addressing problematic pumps since the last pump condition assessment was performed in 2000. Of the seven pumps rated in fair or poor condition in 2000, all have been replaced as of 2010.

The three pumps at PS16 which are rated in fair condition are to be rebuilt in 2011. Pump station rehabilitation projects are scheduled at PS11, PS12, PS15, and PS17 from 2014 to 2015. These projects will provide an opportunity to replace or rebuild the remaining pumps that are rated in fair or poor condition at this time.

Table 8 - Comparison of Pump Ratings

Rating	In Year 2000		In Year 2010	
	Number	%	Number	%
Good	52	88	47	82
Fair	6	10	6	11
Poor	1	2	4	7
Total	59	100	57	100

Maintenance Costs

In addition to the qualitative rankings provided by the mechanics, the District's asset management program was used to track maintenance costs for each of the 57 pumps now in service in the collection system. Table 9 provides a summary of the labor, material, and service costs associated with each pump during the ten year period from 2001-2010 (see attachments). The total costs during this period are displayed graphically in Figure 5.

The pump with the most extensive maintenance costs over the last ten years is Pump 11B, at over $68,000. This pump was rebuilt in 2007 and 2009 and the shaft was repaired in 2010. The pumps at PS7 also have significant maintenance costs, although this is not surprising given that this station conveys slightly less than one-half of the District's average daily flow. A large portion of the maintenance costs for this station can be attributed to pump plugging, as discussed elsewhere in this appendix.

The pumps at PS17 are the most problematic with regard to cost from an overall pump station perspective. During the last ten years the total cost to service these pumps has been over $115,000. As mentioned previously, each pump requires a full rebuild once approximately every three years due to excessive vibration and premature wear of pump components. The District is currently working on a vibration analysis of these pumps with the pump representative. Given the high annual costs to maintain these pumps, replacement of one or more of the units may be required if a satisfactory solution to the vibration problems cannot be found.

As expected, the maintenance costs for many of the new pumps are minimal as they have been installed within the last five years. An exception to this is the Cornell pumps at PS 1, 2, and 10. Maintenance costs for these pumps are relatively high as problems with bearing failures, vibration, and other difficulties have been experienced during the early years of operation. Some of the maintenance costs shown for these pumps are for the District's labor to remove and reinstall the pumps for warranty work by the manufacturer.

Conclusions and Recommendations

Sewage pumps are robust machines and if well maintained can provide many years of service. The District has numerous pumps in service that are 60 years old and five pumps with more than 100,000 operating hours. The great majority of the pumps are considered to be in good condition and capable of providing many more years of service. As the pump population ages and wastewater flows increase it is expected that pump maintenance needs will also increase. The following observations and recommendations are made with regard to the current and future operation and maintenance of raw wastewater pumps in the District's collection system:

1. The District has implemented various predictive maintenance procedures and/or strategies in the last ten years that have provided valuable information and improved maintenance in general. These procedures and strategies include the following:

    a) Installation of sensors on pump bearing housings to monitor unusual vibrations.

    b) Installation of limit switches on check valves to ensure that pumps do not run dry.

    c) Installation of flowmeters downstream of individual pumping units to provide early indication of declining pump capacity.

    d) Installation of bearing temperature sensors on the pump and motor.

    These measures have been primarily implemented at pumping stations where major rehabilitation work has taken place. It is recommended that these procedures continue to be phased in throughout the collection system as part of future rehabilitation work or scheduled maintenance projects.

2. The plugging of pumps with rags and other stringy material has been a major operational concern at PS7 and PS11 since the bar screens were removed beginning in 2006. A significant amount of time is spent by mechanics in unplugging pumps and repairing pump components. Further, the plugging of pumps hinders overall station reliability, especially during high flow events. This issue should continue to be evaluated in future years, and the re-installation of bar screens should be considered if necessary.

3. In general, spare parts for all of the pumps in operation are readily available from local suppliers and/or manufacturers.

4. The most problematic pumps in the collection system, as determined by the Mechanical Maintenance Department, are found at PS11, PS12, PS15, PS16, and PS17. Rehabilitation projects are scheduled to begin at PS11, PS12, PS15, and PS17 in approximately 2015. Replacement or rehabilitation of the problematic pumping units at these stations should be included in the scope of work for these projects.

5. The pumps at PS17 are especially problematic and have high annual maintenance costs. District staff is currently working with the manufacturer's representative on a vibration analysis for these pumps. If a satisfactory solution to the vibration problems experienced by these pumps cannot be found soon, it may be cost effective to replace these units prior to the scheduled station rehabilitation in 2015.

Attachments

1. Table 1: Pumps at District Pump Stations
2. Table 4: Age, Runtime, and Condition of Sewage Pumps at District Pumping Stations (November 2010)
3. Figure 1: Pump Age in Year 2010
4. Figure 2: Distribution of Pump Ages in Year 2010
5. Figure 3: Pump Run Hours
6. Figure 4: Distribution of Pump Run Hours
7. Table 9: Maintenance Costs for MMSD Pumps (Jan. 2001 to Nov. 2010)
8. Figure 5: Maintenance Costs for MMSD Pumps (Jan. 2001 to Nov. 2010)

Table 1 - Pumps at District Stations

	Asset ID & Manufacturer	Description & Model Number	Drive Type	Flow (gpm)	Head (ft)	Serial Number	Outlet Size (in)	Impeller	Impeller Diameter (in)	Operating Speed (rpm)	Horsepower
PS01											
	PMP0108 Cornell	PS01: Pump A 18NHG34A-F30	VFD	14,100	134	131778	18	18NHG34	28.56	890	600
	PMP0109 Cornell	PS01: Pump B 18NHG34A-F30	VFD	14,100	134	131777	18	18NHG34	28.56	890	600
	PMP0103 Fairbanks Morse	PS01: Pump C 5720	VFD	10,375	31	727677	20	L20A1S	21.75	580	150
	PMP0104 Fairbanks Morse	PS01: Pump D 5720	VFD	12,500	41	727676	20	L20A1S	24.00	585	150
PS02											
	PMP0206 Cornell	PS02: Pump A 18NHG34A-F30	VFD	16,500	108	131770	18	18NHG34	28.56	890	600
	PMP0207 Cornell	PS02: Pump B 18NHG34A-F30	VFD	16,500	108	131775	18	18NHG34	28.56	890	600
	PMP0208 Cornell	PS02: Pump C 18NHG34A-F30	Constant	16,500	108	131773	18	18NHG34	28.56	890	600
	PMP0209 Cornell	PS02: Pump D 18NHG34A-F30	Constant	16,500	108	131774	18	18NHG34	28.56	890	600
PS03											
	PMP0301 Fairbanks Morse	PS03: Pump A B5414	Constant	1,050	60	K3D1-050173-1	5	T5D1CU	13.00	1,175	30
	PMP0302 Fairbanks Morse	PS03: Pump B B5414	Constant	1,050	60	K3D1-050173-2	5	T5D1CU	13.00	1,175	30
PS04											
	PMP0401 Allis Chalmers	PS04: Pump A Model 150, 10 x 8 x 17 Type NSW	Constant	2,000	47	1-5279-80811-2-1	8	52-216-465	16.25	860	40
	PMP0402 Allis Chalmers	PS04: Pump B Model 150, 10 x 8 x 17 Type NSW	Constant	2,900	95	1-5279-80811-1-2	8	52-216-465	17.00	1,160	100
	PMP0403 Allis Chalmers	PS04: Pump C Model 150, 10 x 8 x 17 Type NSW	Constant	2,900	95	1-5279-80811-1-1	8	52-216-465	17.00	1,160	100
PS05											
	PMP0501 Patterson	PS05: Pump A NCSVF-4, 6 x 6 x 14.5	Constant	1,800	75	NC-C000889-03	6	D-5873	14.50	1,256	50
	PMP0502 Patterson	PS05: Pump B NCSVF-4, 6 x 6 x 14.5	Constant	1,800	75	NC-C000889-2	6	D-5873	14.50	1,256	50
	PMP0503 Patterson	PS05: Pump C NCSVF-4, 6 x 6 x 14.5	Constant	1,800	75	NC-C000889-1	6	D-5873	14.50	1,256	50

Table 1 - Pumps at District Stations

	Asset ID & Manufacturer	Description & Model Number	Drive Type	Flow (gpm)	Head (ft)	Serial Number	Outlet Size (in)	Impeller	Impeller Diameter (in)	Operating Speed (rpm)	Horsepower
PS06											
	PMP0606 Fairbanks Morse	PS06: Pump A B5721	VFD	7,700	45	176063-1-0	14	L14A1A	14.00	890	125
	PMP0607 Fairbanks Morse	PS06: Pump B B5721	Constant	7,700	45	1760631-1	14	L14A1A	14.00	890	125
	PMP0608 Fairbanks Morse	PS06: Pump C B5721	Constant	7,700	45	1760631-2	14	L14A1A	14.00	890	125
	PMP0609 Fairbanks Morris	PS06: Pump D B5721	Constant	7,700	45	1760631-3	14	L14A1A	14.00	890	125
PS07											
	PMP0701 Fairbanks Morse	PS07: Pump A 5720	Constant	11,500	47	729155	20	L20C1D	24.00	695	250
	PMP0702 Fairbanks Morse	PS07: Pump B C 5721, Size 20	Constant	15,200	53	K3X1-071561	20	L20A1CT	22.50	705	250
	PMP0703 Fairbanks Morse	PS07: Pump C C 5721, Size 20	Constant	19,400	59	K3X1-071560-0	20	L20A1CT	24.00	705	350
	PMP0704 Fairbanks Morse	PS07: Pump D C 5721, Size 20	Constant	19,400	59	K3X1-071560-1	20	L20A1CT	24.00	705	350
PS08											
	PMP0806 P-8A (Formerly Fairbanks Morris	PS08: Pump A 5722	VFD	12,800	58	505931	20	L20C1A	30.00	585	250
	PMP0808 (Formerly 8D) Fairbanks Morris	PS08: Pump B 5722	VFD	12,800	58	505932	20	L20C1A	30.00	585	250
	PMP0809 (Formerly 6C) Fairbanks Morse	PS08: Pump C 5721S	Constant	13,900	60	505933	20	L20A1AV	24.00	705	300
	PMP0810 (Formerly 6D) Fairbanks Morse	PS08: Pump D 5721S	Constant	13,900	60	505934	20	L20A1AV	24.00	705	300
PS09											
	PMP0905 Fairbanks Morse	PS09: Pump A 5434S-T40	Constant	2,300	51	1001739	8	T8D1A	8.00	1,185	40
	PMP0906 Fairbanks Morse	PS09: Pump B 5434S-T40	Constant	2,300	51	1507392	8	T8D1A	8.00	1,185	40
	PMP0904 Fairbanks Morse	PS09: Pump C 5430	Constant	2,300	51	481873	8	T8D1A	8.00	1,185	40

Table 1 - Pumps at District Stations

	Asset ID & Manufacturer	Description & Model Number	Drive Type	Flow (gpm)	Head (ft)	Serial Number	Outlet Size (in)	Impeller	Impeller Diameter (in)	Operating Speed (rpm)	Horsepower
PS10											
	PMP1008 Cornell	PS10: Pump A 18NHG34A-F30	VFD	18,900	94	131771	18	18NHG34	28.56	890	600
	PMP1009 Cornell	PS10: Pump B 18NHG34A-F30	VFD	18,900	94	131772	18	18NHG34	28.56	890	600
	PMP1010 Cornell	PS10: Pump C 18NHG34A-F30	Constant	18,900	94	131776	18	18NHG34	28.56	890	600
PS11											
	PMP1101 Fairbanks Morse	PS11: Pump A 5720	Constant	6,400	43	729252	16	L16A1K	17.00	860	125
	PMP1102 Allis Chalmers	PS11: Pump B Model 150, 16 x 16 x 20 Type NSY	Constant	9,100	49	821-37489-1-3	16	---	18.50	880	150
	PMP1103 Allis Chalmers	PS11: Pump C Model 150, 20 x 20 x 25 Type NSY	Constant	13,300	57	821-37489-3-1	20	---	23.25	705	250
	PMP1104 Allis Chalmers	PS11: Pump D Model 150, 20 x 20 x 25 Type NSY	Constant	13,300	57	821-37489-3-2	20	---	23.25	705	250
PS12											
	PMP1201 Fairbanks Morse	PS12: Pump A 5425 - 10"	Constant	3,400	44	K2N1 053104	10	TALE5AK	20.63	700	50
	PMP1202 Fairbanks Morse	PS12: Pump B 5720 - 16"	Constant	7,200	48	K2N1 053105	16	L16A1G1	17.63	885	100
	PMP1203 Allis Chalmers	PS12: Pump C Model 150, 16 x 16 x 20 Type NSY	Constant	9,000	48	821-37489-1-1	16	---	18.50	880	150
	PMP1204 Allis Chalmers	PS12: Pump D Model 150, 16 x16 x20 Type NSY	Constant	9,000	48	821-37498-1-2	16	---	18.50	880	150
PS13											
	PMP1307 Fairbanks Morse	PS13: Pump A B5721	Constant	8,200	16	1550432	16	L16A1G	18.70	585	50
	PMP1302 Fairbanks Morse	PS13: Pump B 5720 - 16"	Constant	8,200	16	K2P1-055130	16	L16A1G	19.04	585	50
	PMP1303 Fairbanks Morse	PS13: Pump C 5720 -20"	Constant	14,000	20	K2P1-055131	20	L20A1AV	23.38	505	100
PS14											
	PMP1407 Flygt	PS14: Pump A Model 150 16 X 16 X 20 NSY	Constant	7,200	24	1086076204	16	P2689-2	17.38	705	60
	PMP1402 Allis Chalmers	PS14: Pump B Model 150, 16 x 16 x 20 Type NSY	Constant	7,200	24	1-97191-2-1	16	P2689-2	17.38	695	60
	PMP1403 Allis Chalmers	PS14: Pump C Model 150, 18 x 18 x 23 Type NSY	Constant	10,800	29	1-97191-3-1	18	---	22.00	585	100

Table 1 - Pumps at District Stations

	Asset ID & Manufacturer	Description & Model Number	Drive Type	Flow (gpm)	Head (ft)	Serial Number	Outlet Size (in)	Impeller	Impeller Diameter (in)	Operating Speed (rpm)	Horsepower
PS15											
	PMP1501 Fairbanks Morse	PS15: Pump A 5425C - 10"	Constant	4,000	76	K2V1-073520-1	10	TALE5BB	21.00	885	100
	PMP1502 Fairbanks Morse	PS15: Pump B 5425C - 10"	Constant	3,000	68	K2V1-073520-0	10	TALE5A	19.06	885	100
	PMP1503 Allis Chalmers	PS15: Pump C Model 150, 14 x 12 x 20 Type NSM	Constant	6,100	100	821-37489-5-1	12	---	23.00	885	200
PS16											
	PMP1601 Worthington	PS16: Pump A 12 MN 24	Constant	7,000	182	81ZUS8254-2	12	----	22.05	1,185	500
	PMP1602 Worthington	PS16: Pump B 12 MN 24	Constant	7,000	182	81ZUS8254-3	12	----	22.05	1,185	500
	PMP1603 Worthington	PS16: Pump C 12 MN 24	Constant	7,000	182	81ZUS8254-1	12	----	22.05	1,185	500
PS17											
	PMP1701 Goulds	PS17: Pump A NCD 8 x 8 - 17	VFD	2,300	115	M95065A0-01	8	52262	16.50	1,290	100
	PMP1702 Goulds	PS17: Pump B NCD 8 x 8 - 17	VFD	2,300	115	M95065A01-02	8	52262	16.50	1,290	100
	PMP1703 Goulds	PS17: Pump C NCD 8 x 8 - 17	VFD	2,300	115	M95065A01-03	8	52262	16.50	1,290	100

Table 4 - Age, Runtime, and Condition of Sewage Pumps at District Stations

Pump	Manufacturer	Flow (gpm)	Head (ft)	Outlet Size (in	Year Installed	Age in Year 2010 (yrs)	Runtime in Year 2000 (hrs)	Runtime from 2000 to 2010 (hrs)	Total Runtime to Date (hrs)	Condition	Comments
1A	Cornell	14,100	134	18	2005	5	0	22,174	22,174	Good	
1B	Cornell	14,100	134	18	2005	5	0	18,001	18,001	Good	
1C	Fairbanks Morse	10,375	31	20	1950	60	21,353	1,378	22,731	Good	Motor rewound in 2005.
1D	Fairbanks Morse	12,500	41	20	1950	60	3,231	677	3,908	Good	Motor rewound in 2005.
2A	Cornell	16,500	108	18	2005	5	0	28,885	28,885	Good	
2B	Cornell	16,500	108	18	2005	5	0	15,722	15,722	Good	
2C	Cornell	16,500	108	18	2005	5	0	348	348	Good	
2D	Cornell	16,500	108	18	2005	5	0	483	483	Good	
3A	Fairbanks Morse	1,050	60	5	1980	30	24,152	11,913	36,065	Good	
3B	Fairbanks Morse	1,050	60	5	1980	30	21,247	11,054	32,301	Good	
4A	Allis Chalmers	2,000	47	8	1967	43	131,906	61,025	192,931	Good	
4B	Allis Chalmers	2,900	95	8	1967	43	2,564	124	2,688	Good	Impeller diameter increased from 16.25" to 17.00" in 2005.
4C	Allis Chalmers	2,900	95	8	1967	43	1,176	65	1,241	Good	Impeller diameter increased from 16.25" to 17.00" in 2005.
5A	Patterson	1,800	75	6	1996	14	4,535	9,978	14,513	Good	
5B	Patterson	1,800	75	6	1996	14	5,501	17,216	22,717	Good	
5C	Patterson	1,800	75	6	1996	14	11,324	9,885	21,209	Good	
6A	Fairbanks Morse	7,700	45	14	2009	1	0	544	544	Good	
6B	Fairbanks Morse	7,700	45	14	2009	1	0	118	118	Good	
6C	Fairbanks Morse	7,700	45	14	2009	1	0	355	355	Good	
6D	Fairbanks Morse	7,700	45	14	2009	1	0	274	274	Good	
7A	Fairbanks Morse	11,500	47	20	1950	60	103,408	31,789	135,197	Good	
7B	Fairbanks Morse	15,200	53	20	1992	18	32,924	27,232	60,156	Good	
7C	Fairbanks Morse	19,400	59	20	1992	18	5,378	7,256	12,634	Good	
7D	Fairbanks Morse	19,400	59	20	1992	18	3,475	6,149	9,624	Good	
8A	Fairbanks Morse	12,800	58	20	2010	0	0	2,190	2,190	Good	Formerly Pump 8C. Completely rebuilt, including new motor, in 2010. 250 HP; 600 rpm; 30" impeller; VFD.
8B	Fairbanks Morse	12,800	58	20	2010	0	0	2,523	2,523	Good	Formely Pump 8D. Completely rebuilt, including new motor, in 2010. 250 HP, 600 rpm; 30" impeller; VFD.
8C	Fairbanks Morse	13,900	60	20	2010	0	0	1,061	1,061	Good	Formerly Pump 6C (Model 5720). Pump rebuilt and moved to PS8 in 2010 (now Model 5721S). Note that runtime hours reset to zero after rebuild. 300 HP; 710 rpm; 24" impeller; constant speed.

Table 4 - Age, Runtime, and Condition of Sewage Pumps at District Stations

Pump	Manufacturer	Flow (gpm)	Head (ft)	Outlet Size (in	Year Installed	Age in Year 2010 (yrs)	Runtime in Year 2000 (hrs)	Runtime from 2000 to 2010 (hrs)	Total Runtime to Date (hrs)	Condition	Comments
8D	Fairbanks Morse	13,900	60	20	2010	0	0	181	181	Good	Formerly Pump 6D (Model 5720). Pump rebuilt and moved to PS8 in 2010 (now Model 5721S). Note that runtime hours reset to zero after rebuild. 300 HP; 710 rpm; 24" impeller; constant speed.
9A	Fairbanks Morris	2,300	51	8	2003	7	0	4,166	4,166	Good	Pump replaced in 2003 with F-M Vertical Biltogether pump.
9B	Fairbanks Morris	2,300	51	8	2007	3	0	1,766	1,766	Good	Pump replaced in 2007 with F-M Vertical Biltogether pump.
9C	Fairbanks Morris	2,300	51	8	2002	8	0	7,718	7,718	Good	Pump replaced in 2002 with F-M Vertical Biltogether pump.
10A	Cornell	18,900	94	18	2005	5	0	21,177	21,177	Good	
10B	Cornell	18,900	94	18	2005	5	0	25,197	25,197	Good	
10C	Cornell	18,900	94	18	2005	5	0	640	640	Good	
11A	Fairbanks Morse	6,400	43	16	1950	60	98,747	57,993	156,740	Good	
11B	Allis Chalmers	9,100	49	16	1982	28	9,825	12,513	22,338	Poor	Overhauled but has a poor history of snapping shafts.
11C	Allis Chalmers	13,300	57	20	1982	28	152	1,796	1,948	Good	
11D	Allis Chalmers	13,300	57	20	1982	28	79	1,212	1,291	Good	
12A	Fairbanks Morse	3,400	44	10	1969	41	106,828	42,328	149,156	Fair	Pump rebuilt in 2010 by District mechanics.
12B	Fairbanks Morse	7,200	48	16	1969	41	17,398	38,501	55,899	Good	
12C	Allis Chalmers	9,000	48	16	1982	28	396	273	669	Good	
12D	Allis Chalmers	9,000	48	16	1982	28	132	352	484	Good	
13A	Fairbanks Morse	8,200	16	16	2008	2	0	7,266	7,266	Good	Pump replaced in 2008 by Contract.
13B	Fairbanks Morse	8,200	16	16	1970	40	3,658	6,844	10,502	Good	Pump rebuilt in 2008 by Contract. Includes new impeller, impeller wear ring, bearings, mechanical seal and coupling.
13C	Fairbanks Morse	14,000	20	16	1970	40	259	463	722	Good	
14A	Flygt	7,200	24	16	2008	2	0	6,248	6,248	Good	Pump replaced in 2008 by Contract.
14B	Allis Chalmers	7,200	24	16	1971	39	2,281	18,350	20,631	Good	Pump rebuilt in 2008 by Contract. Includes new impeller, impeller wear ring, bearings, mechanical seal and coupling.
14C	Allis Chalmers	10,800	29	18	1971	39	408	269	677	Good	Pump rebuilt in 2008 by Contract. Includes new impeller, impeller wear ring, bearings, mechanical seal and coupling. Impeller diameter increased from 20.50" to 22.00".
15A	Fairbanks Morse	4,000	76	10	1975	35	12,648	1,090	13,738	Fair	
15B	Fairbanks Morse	3,000	68	10	1975	35	82,375	46,962	129,337	Fair	
15C	Allis Chalmers	6,100	100	12	1982	28	65	204	269	Good	
16A	Worthington	7,000	182	12	1982	28	9,639	4,477	14,116	Fair	
16B	Worthington	7,000	182	12	1982	28	8,906	4,492	13,398	Fair	
16C	Worthington	7,000	182	12	1982	28	8,773	4,388	13,161	Fair	

Table 4 - Age, Runtime, and Condition of Sewage Pumps at District Stations

Pump	Manufacturer	Flow (gpm)	Head (ft)	Outlet Size (in	Year Installed	Age in Year 2010 (yrs)	Runtime in Year 2000 (hrs)	Runtime from 2000 to 2010 (hrs)	Total Runtime to Date (hrs)	Condition	Comments
17A	Goulds	2,300	115	8	1996	14	3,250	11,578	14,828	Poor	Vertical arrangement. Numerous vibration/wear issues.
17B	Goulds	2,300	115	8	1996	14	3,250	19,753	23,003	Poor	Vertical arrangement. Numerous vibration/wear issues.
17C	Goulds--Morris	2,300	115	8	1996	14	3,250	10,854	14,104	Poor	Vertical arrangement. Numerous vibration/wear issues.

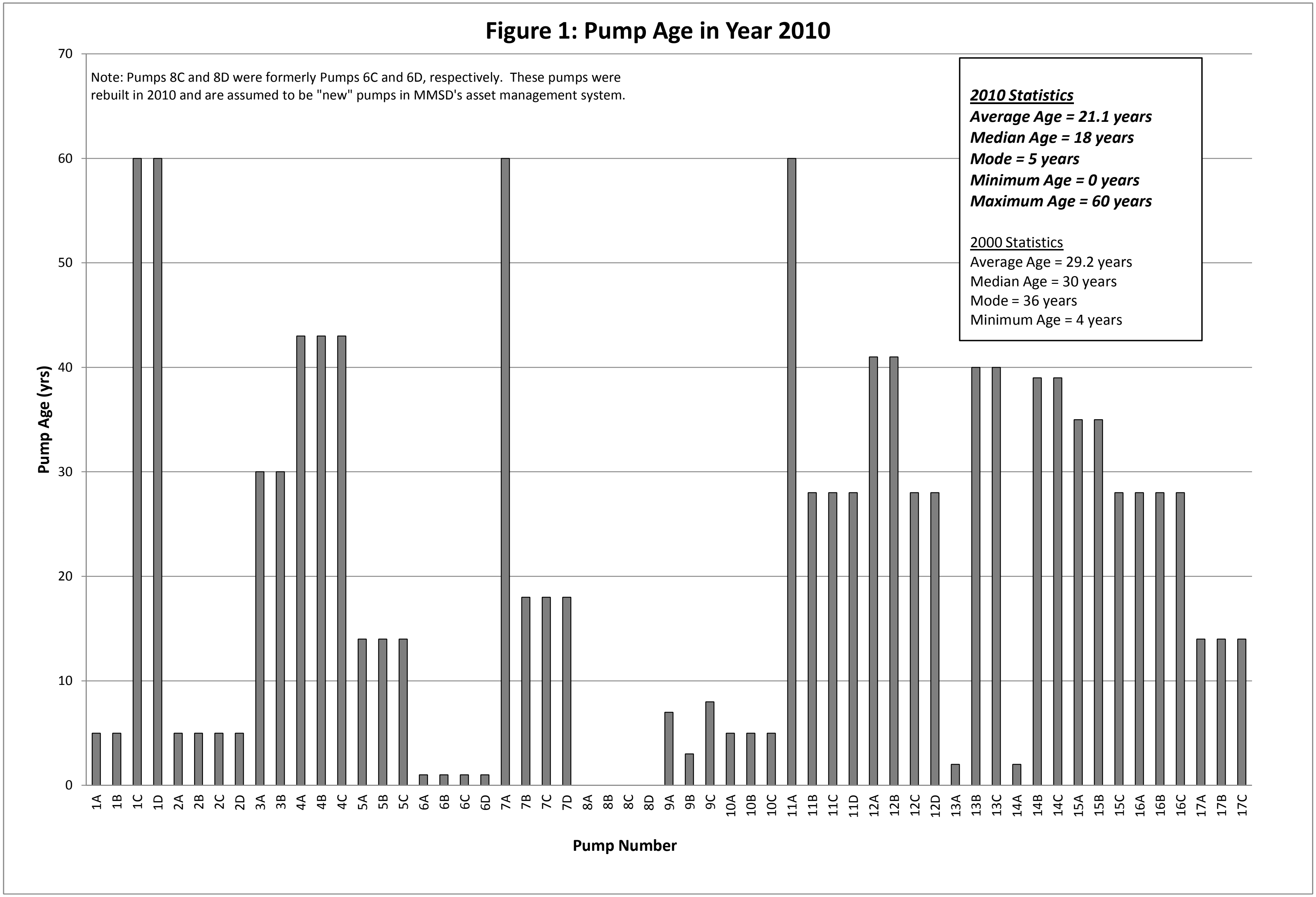


Figure 1: Pump Age in Year 2010
Note: Pumps 8C and 8D were formerly Pumps 6C and 6D, respectively. These pumps were rebuilt in 2010 and are assumed to be "new" pumps in MMSD's asset management system.
2010 Statistics
Average Age = 21.1 years
Median Age = 18 years
Mode = 5 years
Minimum Age = 0 years
Maximum Age = 60 years
2000 Statistics
Average Age = 29.2 years
Median Age = 30 years
Mode = 36 years
Minimum Age = 4 years
Pump Age (yrs)
Pump Number
0
10
20
30
40
50
60
70
1A
1B
1C
1D
2A
2B
2C
2D
3A
3B
4A
4B
4C
5A
5B
5C
6A
6B
6C
6D
7A
7B
7C
7D
8A
8B
8C
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10A
10B
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11A
11B
11C
11D
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17B
17C

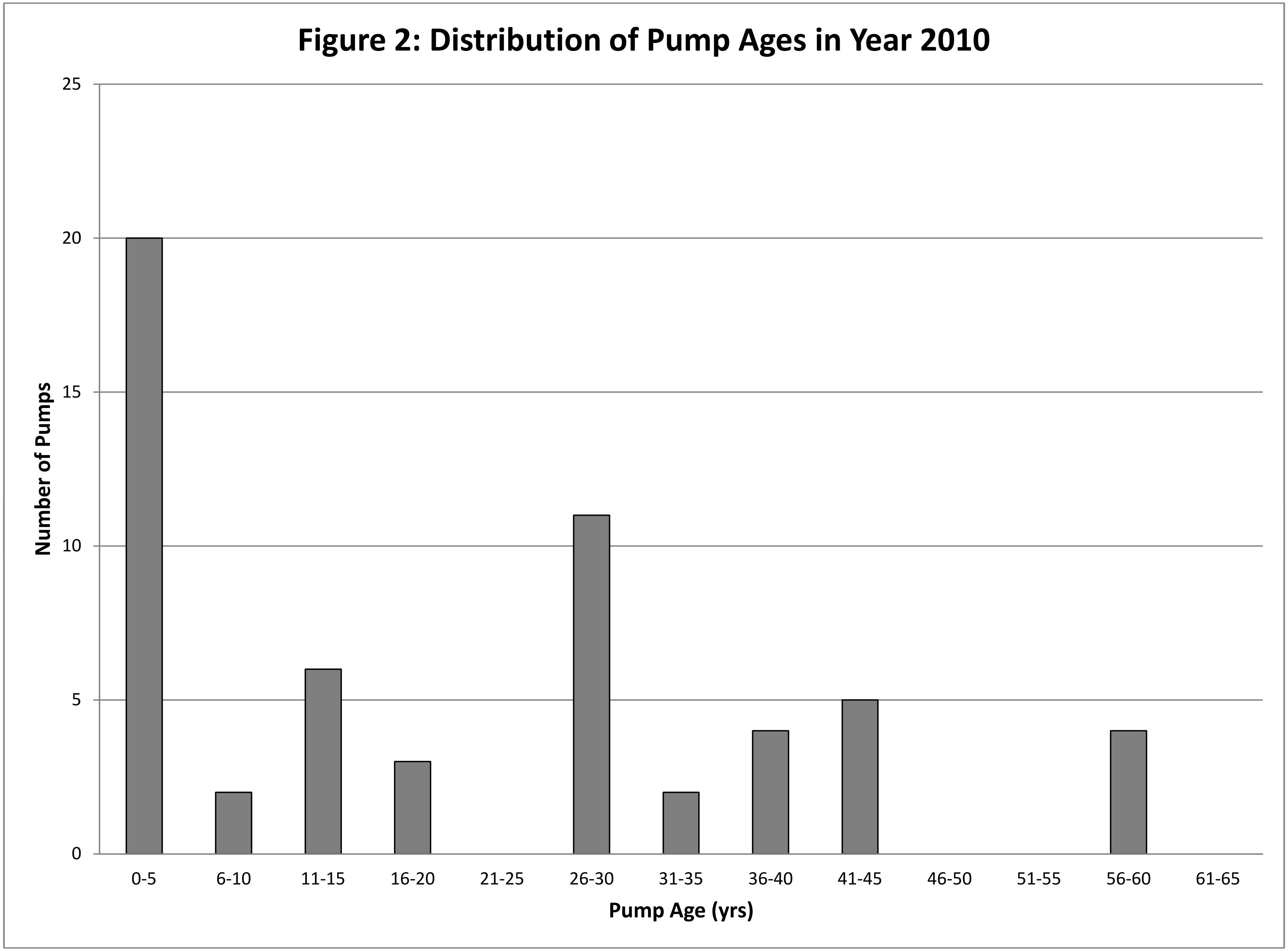
Figure 2: Distribution of Pump Ages in Year 2010
Number of Pumps
0
5
10
15
20
25
0-5
6-10
11-15
16-20
21-25
26-30
31-35
36-40
41-45
46-50
51-55
56-60
61-65
Pump Age (yrs)

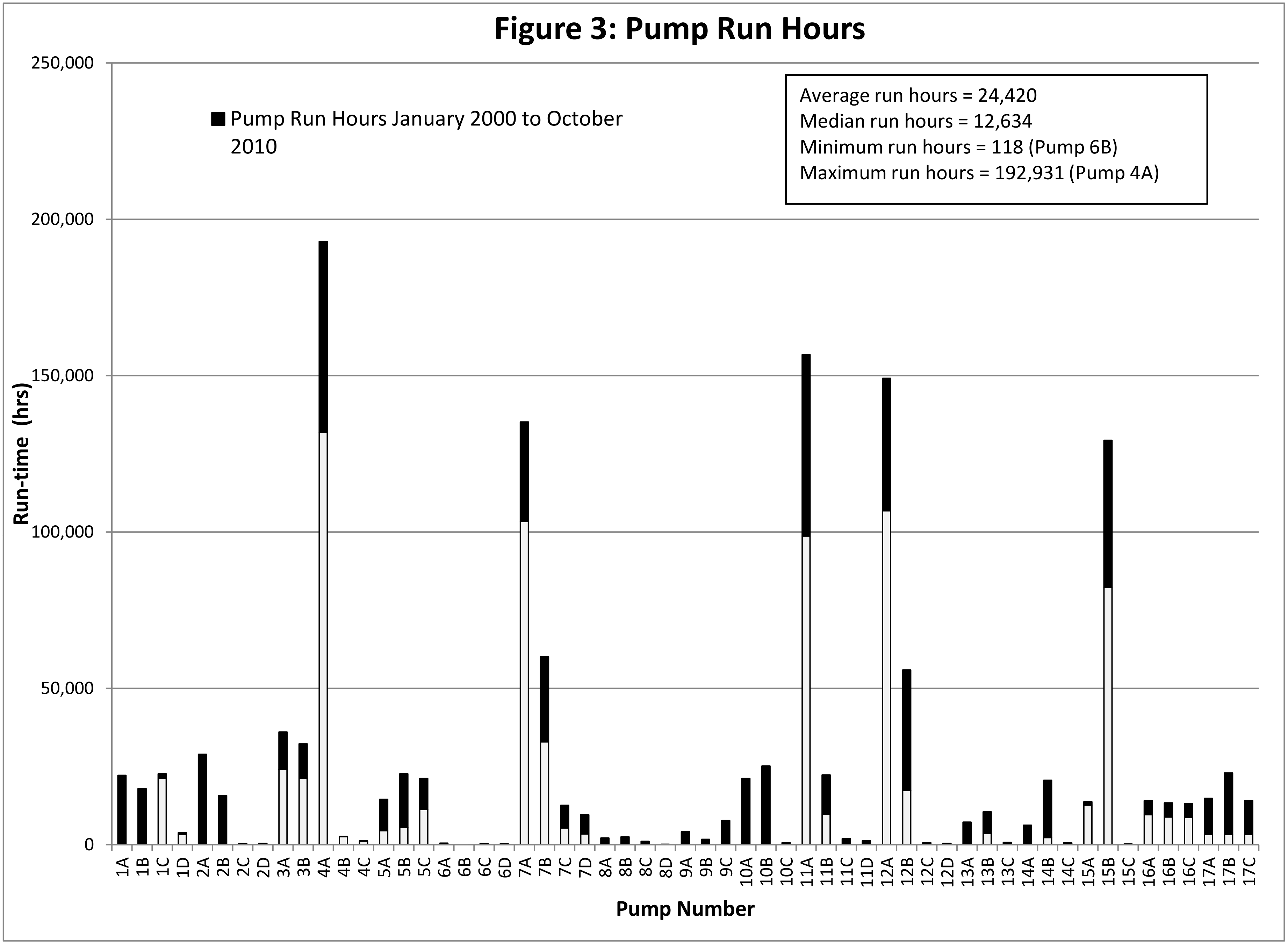

Figure 3: Pump Run Hours
Pump Run Hours January 2000 to October 2010
Average run hours = 24,420
Median run hours = 12,634
Minimum run hours = 118 (Pump 6B)
Maximum run hours = 192,931 (Pump 4A)
Run-time (hrs)
250,000
200,000
150,000
100,000
50,000
0
1A 1B 1C 1D 2A 2B 2C 2D 3A 3B 4A 4B 4C 5A 5B 5C 6A 6B 6C 6D 7A 7B 7C 7D 8A 8B 8C 8D 9A 9B 9C 10A 10B 10C 11A 11B 11C 11D 12A 12B 12C 12D 13A 13B 13C 14A 14B 14C 15A 15B 15C 16A 16B 16C 17A 17B 17C
Pump Number

Figure 4: Distribution of Pump Run Hours

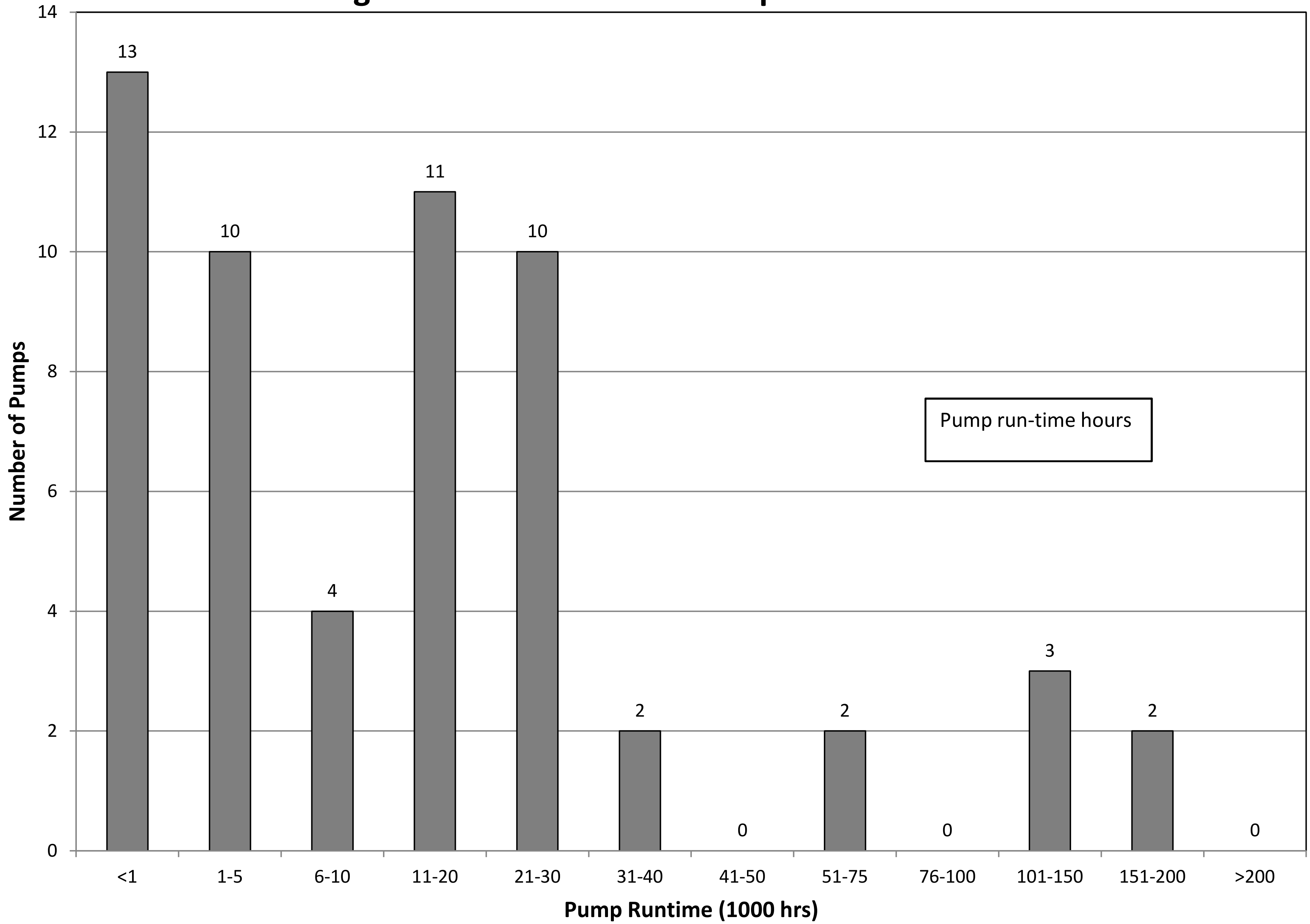

Table 9 - Maintenance Costs for MMSD Pumps (January 2001 to November 2010)

Pump Station	Pump	Labor, Material & Service Costs by Year 2001	2002	2003	2004	2005	2006	2007	2008	2009	2010	Total Costs ($)	Comments
1	1A							$301.26	$66.35	$16,775.67	$136.23	$17,279.51	Rebuilt pump with new bearings and mechanical seal in 2009.
	1B									$121.15	$403.13	$524.28	
	1C			$50.21		$293.45			$6,854.98	$2,846.73	$391.59	$10,436.96	Replaced wear rings on impeller and volute in 2008.
	1D	$219.25			$505.37	$314.36	$183.40			$27.68		$1,250.06	
2	2A								$8,125.14	$929.21	$217.70	$9,272.05	Removed and reinstalled pump for warranty repairs in 2008.
	2B					$121.94	$34.77		$98.19	$101.76		$356.66	
	2C						$362.06				$3,385.43	$3,747.49	Replaced mechanical seal and sleeve in 2010.
	2D								$2,157.61	$1,024.06	$2,745.24	$5,926.91	Replaced mechancial seal in 2010.
3	3A		$211.09		$4,984.90							$5,195.99	Rebuilt pump with new bearings, mechanical seal, sleeve and impeller in 2004.
	3B		$60.21		$4,692.15							$4,752.36	Rebuilt pump with new bearings, mechanical seal, sleeve and impeller in 2004.
4	4A	$138.72				$266.90	$526.48				$87.57	$1,019.67	
	4B	$82.58		$60.17	$4,294.01	$24.17	$76.58					$4,537.51	Replaced impeller in 2004.
	4C			$60.17	$4,121.83							$4,182.00	Replaced impeller in 2004.
5	5A							$192.26		$504.29	$671.38	$1,367.93	
	5B				$567.25		$40.11	$468.46	$3,318.35		$355.81	$4,749.98	Repaired leaking volute in 2008.
	5C	$50.13					$34.77	$115.16	$6,196.89			$6,396.95	Repaired adjustable frequency drive and added motor soft starts in 2008.
6	6A										$211.16	$211.16	
	6B											$0.00	
	6C										$618.55	$618.55	
	6D										$30.93	$30.93	
7	7A					$26.03	$256.00	$134.78	$146.41	$12,544.67		$13,107.89	Rebuilt pump with new bearings, shaft, mechanical seal, and impeller in 2009.
	7B			$93.39			$2,216.43	$12,070.13	$754.97	$18,138.09	$2,026.19	$35,299.20	Replaced wear ring and repaired impeller in 2007. Replaced bearings, sleeve, mechanical seal, and wear rings in 2009. Replaced impeller twice in 2009.
	7C				$146.80		$418.62	$492.94	$2,292.44	$1,334.15	$1,003.50	$5,688.45	
	7D						$1,222.55	$573.32	$617.04	$15,671.64	$6,321.99	$24,406.54	Replaced impeller in 2009.
8	8A										$7.80	$7.80	
	8B											$0.00	
	8C											$0.00	
	8D											$0.00	
9	9A					$155.04	$100.57	$142.55				$398.16	New pump installed in 2004.
	9B									$95.51		$95.51	New pump installed in 2007.
	9C			$38.34			$103.96				$56.46	$198.76	New pump installed in 2002.

Table 9 - Maintenance Costs for MMSD Pumps (January 2001 to November 2010)

Pump Station	Pump	Labor, Material & Service Costs by Year 2001	2002	2003	2004	2005	2006	2007	2008	2009	2010	Total Costs ($)	Comments
10	10A							$220.62	$11,817.19	$6,821.33	$424.95	$19,284.09	Repaired noisy bearing and leaky seal in 2008.
	10B					$259.21		$80.36	$2,914.23	$10,399.12	$6,218.87	$19,871.79	Removed and reinstalled pump for warranty repairs in 2009. Replaced wear rings in 2010.
	10C									$255.45	$662.76	$918.21	
11	11A			$2,691.63			$8,641.19	$230.20	$1,608.00	$259.58	$96.04	$13,526.64	
	11B			$51.08	$8,643.80	$36.05	$620.69	$12,729.53	$9,052.60	$26,913.30	$10,324.66	$68,371.71	Replaced mechanical seal, suction plate, wear rings, and coated impeller in 2007. Rebuilt pump with new shaft, impeller, sleeve, bearings, and seal in 2009. Repaired broken shaft in 2010.
	11C				$85.56	$48.21		$378.35	$3,083.02	$527.46	$1,237.03	$5,359.63	
	11D			$44.13	$382.03	$294.00	$159.10	$139.41	$73.64	$667.03	$1,137.14	$2,896.48	
12	12A				$38.32	$84.33				$543.79	$24,124.66	$24,791.10	Rebuilt pump with new impeller, bearings, and mechanical seal in 2010.
	12B		$3,490.58			$20.54				$644.72		$4,155.84	
	12C	$626.11		$51.90	$34.68	$146.05		$1,158.87	$50.69	$911.01	$2,186.15	$5,165.46	
	12D	$646.14	$191.55	$31.47	$68.62	$24.04	$158.43		$135.32	$81.26	$1,005.07	$2,341.90	
13	13A									$327.93	$483.65	$811.58	New pump installed in 2008.
	13B	$108.61	$76.33	$118.08	$354.57	$331.64	$210.30		$411.33	$1,380.27	$237.85	$3,228.98	Removed and reinstalled pump for warranty work in 2009.
	13C				$68.96	$7,231.02	$1,155.24	$51.43			$76.45	$8,583.10	Rebuilt pump with new bearings, seals, casing ring, and impeller ring in 2005.
14	14A											$0.00	New pump installed in 2008.
	14B			$108.30	$182.97	$377.28	$47.59	$132.81	$186.22			$1,035.17	
	14C			$4,864.89	$275.94	$96.16		$76.93	$53.27		$104.84	$5,472.03	Repaired check valve in 2003.
15	15A			$121.91				$146.57	$8,315.43			$8,583.91	Replaced impeller wear ring in 2008.
	15B					$181.23		$667.47	$11,470.11			$12,318.81	Replaced bearings, mechanical seal, wear rings and rebuilt impeller in 2008.
	15C				$74.71			$13.52	$50.69			$138.92	
16	16A	$4,160.88	$1,533.42	$170.45	$341.05	$20.54	$105.45	$94.86	$416.80	$56.19	$419.52	$7,319.16	Repaired coupling in 2001.
	16B	$32.97	$350.97	$150.22	$102.94	$106.37	$77.67	$57.44	$186.18	$56.19	$25.26	$1,146.21	
	16C	$32.97	$135.54	$235.44	$39.88	$20.54	$45.18	$217.41	$114.25	$196.12	$215.88	$1,253.21	
17	17A					$64.99	$10,842.75	$555.13	$126.64	$3,810.04	$15,444.89	$30,844.44	Rebuilt pump with new bearings and mechanical seal in 2006. Rebuilt pump housing, repaired impeller and shaft, and replaced bearings, seal, and wear rings in 2010.
	17B		$12,513.31	$5,139.42			$18,222.93	$128.58	$19,055.24	$2,456.69	$1,328.96	$58,845.13	Rebuilt pump with new bearings, mechanical seal, impeller and sleeve in 2002. Rebuilt bearing housing and replaced bearings, seal, shaft and impeller wear ring in 2006. Rebuilt pump with new shaft and impeller in 2008.
	17C					$9,904.11	$4,807.10	$128.59	$53.09	$9,952.32	$3,105.30	$27,950.51	Rebuilt pump with new bearings, mechanical seal, grease seals, and sleeve in 2005-2006. Repaired electrical box for heater and motor thermal protection circuits in 2009.

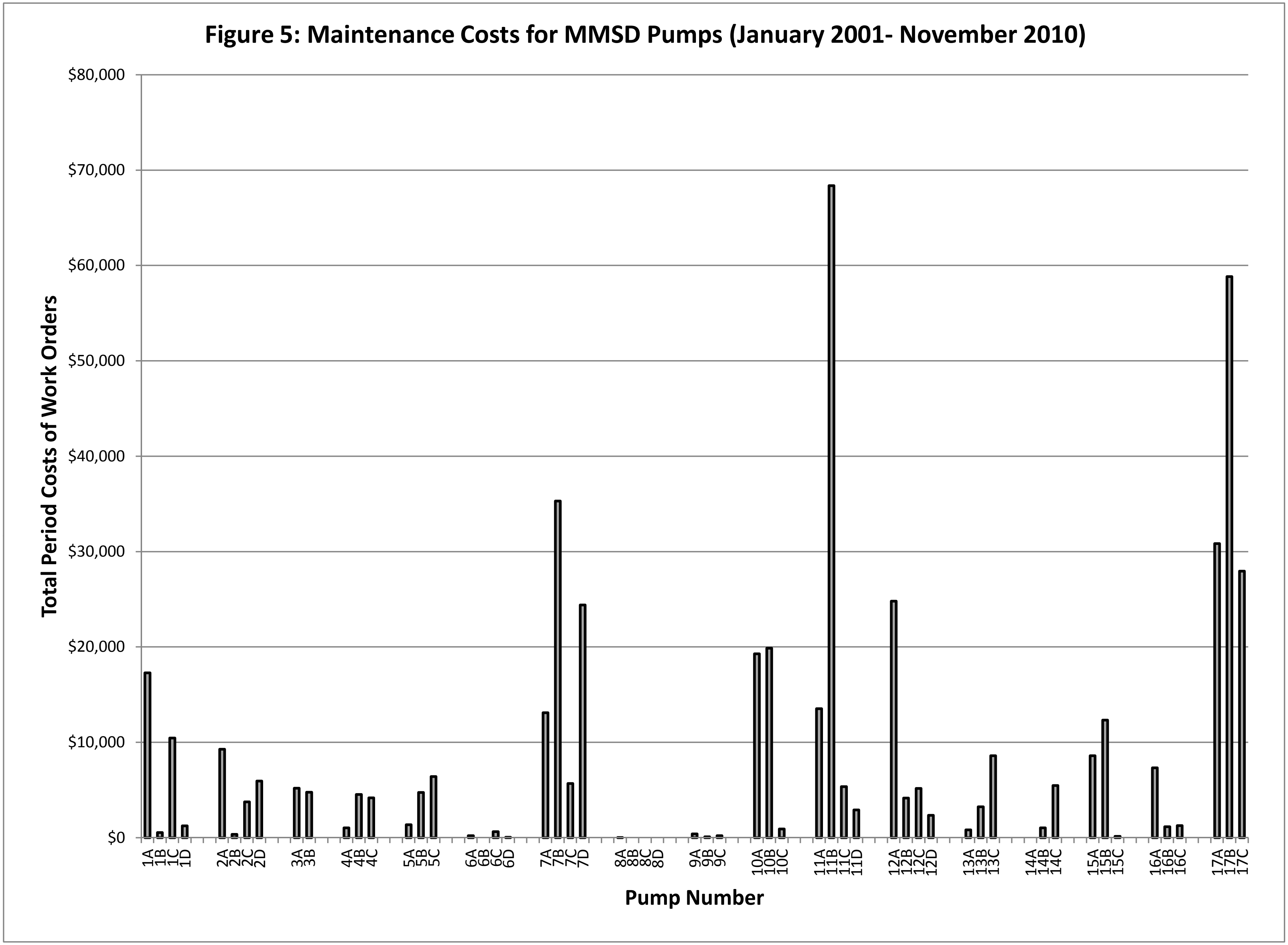
Figure 5: Maintenance Costs for MMSD Pumps (January 2001- November 2010)
Total Period Costs of Work Orders
$80,000
$70,000
$60,000
$50,000
$40,000
$30,000
$20,000
$10,000
$0
1A 1B 1C 1D 2A 2B 2C 2D 3A 3B 4A 4B 4C 5A 5B 5C 6A 6B 6C 6D 7A 7B 7C 7D 8A 8B 8C 8D 9A 9B 9C 10A 10B 10C 11A 11B 11C 11D 12A 12B 12C 12D 13A 13B 13C 14A 14B 14C 15A 15B 15C 16A 16B 16C 17A 17B 17C
Pump Number

Appendix A3
Connector Lines Between Stations
June 1999 (Updated April 2010)

MADISON METROPOLITAN
SEWERAGE DISTRICT

COLLECTION SYSTEM TECHNICAL GUIDELINES

APPENDIX 3

CONNECTOR LINES BETWEEN STATIONS

Stations 1-2, 2-8, 6-10, 4-8, 15-5, 15-16, 16-5, and 13-1

Author: Dick Klaas, June 1999

Updated: Todd Gebert, April 2010

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Introduction

District personnel have discussed and briefly investigated the possibility of constructing connector lines between several pumping stations. The main advantage of connector lines is to improve reliability during emergency situations. Connector lines can be very valuable if the force main or the pumping station develop major problems causing a loss of flow handling capabilities for a long period of time. Major problems are defined here as problems that would take a day or more to repair. Some stations can be out of service longer than others, but all stations would be a real concern if an outage lasted a day or more. Without connector lines there probably would be no other way to handle the flows during this outage time. Connector lines could also be used to shave peak flows, if needed.

The purpose of this memo is to identify existing and possible new connector lines, to comment on their usefulness, and to estimate the costs of constructing additional connector lines.

Background Information

Madison Metropolitan Sewerage District pumping stations were not designed with connector lines between them. System expansion over the years has provided opportunities to allow some transfer of flow from one station to another. All of these changes (interconnection of facilities) were done with very little cost to the District. Most often an existing facility with slight modification could be used in conjunction with the new facility being constructed.

Crosstown Force Main

The Crosstown force main now serves as the primary pumping option between Station 1 and Station 2, but it was not designed for that reason. It was originally constructed in 1914 to pump from old Booster Station 2 (near Brittingham Park) to old Booster Station 1 (near First Street), which then pumped to the Burke Treatment Plant. The Crosstown force main was replaced from 2000-2002 and is currently used to convey daily flows from Station 1 to Station 2. This reduces the flow that was previously pumped from PS1 to PS6 and subsequently PS7. In emergency situations flow can be reversed so that flow is from Station 2 to Station 1. These stations have similar firm and maximum pumping capacities after rehabilitation work was completed in 2005 (see Table 4.2 in Chapter 4).

Station 2-8 or 8-2

A portion of the Southwest Interceptor from Station 2 to the intersection of Haywood Drive and Mills Street serves as a connector line between Station 2 and Station 8. This section of sewer was not constructed as a connector line, but serves as one when either station is out of service long enough to back up the flow into this section of sewer. At present this section of sewer does not have adequate capacity to convey average daily flows that are diverted from either Station 2 or Station 8.

Station 15-5 and 15-16

Incoming gravity flow to Station 15 can be diverted to Station 5 through MH05-102A located near Station 15. The West Interceptor flow to Station 15 was originally handled by Station 5. The flow upstream of MH05-102A was diverted to Station 15 when the station was put in service in 1974. This manhole has a slide gate with a small hole in the middle of the gate to allow flow to continue down the West Interceptor. The hole is now above the normal water elevation so that flow through the hole occurs only during high flow situations.

Station 15 force main can be diverted to Station 16, if necessary. This diversion relieves the West Interceptor and Station 8.

Station 16-5

Incoming gravity flow to Station 16 can be diverted to Station 5 through the Gammon Extension by overflowing the dam in MH05-230 which is located across Gammon Road from Station 16. This would reduce flows to Stations 12 and 11.

Station 13-1

Prior to 1971, a portion of the Station 13 service area flowed to Station 1. This area includes approximately 2,150 acres adjacent to Warner Park in the City of Madison. Due to capacity constraints at Station 1 and the extension of the District’s Northeast Interceptor to Waunakee and DeForest in the early 1970’s, the City of Madison constructed a new interceptor in 1971 that diverted flow from the Warner Park area to Station 13. The infrastructure to convey flows to Station 1 is still largely in place, although modest system improvements would be needed to provide the required capacity.

Connectors and Potential Improvements

The following is a brief description of when and how the existing connector lines work. Other possible options were investigated to determine if other connectors are needed.

Station 2-8 Connector

The Southwest Interceptor (SWI) from Station 2 to MH08-106 at the intersection of Haywood Drive and Mills Street can be used to divert flow from Station 2 to Station 8. This diversion was used several times during repairs to the Station 2 force main prior to its replacement in 2001. Most repairs were needed to fix leaky joints, but there were also several pipe breaks. Leaky joints might not require force main shut down for more than a few hours. Pipe breaks have taken the force main out of service for days. The 1970 break of an elbow at Sayle Street and Van Duessen Street took the force main out of service for a week (see Appendix 5 for details).

Flow from Station 2 to Station 8 through the diversion section of the SWI at several different wet well elevations has been calculated. The maximum reliable diversion capacity is 3.9 MGD at a PS2 wetwell elevation of -2.00 (see Appendix 1). The invert elevation of the SWI at MH08-106 (Haywood and Mills) is -5.85 and at MH02-401 (near Station 2) the invert elevation is –9.75. The diversion length is approximately 3,200 feet, with a pipe slope of 0.12 % towards Station 2. Assuming a wet well elevation of -2.00 at Station 2, the calculated capacity is 3.9 MGD with a calculated water surface slope of 0.058%. Based on past station outages, the wet well should not be maintained any higher than -2.00 to minimize the risk of flooding. A survey of basement elevations around Station 2 found that most basements have elevations in the general range of -0.5 to 0.5. One backup was reported in 1999 when the wet well rose to Elevation -0.8.

Station 2 and Station 8 average daily and peak hourly flows are shown in Table 1 for 2009 flows and projected 2030 flows:

Table 1 - Stations 2 and 8 Flows

	Year 2009		Year 2030	
Pumping Station No.	Average Daily Flow (mgd)	Peak Hourly Flow (mgd)	Average Daily Flow (mgd)	Peak Hourly Flow (mgd)
2	10.15	28.15	10.74	29.52
2 (less 1)	5.93	17.90	5.21	16.04
8	7.60	22.07	9.31	26.18
8 (less 15)	6.24	18.69	7.38	21.53

Note: Average flows for 2009 are taken from MMSD pumping records. Average flows for 2030 are projected per CARPC's "MMSD Collection System Evaluation". All peak flows are derived from Madison Design Curve.

As mentioned previously, routine daily operation is for flows from Station 1 to be conveyed to Station 2 through the Crosstown force main. In the event that a problem develops at Station 2 or in the Station 2 force main, flows from Station 1 could be temporarily diverted to Station 6. For this reason, flows at Station 2 are shown in Table 1 with and without flow contribution from Station 1. Similarly, if operational problems were encountered at Station 8 or the Station 8 force main, flows could be diverted from Station 15 to Station 16.

All existing and 2030 average daily flows (~5.2 mgd) at Station 2 exceed the capacity of the existing 24" SWI diversion line (3.9 mgd), even with diversion of Station 1 flows away from Station 2.

Station 8-2 Connector

The SWI from MH08-106 to Station 2 can be used to divert flow from Station 8 during an outage of the station or during force main repairs. Based on past experience, basement flooding in the Station 8 service area starts at a wet well level of approximately +1.00 (see Appendix 6 for details). Assuming a wet well elevation one foot below this (0.00) at Station 8, the calculated slope of the water surface is 0.12% and the calculated capacity is 6.8 MGD (Appendix 3). This capacity is calculated using a Manning's n=0.015 and a slight decrease in pipe diameter since the iron build-up in the Haywood Drive diversion line is severe. This diversion is not fully capable of handling the existing average daily flow of 7.60 MGD to Station 8 but could potentially handle lower diurnal flows at night.

Diverting Station 15 flow to Station 16 would not relieve Station 8 quickly enough to divert remaining Station 8 flow through the existing SWI. The average flow time from Station 15 to Station 8 is 8.8 hours. This means the flow reduction at Station 8 would not be seen for 8.8 hours after switching the valves. Basements flooded within 3 hours during the Station 8 outage of June 24, 1998.

The 2002 *Collection System Facilities Plan* recommended investigating the possibility of reconfiguring sewers in the Randall Avenue area as another way to divert flow from Station 8 to Station 2. This would involve diverting flow from the West Interceptor/Randall Relief sewer to either the Spring Street Relief sewer or the West Interceptor on Regent Street. In 2003 the City of Madison completed a construction project at the intersection of Randall Avenue and Regent Street that redirected approximately 0.30 mgd of average daily flow from the Randall Relief to the West Interceptor. Given this diversion of flow and the possibility of heavy iron deposits in the 24" cast iron West Interceptor that may reduce capacity, it is not recommended to divert additional flow into this sewer. Conversely, opportunities may exist to divert flows from the Randall Relief Sewer to the Spring Street Relief sewer. CARPC's capacity evaluation projects that peak flows in the Spring Street Relief will be approximately 30%-35% of capacity by 2030. Appendix A8 of the update to the 2002 *Collection System Facilities Plan* includes further discussion on capacity needs in the West Side Conveyance System.

Haywood Drive Replacement Sewer

Due to the age, condition, and capacity of the 24" cast iron sewer on Haywood Drive, consideration should be given to replacing this sewer. A replacement line would provide much more reliability during outages, including all of the following:

- Station 2 outage
- Station 8 outage
- Station 2 force main problem
- Station 8 force main problem

Installation of a larger sewer is not needed to convey average daily flows, but the additional capacity provided would be very useful for flow diversion between Stations 2 and 8. Approximate diversion capacities between the stations are shown in Table 2 for both the existing 24" sewer and a 36" replacement sewer.

Table 2 - Connector Capacities for Stations 2 and 8

	Year 2030			
Pumping Station No.	Average Daily Flow (mgd)	Peak Hourly Flow (mgd)	Existing Diversion Capacity in 24" Haywood sewer (mgd)	Proposed Diversion Capacity in 36" Haywood sewer (mgd)
Diversion from PS 2 to PS 8				
2	10.74	29.52	3.90	10.40
2 (less 1)	5.21	16.04	3.90	10.40
Diversion from PS 8 to PS 2				
8	9.31	26.18	6.80	21.70
8 (less 15)	7.38	21.53	6.80	21.70

Assuming a unit cost of $700 per foot for a new 36" sewer, the cost to replace the Haywood Drive sewer and provide additional diversion capacity between Stations 2 and 8 is estimated to be approximately $1,000,000.

With the replacement sewer in place, 2030 average daily flows to Station 2 could very nearly be fully diverted to Station 8. 2030 peak hourly flows could not be fully diverted from Station 2 to Station 8, even with flow diversion to Station 6. Average daily flows to Station 8 could be safely diverted to Station 2, although peak flows could not be fully diverted.

In summary, the existing 24” sewer on Haywood Street does not have adequate capacity to safely divert average daily flows between Station 2 and Station 8. A 36” replacement sewer is needed to divert the anticipated 2030 average daily flows from each station. A 36” sewer would also allow for the diversion of a significant portion of peak hourly flows between the stations, although it could not be expected to convey all peak flows.

Station 1-2 Connector (Crosstown FM)

The Crosstown force main was replaced between 2000 and 2002 with new 24” and 30” diameter pipe. Previously this force main had been used during heavy rainfalls to divert flows from Station 1 to Station 2 and provide relief for Station 6. Since 2002 the Crosstown force main has been used to convey average daily and peak flows from Station 1 to Station 2. This change in operation has provided capacity relief for Stations 6 and 7. A small amount of flow is directed towards Station 6 on a daily basis to flush out the force main in an effort to reduce odors.

The Crosstown force main system has sufficient flexibility to permit pumping from Station 2 to Station 1. This mode of operation would typically only be used in the event of an outage at Station 2 or with the Station 2 force main. Reconfiguring the system to allow pumping to Station 2 requires manual intervention, including opening/closing several valves, and should be tested periodically to ensure proper operation.

Station 6-10 Connector

Background & Purpose

There are no connector lines between Station 6 and Station 10 at this time. The purpose of a connector would be to allow diversion of flow between Station 6 and Station 10. One of the primary reasons for investigating this connector is that the stations have wet wells at similar elevations. This unusual condition would make it much easier to transfer flows between stations than in other instances. A connector between Stations 6 and 10 would increase the reliability of District facilities if any of the following occurred:

- Station 6 outage
- Station 10 outage
- Station 6 force main problem
- Station 10 force main problem

In January of 2009 a contractor performing soil borings on Monona Drive drilled a hole into the Station 6 force main, disabling it for several hours. During the outage wastewater had to be hauled by truck from Station 6 to other points in the collection system. A connector line from Station 6 to Station 10 would have likely reduced or eliminated the need for hauling wastewater while the force main was out of service.

Another benefit of building a connector line between Station 6 and Station 10 is to reduce the chance of flooding basements in the Johns Street area in the Station 6 basin. When the wet well level at Station 6 reaches elevation –5.0, the City of Madison is called to isolate the Johns Street sewer from the rest of the Station 6 service area. There is not much time to react during high flows since the Station 6 wet well level rises very rapidly. At one time the City had plans to build a pumping station adjacent to Station 6. The proposed pumping station would act to isolate the local sewers along Johns Street from the Station 6 wet well. The City elected not to build this local pumping station, in part due to the District's change in operation in 2002 when flows from Station 1 were rerouted from Station 6 to Station 2. The proposed connector line would act to greatly mitigate flooding in the Johns Street area.

Exceeding the capacity at Station 6 will eventually cause an overflow into Starkweather Creek and/or Lake Monona. There is an overflow flap gate at MH06-102 that would overflow to the creek. Before the overflow elevation is reached many basements would flood in the Johns Street area, as previously mentioned. Station 10 previously had an overflow for the incoming interceptor sewer at MH10-114, near Sycamore Road. This overflow was abandoned in 2010 as part of the new relief and replacement sewers that were installed from Station 10 to Lien Road.

Route Alternatives and Cost

A connector line could flow by gravity from Station 6 to Station 10 or via force main between the two stations. Two alternate routes for a gravity connector are shown in Appendix 7. The connector line for Alternate 1 would connect to MH06-102 and travel along the east side of Starkweather Creek to O.B. Sherry Park. The sewer would extend northeasterly across the park to Milwaukee Street, at which point it would head to the north across lands owned by the Voit Concrete Company. The sewer would travel to the south and east of the existing sand pit on the Voit site and finally extend east to MH10-102 across lands owned by the City of Madison. The total length of the route is approximately 6,300 feet, with one railroad crossing and significant dewatering expected across the wetlands owned by the City of Madison. Excavation for a gravity main in the vicinity of Milwaukee Street would be on the order of 25-30 feet (see Appendix 10 for proposed invert elevations and manhole depths for both route options). Easements would be needed for much of the route for lands owned by the Voit Concrete Company and the City of Madison.

The connector line for Alternate 2 would be a more direct route to Station 10 along City of Madison streets. It would connect to the wet well at Station 6 and then head to the northeast to Station 10 along Harding Street, Richard Street, and Schenk Street. The total length of this option is approximately 5,600 feet, so it is significantly shorter than Alternate 1. The depths of the sewer are generally 20-25 feet along the entire length. Due to excavation on City streets the unit price of construction is expected to be considerably more than that for Alternate 1 due to additional factors such as traffic, other utilities, and pavement removal and replacement. Land acquisition for this option should be minimal.

It is likely that Alternate 1 would be the preferred route based on costs. Although certain segments involve deep construction and would require dewatering, there is little impact to City streets and interferences with other utilities should be minimal. Using a rough estimation of $800 per lineal foot for installation of a 48" gravity sewer, the connector line is estimated to cost $5.0 million.

Flowrates

Average daily and peak hourly flowrates at both stations for existing and future conditions are shown in Table 3. The carrying capacity of the connector line should be able to convey, at a minimum, the average daily flow through the year 2030.

Table 3 - Stations 6 and 10 Flows

	Year 2009		Year 2030	
Pumping Station No.	Average Daily Flow (mgd)	Peak Hourly Flow (mgd)	Average Daily Flow (mgd)	Peak Hourly Flow (mgd)
6	2.99	9.89	1.74	6.37
10	8.77	23.52	13.26	35.26

Note: Average flows for 2009 are taken from MMSD pumping records. Average flows for 2030 are projected per CARPC's "MMSD Collection System Evaluation". All peak flows are derived from Madison Design Curve.

The design of a connector line should also consider the operating parameters outlined in Table 4 for each station.

Table 4 - Stations 6 and 10 Design Parameters

Conditions	PS 6	PS 10
High Water Alarm	-5.5	-5.0
Overflow Elev.	+1.0	N/A
Flooding Elev.	-6.0	+2.0
Manhole Inverts	-8.9 at 6-102	-10.0 (10-104) & -10.89 (10-102A)
Large Pump Start	-6.2	-5.5

Flooding elevations listed above are critical for the connector line design. The maximum head allowed on the connector line would be at an elevation of –6.0 at Station 6 and +2.0 at Station 10. It would be advantageous for the connector line to have enough capacity to convey both average daily and peak hourly flowrates for future conditions, although the conveyance of peak flowrates may not be attainable. A 48" connector line could convey the majority of CARPC's projected peak flows in 2030 between the stations (see Appendices 8 and 9). The capacity of this diversion line and the average daily and peak hourly flowrates for 2030 are shown in Table 5.

Table 5 - Connector Capacities for Stations 6 and 10

Pumping Station No.	Year 2030		Capacity in Proposed 48" Diversion Section (mgd)
	Average Daily Flow (mgd)	Peak Hourly Flow (mgd)	
Diversion from PS 6 to PS 10			
6	1.74	6.37	5.6
Diversion from PS 10 to PS 6			
10	13.26	35.26	25.9

Note: Average flows for 2030 are projected per CARPC's "MMSD Collection System Evaluation". All peak flows are derived from Madison Design Curve.

There is little difference in elevation between flooding in the PS6 service areas (Elev = -6.0) and the elevation at which the pumps at PS 10 typically turn on (Elev = -7.50). As a result, there is minimal capacity in the diversion line from PS6 to PS10 under normal operating conditions. Approximately 5.6 mgd of flow could be transferred from Station 6 to Station 10 in an emergency. This amount of flow is greater than the average daily flow to PS6, but slightly less than the peak hourly flowrate at PS6 for 2030 projections.

With regard to the PS10 to PS6 diversion, it should be noted that the firm and maximum pumping capacity of PS6 is 24.2 MGD. Thus, Station 6 would not be able to handle the estimated diversion capacity of 25.9 MGD as shown in Table 5. Either additional capacity would need to be added at Station 6 or a smaller diversion line (42") could be installed.

Station 4-8 Connector

The 2002 *Collection System Facilities Plan* discussed the construction of a connector line from Station 8 forcemain to Station 4. Due to the relatively high costs involved to construct this line, another means of providing reliability for Station 4 was desired. A less expensive project involving the installation of valves to the force mains from Stations 2 and 4 was identified and completed in 2000.

The Station 4 force main connects to the Station 2 force main just to the east of Station 4. Prior to the PS2 Forcemain Replacement project, a break in the PS2 forcemain in either direction from PS4 would disable both force mains. By adding a valve just north of the Station 4 connection, Station 2 can be isolated if a Station 2 force main break occurs between Station 4 and Station 2. In this case Station 4 flow can continue to be pumped to the plant during the repair of a break between these stations. Another valve just south of the Station 4 connection allows Station 4 flow to pump to Station 2 if a force main break occurs between this valve and the meter vault at the treatment plant's headworks facility.

Connecting the Stations 2 and 4 force main lines has added reliability for force main breaks but not for station problems. Hauling Station 4 flow to the plant with Metrogro semi trucks or using a generator are the current contingency plans for any Station 4 outage.

Given the additional flexibility provided by the valve installation project in 2000, no additional connector lines are proposed for Station 4 at this time.

Station 15-5 and 15-16 Connectors

Incoming gravity flow to Station 15 can be diverted to Station 5 through MH05-102A located near Station 15. The West Interceptor flow was originally conveyed by Station 5. This flow upstream of MH05-102A was diverted to Station 15 in 1974 when the station was put in service. This manhole has a slide gate with a small hole in the middle of the gate to allow flow to continue down the West Interceptor. The hole is now above the normal water elevation so that flow through the hole occurs only during high flow situations.

Due to corrosion problems in the West Interceptor downstream of Station 15, consideration was given to abandoning a stretch of this system along Lake Mendota between Marshall Park and Baker Avenue. However, abandonment of this portion of the system would require the construction of new local sewers to maintain service to properties currently served directly by the West Interceptor. In addition, abandonment of the West Interceptor in this area would eliminate a valuable relief option for Station 15. The District intends to rehabilitate the corroded portions of the West Interceptor with a cured-in-place lining in 2011.

Station 15 was out of service on June 18, 1998 for almost 3 hours. Flow ran over the slide gate in MH05-102A without causing any known backup problems. Capacity of the 14" and 16" West Interceptor segments downstream of this manhole are 2.1 MGD and 2.9 MGD, respectively. These segment capacities are estimates based upon lining of the 1931 cast iron pipe. Station 15 average daily flow in 2009 was 1.36 MGD, with the 2030 estimated average daily flow increasing to 1.83 MGD. Thus, this portion of the West Interceptor can be relied upon for diverting Station 15's average daily flows until 2030, but the line capacity is likely exceeded for peak flows.

Other options available during Station 15 outages or force main problems include using a generator to run the station or pumping flow to Station 16 through a diversion force main. The diversion force main relieves the West Interceptor system and Station 8 and could be used during specific force main repairs. Force main problems downstream of MH15-01360 (near Allen Boulevard and University Avenue) could be repaired while diverting flow to Station 16.

Station 16-5 Connector

Incoming gravity flow to Station 16 can be diverted to Station 5 by overflowing the dam in MH05-230, located across Gammon Road from Station 16. This would also reduce flows to Stations 12 and 11. Station 16 average daily flow in 2009 (without Station 15 flow included) is 1.71 MGD, with an estimated increase to 3.05 MGD by 2030. Minimum capacity in the West

Interceptor downstream of MH05-230 is 1.39 MGD. Therefore, there is insufficient capacity in the West Interceptor diversion to handle all of the existing Station 16 flow.

Station 16 and its force main are 30 years old. The likelihood of failures for these facilities is less than in other portions of the collection system. Station controls have been a concern but were recently upgraded. Additional diversion capabilities for this station are not required at this time.

Station 13 Flow Diversion to Station 1

Background and Purpose

Currently there is little redundancy in the District's Eastside collection system. Other than Station 1, no pumping station in this part of the system has the ability to back up or relieve another station in the event of a station or force main outage. In an effort to provide more redundancy, connector lines between Stations 6 and 10 and between Stations 7 and 18 have been proposed and discussed in the Collection System Facilities Plan. Another location where redundancy could be implemented in the Eastside collection system is between Stations 13 and 1.

Prior to 1971, flows from the Warner Park area in the City of Madison were routed to Station 1 on N. First Street. Flow was conveyed along Packers Avenue to Oscar Mayer through City of Madison sewers that were originally constructed in the 1940's to serve the Dane County sanitarium on Northport Drive. A relief sewer was constructed along this route in the 1960's to provide additional capacity. MMSD sewers conveyed the flow from the end of the City's sewers at Oscar Mayer to Station 1.

In the late 1960's and early 1970's the District constructed Stations 13 and 14 and extended the Northeast Interceptor to the villages of DeForest and Waunakee. Due to capacity constraints in the Station 1 service area and with the Warner Park area continuing to grow, the City elected to build a new diversion sewer (Truax Interceptor) to connect the Warner Park lands to the Northeast Interceptor and Station 13. This diversion sewer begins at the intersection of Packers Avenue and International Lane and currently directs all flow from the Warner Park area to Station 13 via slide gates.

Flowrates at Pumping Stations

The District has made significant improvements to conveyance and pumping capacity in the Station 1 service area since the 1960's and 1970's. Adequate capacity is now available such that flows from the City of Madison's Fremont Pumping Station and the County sanitarium sewershed could be redirected to Station 1, if desired. The lands generating the flows to be diverted comprise approximately 2,150 acres and are shown as subbasins 13-A, 13-D, and 13-E on CARPC's subbasin delineation map in Appendix 11 (Figure 3-34). Using CARPC's 2030 TAZ flow estimates, an average daily flow of 1.23 MGD and a peak hourly flow of 3.06 MGD could be diverted by gravity from Station 13 to Station 1.

Benefits of Flow Diversion

There are several benefits to diverting a portion of the Station 13 flow to Station 1. Most importantly, the diversion would allow for redundancy and flexibility in this portion of the collection system. This is an important consideration in that the Station 13 service area is generally a low-lying area with a history of infiltration and inflow concerns.

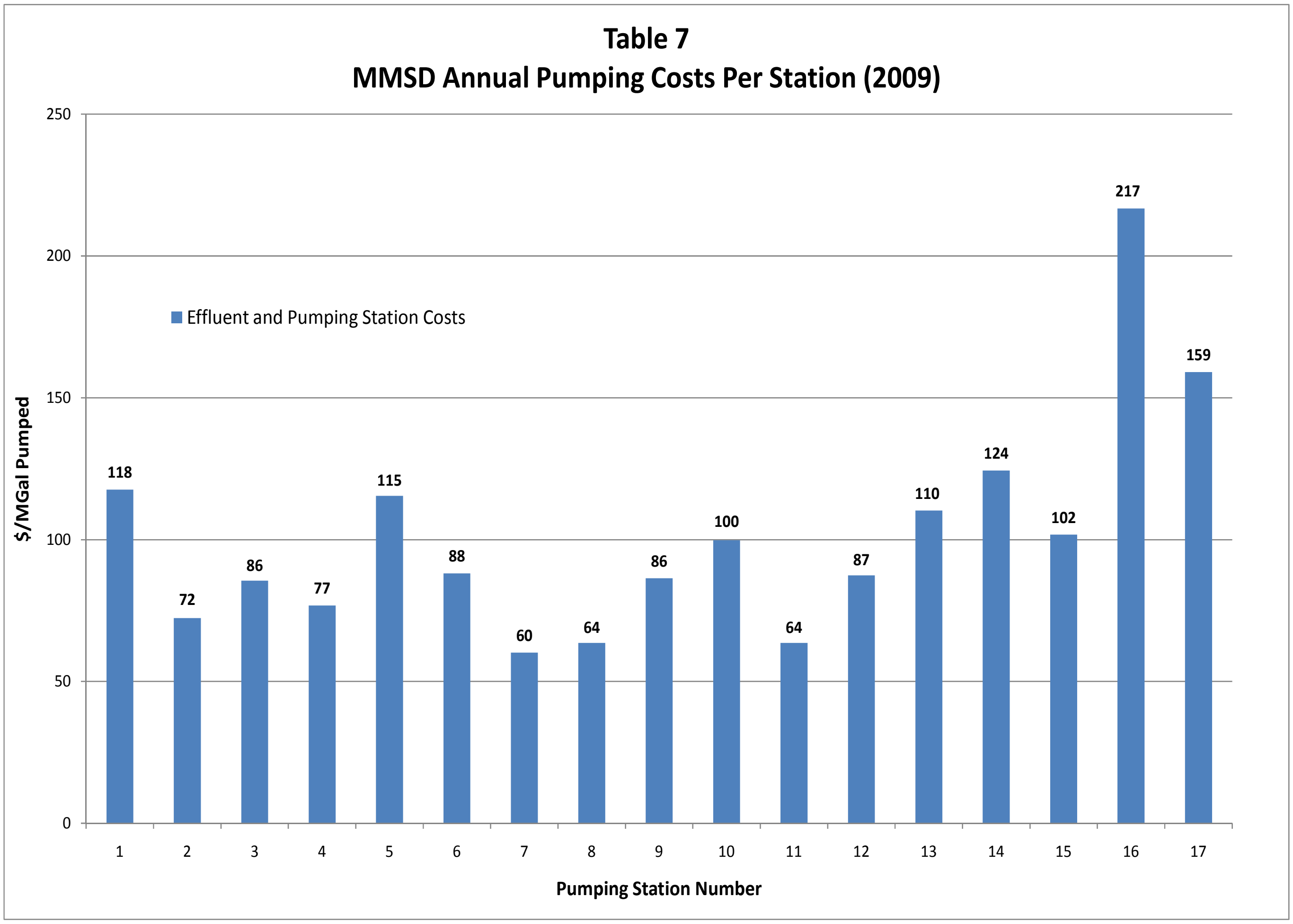
The flow diversion would also postpone the need for firm pumping capacity improvements at Station 13 and in the interceptor downstream of PS13 by at least ten years. Table 6 shows the major facility improvements affected by the PS13 flow diversion and the required timing of these improvements. CARPC's flow projections using both Traffic Analysis Zone (TAZ) data and Uncertainty Factor (UF) data were used in determining the timing of the improvements.

Table 6 - Summary of Improvements for PS 13 Flow Diversion

	Year Improvement is Required			
	No Diversion		With Diversion	
Improvement	TAZ Flows	UF Flows	TAZ Flows	UF Flows
Increase PS 13 Firm Capacity	2020	2010	2036	2021
Relief for NEI – Truax Extension (MH10-145 to MH10-121)	2032	2017	2043	2027
Construct interceptor to connect PS 13 service area to PS 1	Not needed	Not needed	Prior to flow diversion	Prior to flow diversion

It is not anticipated that any capacity improvements would be needed at downstream Stations 1 and 2 if flow were diverted from the Station 13 service area. Both of these stations were recently rehabilitated and have sufficient firm capacity to accept the diverted flow. The effect of the diversion on flowrates at all of the downstream pumping stations is shown in Appendix 12 for various development scenarios.

Diverting flow from the PS13 service area would result in a small increase in overall pumping costs. Currently flow from the PS13 subbasins is pumped at three stations (13, 10, and 7). With the diversion the flow would be pumped at two stations (1 and 2). As can be seen in Table 7, the total unit cost to pump flow through Stations 13, 10 and 7 and the treatment plant's effluent force main is approximately $110/MGal. The costs for pumping through Stations 1 and 2 and the effluent force main are approximately $118/MGal. Thus, there is a small increase in pumping costs associated with diverting the flow to Station 1.



Infrastructure and Costs

Much of the infrastructure to convey the diverted flow is already in place and has available capacity. In 2002 the District completed its upgrade of the North Basin Interceptor from Station 1 to the intersection of Pennsylvania Avenue and Commercial Avenue. This new 36" sewer has adequate capacity to accommodate the diverted flow. Sufficient capacity should also be available in the City's sewer system along Packers Avenue from International Lane to a point approximately 650 feet south of Aberg Avenue. At this point the City's sewers decrease in size and additional capacity would have to provided to the terminus of the North Basin Interceptor at Pennsylvania Avenue and Commercial Avenue.

The District has abandoned facilities along the Packers Avenue and Commercial Avenue corridor. At one time the Burke Outfall and a 30" cast iron sewer connected to the City's sewers south of Aberg Avenue and conveyed flow all the way to Station 1. These facilities were originally constructed in 1911 and 1912 to convey flow to and from the Burke Treatment Plant east of STH 113 and were eventually converted to gravity sewers. Due primarily to structural considerations, portions of these facilities were abandoned in the 1990's, with the entire length abandoned fully by 2002.

The length of new sewer along Packers Avenue and Commercial Avenue that would be required is approximately 2,700 feet in length (see Appendix 13 for map). Assuming installation of a new 30" sewer at a unit cost of $700 per foot, the approximate project cost would be $1.9 million. A present worth analysis was conducted to compare the life cycle costs over a 40-year period for operation under the existing conditions as opposed to diverting flow as previously discussed (see Appendix 14). The present worth cost to operate under existing conditions is approximately $8.6 million, while the present worth costs to divert the flow is approximately $8.1 million. Thus, it should be economically feasible to construct this project and operate as outlined above. A more thorough cost analysis should be conducted to evaluate this project as capacity needs at PS13 and in the NEI (Truax Extension) become more imminent.

It is assumed that installation of a replacement sewer could proceed along the Packers Avenue Service Road and Commercial Avenue adjacent to the abandoned Burke Outfall. This assumption requires further investigation as a portion of the Burke Outfall was abandoned in 1995 at the City's request to allow installation of a new storm sewer for Oscar Mayer. Conflicts with this storm sewer and other utilities may pose a problem for installation of a new replacement sewer in this corridor.

Conclusions

Little flexibility and redundancy is currently available in the District's Eastside collection system, especially in the upper reaches of the system (Stations 13 and 14). The ability to divert flow from a portion of the Station 13 service area to Station 1 would provide options during high-flow events or extended station or force main outages downstream. While this diversion would address only a fraction of the total flow to Station 13, it may prove especially useful in very intense and localized storms, such as the storms that caused Stations 13 and 14 to be bypassed for a short time in June of 2008.

At present there is not a pressing need to implement this diversion. The project is expected to postpone the need to provide capacity relief in the NEI (Truax Extension) and additional firm pumping capacity at Station 13 by an additional ten years, although PS13 will likely require a major rehabilitation for equipment prior to 2030. The effect on other system improvements due to this diversion is negligible. Nevertheless, the cost to implement this project is relatively affordable and should be considered a long-term goal. The City's sewers along the Packers Avenue Service Road are 70-80 years old and are approaching the end of their service lives. When the City elects to replace or rehabilitate these sewers the District should consult with the City and investigate cost-sharing alternatives to provide additional capacity in this corridor.

Summary and Recommendations

This memo reviewed the status of existing station connector lines and identified potential improvement projects and potential new projects. It is recommended that the following projects be evaluated during the overall facility plan project prioritizing procedure.

1. Replace the Southwest Interceptor from MH08-106 (Haywood Drive and Mills Street) to MH 2-606 (Haywood Drive and West Shore Drive) with a 36” line to serve as a gravity connector between Stations 2 and 8. Approximate cost is estimated at $1 million.

2. Investigate a possible new 48” connector between Stations 6 and 10 at an approximate cost of $5.0 million.

3. As a long-term consideration, explore opportunities to divert flow on a daily or event basis from the Station 13 service area to Station 1. District staff should consult with the City of Madison on cost-sharing alternatives to upgrade sewer capacity along the Packers Avenue Service Road and Commercial Avenue between Packers Avenue and Pennsylvania Avenue.

Appendix

1. Emergency diversion from PS 2 to PS 8 – Existing conditions
2. Emergency diversion from PS 2 to PS 8 – Proposed conditions
3. Emergency diversion from PS 8 to PS 2 – Existing conditions
4. Emergency diversion from PS 8 to PS 2 – Proposed conditions
5. 1970 PS 2 FM broke elbow near Sayle Street
6. 1998 PS 8 power outage
7. Alternate route map for Stations 6-10 connector line
8. Emergency diversion from PS 6 to PS 10
9. Emergency diversion from PS 10 to PS 6
10. PS 6-10 Connector Characteristics
11. Map of Pump Station 13 service area
12. Pump Station 13 flow diversion – Effects on downstream pump stations
13. Pump Station 13 Diversion Sewer
14. 40-Year Present Worth Cost Analysis for Pump Station 13 Diversion

APPENDIX 1 - EMERGENCY DIVERSION FROM PS 2 TO PS 8
Existing Conditions

FIND: Wet well elevations at Pump Station No. 2 for various rates of "backflow" from PS 2 to PS 8.

I. PHYSICAL CHARACTERISICS OF DIVERSION (EXISTING)

Haywood Drive Section		**Shore Drive Section**	
Length, L, of 24" overflow (ft) =	1,438	Length, L, of 36" overflow (ft) =	1,770
Pipe diameter, D (ft) =	1.92	Pipe diameter, D (ft) =	3.00
Pipe area, A (ft^2) =	2.8853	Pipe area, A (ft^2) =	7.0686
Hydraulic radius, R (ft) =	0.48	Hydraulic radius, R (ft) =	0.75
Manning's n =	0.015	Manning's n =	0.013

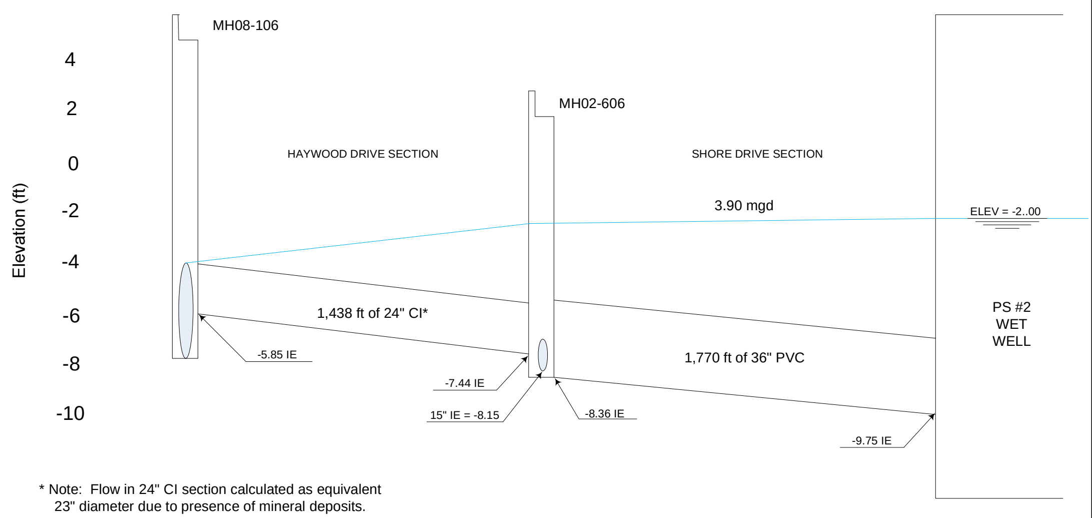
Water surface elevation at discharge (MH08-106) = -3.85 (assumes 48" Randall Relief flowing full)

II. CALCULATE FLOW BY MANNING'S EQUATION (EXISTING)

$Q = (1.49/n) * A*R^{2/3} *S^{1/2}$

$\Delta H = ((Q*n)/(1.49*A*R^{2/3}))^2 * L$

Diversion Flow, Q (mgd)	Diversion Flow, Q (cfs)	Head Loss, ΔH, in Haywood Drive Section (ft)	Water Surface Elevation at MH02-606 (ft)	Head Loss, ΔH, in Shore Drive Section (ft)	Water Surface Elevation at PS 2 (ft)
8.00	12.38	7.19	3.34	0.61	3.94
7.00	10.83	5.50	1.65	0.46	2.12
6.00	9.28	4.04	0.19	0.34	0.53
5.00	7.74	2.81	-1.04	0.24	-0.81
4.00	6.19	1.80	-2.05	0.15	-1.90
3.90	**6.03**	**1.71**	**-2.14**	**0.14**	**-2.00**
3.00	4.64	1.01	-2.84	0.09	-2.75
2.00	3.09	0.45	-3.40	0.04	-3.36
1.00	1.55	0.11	-3.74	0.01	-3.73
0.00	0.00	0.00	-3.85	0.00	-3.85



* Note: Flow in 24" CI section calculated as equivalent 23" diameter due to presence of mineral deposits.

EMERGENCY DIVERSION – PS 2 TO PS 8

Existing Conditions

APPENDIX 2 - EMERGENCY DIVERSION FROM PS 2 TO PS 8
Proposed Conditions

FIND: Wet well elevations at Pump Station No. 2 for various rates of "backflow" from PS 2 to PS 8.

I. PHYSICAL CHARACTERISICS OF DIVERSION

Haywood Drive Section		**Shore Drive Section**	
Length, L, of 24" overflow (ft) =	1,438	Length, L, of 36" overflow (ft) =	1,770
Pipe diameter, D (ft) =	3.00	Pipe diameter, D (ft) =	3.00
Pipe area, A (ft^2) =	7.0686	Pipe area, A (ft^2) =	7.0686
Hydraulic radius, R (ft) =	0.75	Hydraulic radius, R (ft) =	0.75
Manning's n =	0.013	Manning's n =	0.013

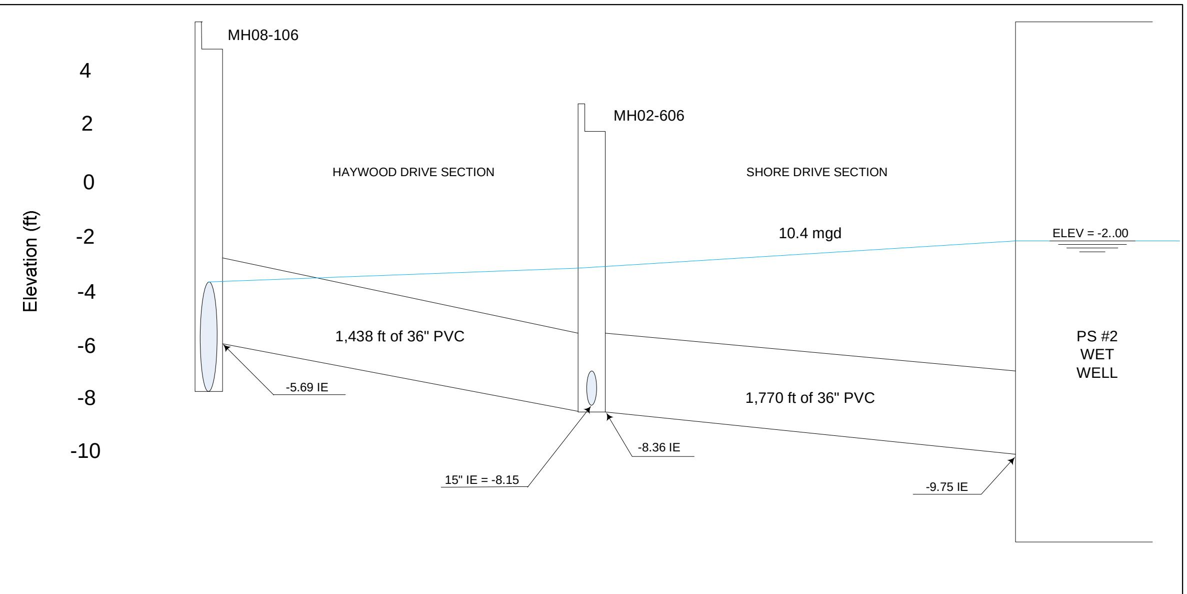
Water surface elevation at discharge (MH08-106) = -3.85 (assumes 48" Randall Relief flowing full)

II. CALCULATE FLOW BY MANNING'S EQUATION

$Q = (1.49/n) * A*R^{2/3} *S^{1/2}$

$\Delta H = ((Q*n)/(1.49*A*R^{2/3}))^2 * L$

Diversion Flow, Q (mgd)	Diversion Flow, Q (cfs)	Head Loss, ΔH, in Haywood Drive Section (ft)	Water Surface Elevation at MH02-606 (ft)	Head Loss, ΔH, in Shore Drive Section (ft)	Water Surface Elevation at PS 2 (ft)
16.00	24.75	1.97	-1.88	2.43	0.55
15.00	23.21	1.73	-2.12	2.14	0.02
14.97	23.16	1.73	-2.12	2.13	0.00
14.00	21.66	1.51	-2.34	1.86	-0.48
13.00	20.11	1.30	-2.55	1.60	-0.94
12.00	18.56	1.11	-2.74	1.37	-1.37
11.00	17.02	0.93	-2.92	1.15	-1.77
10.37	**16.04**	**0.83**	**-3.02**	**1.02**	**-2.00**
10.00	15.47	0.77	-3.08	0.95	-2.13
9.00	13.92	0.62	-3.23	0.77	-2.46
8.00	12.38	0.49	-3.36	0.61	-2.75
7.87	12.17	0.48	-3.37	0.59	-2.78
7.00	10.83	0.38	-3.47	0.46	-3.01
6.00	9.28	0.28	-3.57	0.34	-3.23



EMERGENCY DIVERSION – PS 2 TO PS 8

Proposed Conditions – Haywood Drive Replacement Sewer

APPENDIX 3 - EMERGENCY DIVERSION FROM PS 8 TO PS 2
Existing Conditions

FIND: Wet well elevations at Pump Station No. 8 for various rates of "backflow" from PS 8 to PS 2

I. PHYSICAL CHARACTERISICS OF DIVERSION

Haywood Drive Section		**Wingra Drive Section**	
Length, L, of 24" overflow (ft) =	1,438	Length, L, of 48" overflow (ft) =	3,179
Pipe diameter, D (ft) =	1.92	Pipe diameter, D (ft) =	4.00
Pipe area, A (ft^2) =	2.885254167	Pipe area, A (ft^2) =	12.5664
Hydraulic radius, R (ft) =	0.48	Hydraulic radius, R (ft) =	1.00
Manning's n =	0.015	Manning's n =	0.013

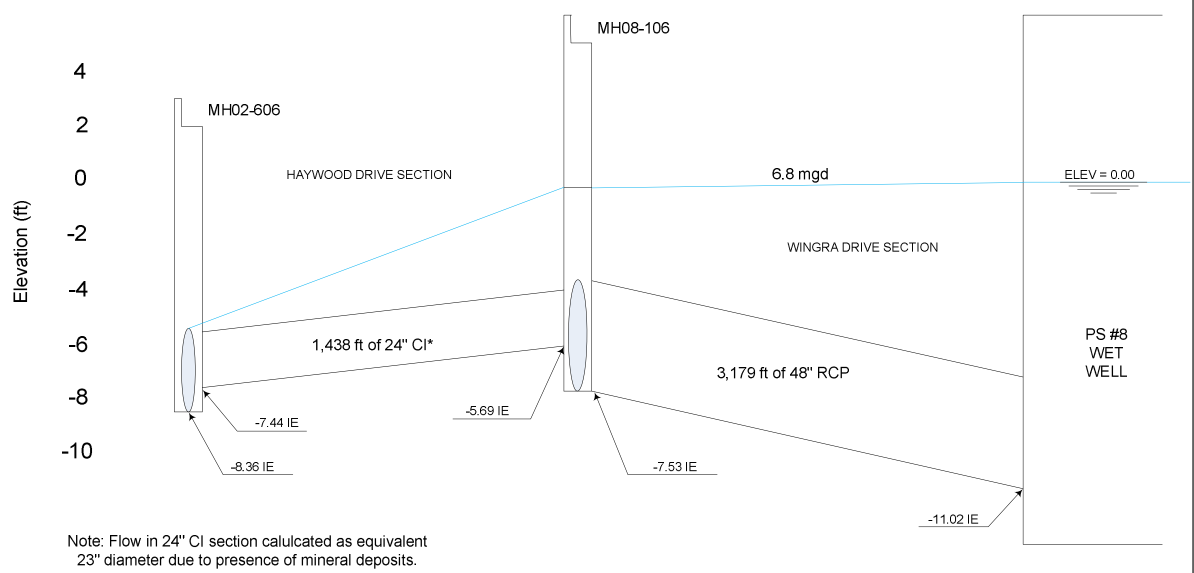
Water surface elevation at discharge (MH02-606) = -5.36 (assume 36" on Shore Drive is flowing full)

II. CALCULATE FLOW BY MANNING'S EQUATION

$Q = (1.49/n) * A*R^{2/3} *S^{1/2}$

$\Delta H = ((Q*n)/(1.49*A*R^{2/3}))^{2} * L$

Diversion Flow, Q (mgd)	Diversion Flow, Q (cfs)	Head Loss, ΔH, in Haywood Drive Section (ft)	Water Surface Elevation at MH08-106 (ft)	Head Loss, ΔH, in Wingra Drive Section (ft)	Water Surface Elevation at PS 8 (ft)
8.00	12.38	7.19	1.83	0.23	2.06
7.00	10.83	5.50	0.14	0.18	0.32
6.80	**10.52**	**5.19**	**-0.17**	**0.17**	**0.00**
6.00	9.28	4.04	-1.32	0.13	-1.19
5.23	**8.09**	**3.07**	**-2.29**	**0.10**	**-2.19**
5.00	7.74	2.81	-2.55	0.09	-2.46
4.00	6.19	1.80	-3.56	0.06	-3.50
3.00	4.64	1.01	-4.35	0.03	-4.32
2.00	3.09	0.45	-4.91	0.01	-4.90
1.00	1.55	0.11	-5.25	0.00	-5.24
0.00	0.00	0.00	-5.36	0.00	-5.36



EMERGENCY DIVERSION – PS 8 TO PS 2

Existing Conditions

APPENDIX 4 - EMERGENCY DIVERSION FROM PS 8 TO PS 2

Proposed Conditions

FIND: Wet well elevations at Pump Station No. 8 for various rates of "backflow" from PS 8 to PS 2

I. PHYSICAL CHARACTERISICS OF DIVERSION

Haywood Drive Section

Length, L, of 24" overflow (ft) =	1,438
Pipe diameter, D (ft) =	3.00
Pipe area, A (ft^2) =	7.0686
Hydraulic radius, R (ft) =	0.75
Manning's n =	0.013

Wingra Drive Section

Length, L, of 48" overflow (ft) =	3,179
Pipe diameter, D (ft) =	4.00
Pipe area, A (ft^2) =	12.5664
Hydraulic radius, R (ft) =	1.00
Manning's n =	0.013

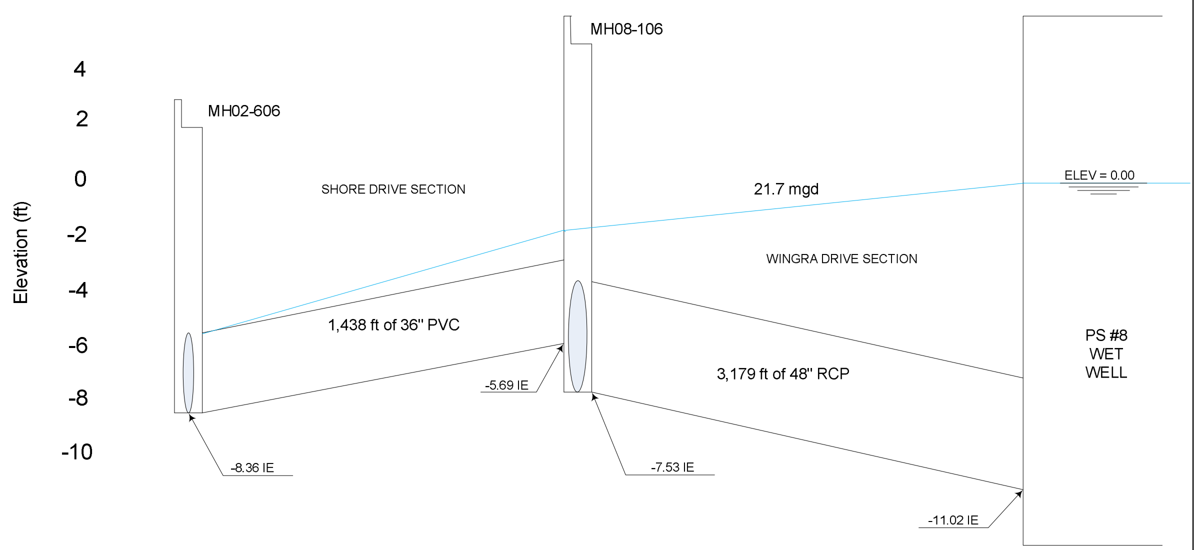
Water surface elevation at discharge (MH02-606) = -5.36 (assume 36" on Shore Drive is flowing full)

II. CALCULATE FLOW BY MANNING'S EQUATION

$Q = (1.49/n) * A*R^{2/3} *S^{1/2}$

$\Delta H = ((Q*n)/(1.49*A*R^{2/3}))^2 * L$

Diversion Flow, Q (mgd)	Diversion Flow, Q (cfs)	Head Loss, ΔH, in Haywood Drive Section (ft)	Water Surface Elevation at MH08-106 (ft)	Head Loss, ΔH, in Wingra Drive Section (ft)	Water Surface Elevation at PS 8 (ft)
28.00	43.32	6.04	0.68	2.88	3.56
27.00	41.77	5.62	0.26	2.67	2.93
26.00	40.22	5.21	-0.15	2.48	2.33
25.00	38.68	4.82	-0.54	2.29	1.75
24.00	37.13	4.44	-0.92	2.11	1.19
23.00	35.58	4.08	-1.28	1.94	0.66
22.00	34.03	3.73	-1.63	1.78	0.15
21.70	**33.57**	**3.63**	**-1.73**	**1.73**	**0.00**
21.00	32.49	3.40	-1.96	1.62	-0.34
20.00	30.94	3.08	-2.28	1.47	-0.81
19.00	29.39	2.78	-2.58	1.32	-1.25
18.00	27.85	2.50	-2.86	1.19	-1.67



EMERGENCY DIVERSION – PS 8 TO PS 2

Proposed Conditions – Haywood Drive Replacement Sewer

APPENDIX 5

January 21, 1970

PRESSURE LINE BREAK
SAYLE & VAN DUESEN
STREETS

On Wednesday, January 14, 1970, at about 9:00 A.M. the operator at Nine Springs reported unusually low effluent flow and high raw sewage flow to the plant. Checks were made of all tanks and overflows without locating the trouble. At about 1:30 P.M. a call was received from the City Engineering Department that there was a geyser at Sayle and Van Duesen Streets east of our old #4 Pumping Station. This was right over a bend in the 30" cast iron pressure line from #2 Pumping Station to Nine Springs. Procedures were instituted to shut off the flow in the broken main by diverting the flow from #2 across town to #1 Station, to bypass #4 Station into Murphys Creek and bypass #3 Station to the marsh. Since sewage was flowing from the meter vault mixing chamber back to the break, stop logs were ordered into the #2 incoming line. Calls were made to local contractors to get men and digging equipment. Flow at the break continued very heavy until 8:00 P.M. when it was discovered that the stop logs were in #11 line. They were moved to #2 line which reduced but did not stop all the flow immediately.

n Thursday, January 15, 1970, Icke Const. Co. arrived but flow at the reak prevented any work. The stop logs at Nine Springs were tightened and sand bagged until leaking appeared negliable. The shut off valve at Pumping Station No. 2 into the pressure line was tightened down another 29 turns and the flow at the break stopped. Pumping was started and by about 10:00 P.M. we could see that a piece of the 30" elbow had blown out. Pumping was continued through the night to drain the line.

On Friday, January 16, 1970, Icke Co. dug out the break. The blown 45° elbow was broken out and replaced with an elbow from our stock. The bell next to the elbow was cut off to get installation room and re-leaded. A stainless steel, 6 part, Adams repair sleeve was obtained from the City Water Utility and installed. When sewage was turned into the line, the sleeve leaked.

On Saturday, January 17, 1970, a 6" piece of 30" pipe was cut and inserted in the line and the sleeve was tried again. Efforts to find a different type of clamp sleeve in stock of other utilities and manufacturers were in vain.

On Sunday, January 18, 1970, a 15" cast iron lead joint repair sleeve was brought from our stock. The old bell piece and spacer were removed nd new ones were cut to reduce the clearance space in the line. A 2" ent in the cover of the inspection tee across the creek from the break was opened to keep the line dry for pouring the lead joints. Valves were closed at #4 Station to isolate it from the line when it was found to be siphoning into the pressure line. The cast iron sleeve was installed and

Jan. 21, 1970

PRESSURE LINE BREAK
SAYLE & VAN DUESEN
STREETS - continued

leaded and gravel bedding was placed under the exposed pipe.

On Monday, January 19, 1970, the elbow was braced and #4 Station was turned into the line at 11:00 A.M. About noon #3 Station was turned in and all bypassing was stopped. The entire pressure line was full and overflowing at the meter vault mixing chamber by about 3:00 P.M.. A concrete thrust block was poured, covered with hay and allowed to set.

On Wednesday, January 21, 1970, valves were operated putting #2 Station on the line to Nine Springs at about 3:00 P.M. thus returning the system to normal pumping.

From 4:00 P.M. on January 14 to 3:00 P.M. on January 21, the sewage flow to Pumping Station #2 was pumped to Pumping Station #1 and flowed by gravity to Pumping Station No. 8.

The bypassing consisted of the flow from the break while the pipe was being taken out of service, the sewage to drain the line and the flow to Pumping Station #4 and Pumping Station #3.

JTW

JOHN T. WRIGHT

APPENDIX 6

MEMORANDUM

To: Pump Station 8 File

From: Paul Nehm

Subject: June 24, 1998 Power Outage at Pump Station 8

Date: June 24, 1998

At 4:50 am on June 24, 1998 the preferred power lines at both Pump Station 2 and Pump Station 8 were lost during a storm. Both stations immediately switched to emergency power. At Pump Station 2 the well level was high enough for Pump A to turn on. The well had just been pumped down at Pump Station 8 so none of the pumps came on.

At 5:30 am both stations switched back to their preferred power lines. At this time the well level at Pump Station 8 was about 0.2 ft higher than the level that Pump A is set to turn on. By 5:40 am the well level had risen to -7.7 ft which was about 0.8 ft above the start level for Pump A but still 0.5 ft below the start level for Pump B. Jeff Woerpel placed the station in computer control and tried to turn on Pump A. Even though the TLM screen showed that preferred power was available, the pump would not turn on. Jeff returned the station to local control. Since his review of our emergency response manual showed that Pump Station 8 could be without power for two hours and it was only an hour from our normal starting time, we did not call out any maintenance personnel.

When Roy Swanke arrived at the plant he was told of the situation and he immediately sent Dave Helgesen to the station. Dave arrived at Pump Station 8 shortly before 7:00 am. He found that the lights at the station worked, but he could not start any of the pumps using either power source. Dave relayed this information to Roy who contacted Dave Lundey. Dave Lundey sent Jerry DuBois and Dave Smith to the station.

At about 7:20 am Dave Helgesen called the plant to say that the electricians could not get any of the pumps to run either. Dave Lundey immediately went to the station. At about 7:50 am Dave Lundey called the control room to request that the operator report to MG&E that there was a power problem at the station and to ask that someone bring our portable generator to the station. Rick Neath made the call to MG&E. It appeared that they were not aware of the problem. Dick Hockett took the generator to Pump Station 8.

Because Pump Station 8 has two power feeds, it is not set up with a plug in connection for the generator. The electricians had to hard wire the generator to the station. At about 8:50 am Roy Swanke called to say that the generator was running Pump B. This pump pumped at about 14.8 MGD. TLM communications with the station had been lost shortly before this. It is estimated that the wet well level was slightly above +2.0 at this time.

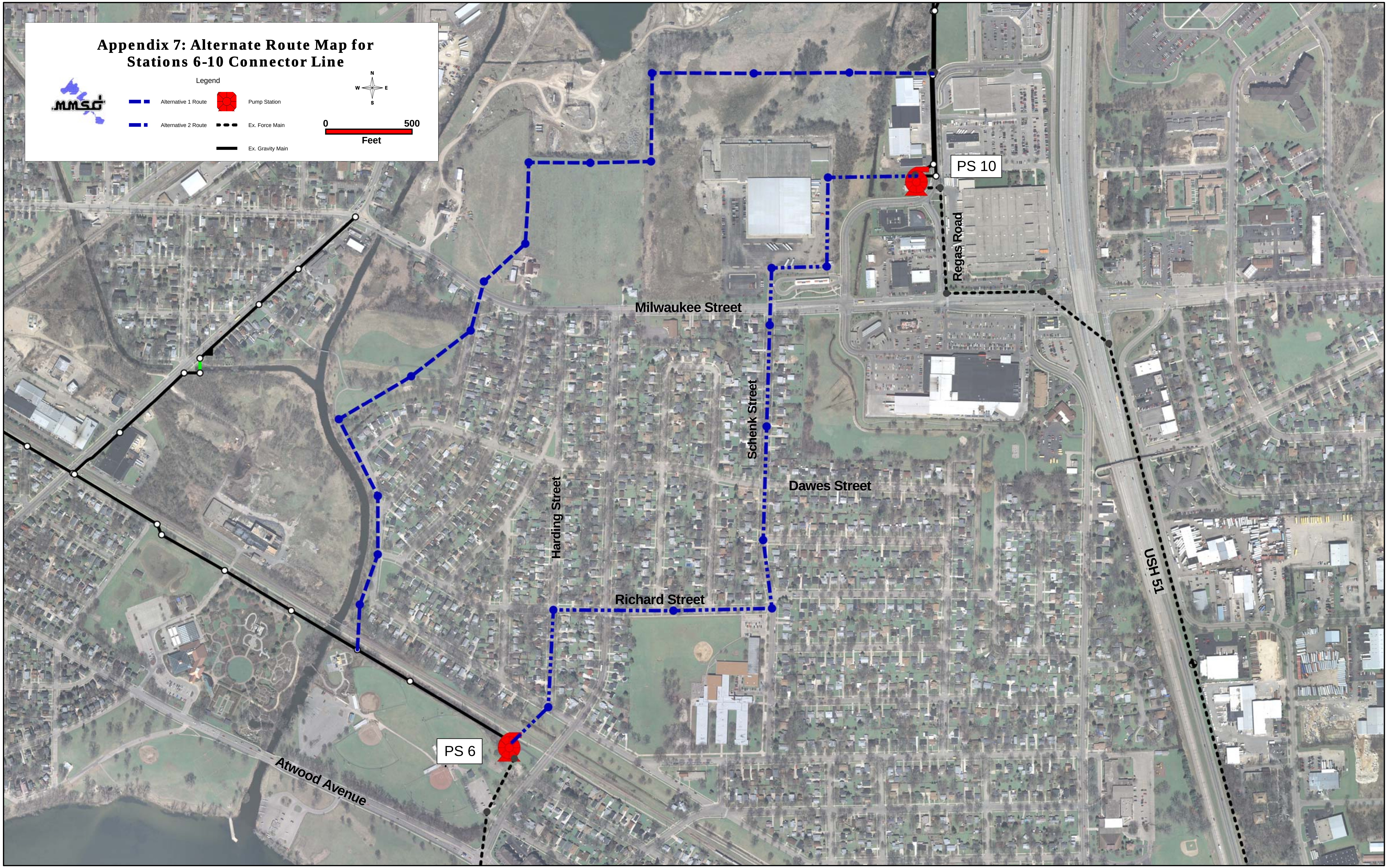
Shortly after this Duane Sippola called the plant to ask if we had any pump station problems. He had received two calls of basement flooding in the Wingra Drive area. While we were talking his dispatcher told him of two additional flooded basements and also reported that the zoo was experiencing backups. I told Duane that the well level was dropping since Pump B was now operating at the station and the sewer system was overflowing to Pump Station 2. Later it was determined that the City received the first notice of basement flooding at 8:30 am. At that time the wet well elevation was about +0.9 feet.

At 9:30 am MG&E had made a temporary repair to allow the emergency line to be used. Pump D was turned on. TLM communications with the station were again off at this time, but it is estimated that the well elevation was about -3.5 ft. This pump continued to drop the well level until it shut off at 11:08am.

The collection system has a cross connection which allows wastewater to flow between Pump Stations 2 and 8 if the well level at either station rises high enough. At about 7:45 am Larry Marquardt was asked to check this connection and to check the level of the water in the overflow manhole to Wingra Creek. Larry reported that the level in the cross connection manhole was about five feet from the top of the manhole and water was flowing to Pump Station 2. He also found the level of the water in the overflow manhole to Wingra Creek to be about three feet from the top of the stoplogs in the manhole. We believe that the wet well depth rose about another 1.5 feet from the time that Larry made these observations. This would mean that the high water elevation in the overflow manhole was about 1.5 feet below the top of the stoplogs.

I should be noted that there seems to be some discrepancy between the wet well elevations as displayed on the TLM screens and as read in the stations. Early in the morning Rick Neath was reading an elevation which was about one foot lower than what Dave Helgesen was reading at the station. This needs to be checked.

Another item of interest is that in the forty minutes from 4:50 am to 5:30 am, the wet well level at Pump Station rose about six feet. For the next three hours it rose 1.2 to 1.8 feet every 30 minutes.



Appendix 7: Alternate Route Map for
Stations 6-10 Connector Line
Legend
Alternative 1 Route
Alternative 2 Route
Pump Station
Ex. Force Main
Ex. Gravity Main
0
500
Feet
N
W
E
S
PS 10
Regas Road
Milwaukee Street
Schenk Street
Dawes Street
Harding Street
Richard Street
USH 51
PS 6
Atwood Avenue

APPENDIX 8 - EMERGENCY DIVERSION FROM PS 6 TO PS 10

Proposed Conditions

FIND: Wet well elevations at Pump Station No. 6 for various rates of "backflow" from PS 6 to PS 10.

I. PHYSICAL CHARACTERISICS OF DIVERSION

Length, L, of 42" overflow (ft) =	1,043	Length, L, of 48" overflow (ft) =	6,300
Pipe diameter, D (ft) =	3.50	Pipe diameter, D (ft) =	4.00
Pipe area, A (ft^2) =	9.62115	Pipe area, A (ft^2) =	12.5664
Hydraulic radius, R (ft) =	0.88	Hydraulic radius, R (ft) =	1.00
Manning's n =	0.013	Manning's n =	0.013

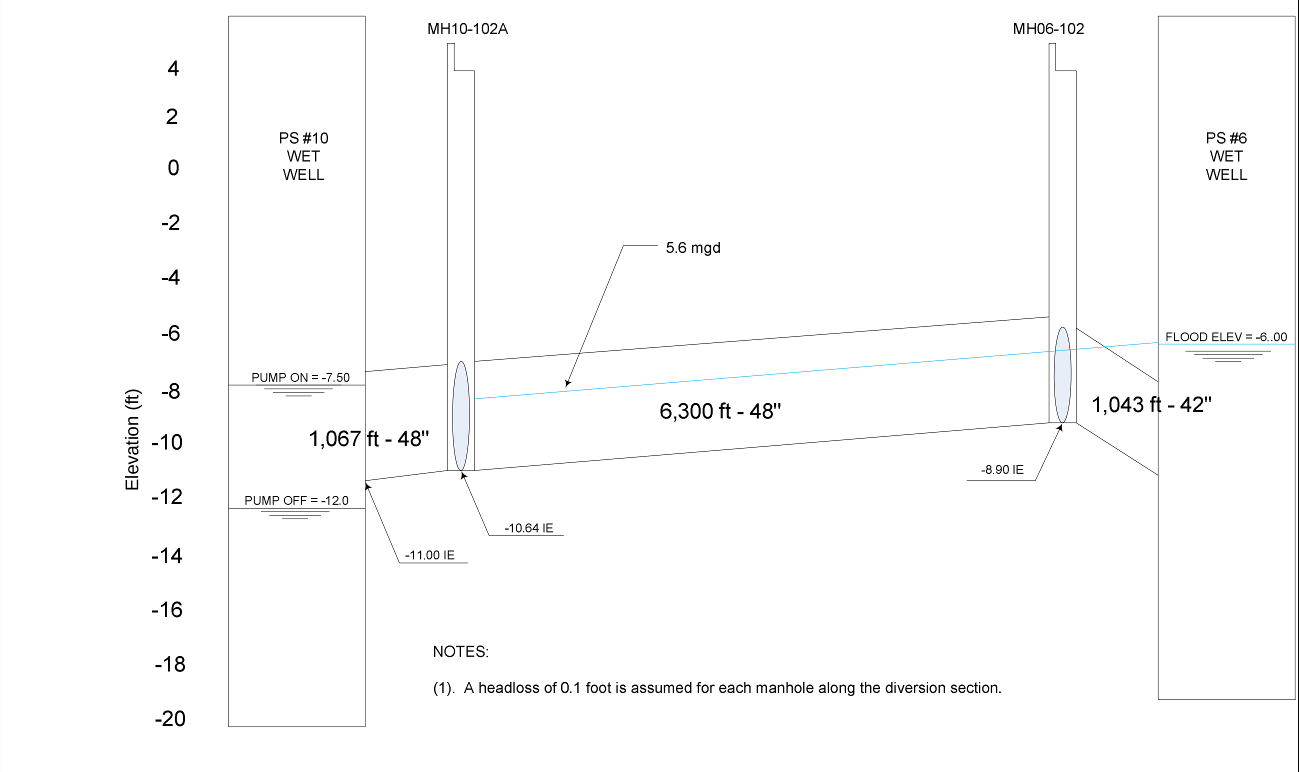
Assumed water surface elevation at discharge (MH10-102A) = -8.00

II. CALCULATE FLOW BY MANNING'S EQUATION

$Q = (1.49/n) * A*R^{2/3} *S^{1/2}$

$\Delta H = ((Q*n)/(1.49*A*R^{2/3}))^2 * L$

Diversion Flow, Q (mgd)	Diversion Flow, Q (cfs)	Head Loss, ΔH, in 42" Diversion Section (ft)	Head Loss, ΔH, in 48" Diversion Section (ft)	Water Surface Elevation at PS 6 (ft)
12.00	18.56	0.55	2.55	-4.90
10.00	15.47	0.45	2.23	-5.33
9.00	13.92	0.40	2.09	-5.51
8.00	12.38	0.36	1.97	-5.68
6.00	9.28	0.29	1.76	-5.95
5.55	8.59	0.28	1.72	-6.00
4.00	6.19	0.24	1.62	-6.14
2.00	3.09	0.21	1.53	-6.26
0.00	0.00	0.20	1.50	-6.30



EMERGENCY DIVERSION – PS 6 TO PS 10

APPENDIX 9 - EMERGENCY DIVERSION FROM PS 10 TO PS 6

Proposed Conditions

FIND: Wet well elevations at Pump Station No. 10 for various rates of "backflow" from PS 10 to PS 6.

I. PHYSICAL CHARACTERISICS OF DIVERSION

Length, L, of 48" overflow (ft) =	1,067	Length, L, of 48" overflow (ft) =	6,300
Pipe diameter, D (ft) =	4.00	Pipe diameter, D (ft) =	4.00
Pipe area, A (ft^2) =	12.5664	Pipe area, A (ft^2) =	12.5664
Hydraulic radius, R (ft) =	1.00	Hydraulic radius, R (ft) =	1.00
Manning's n =	0.013	Manning's n =	0.013

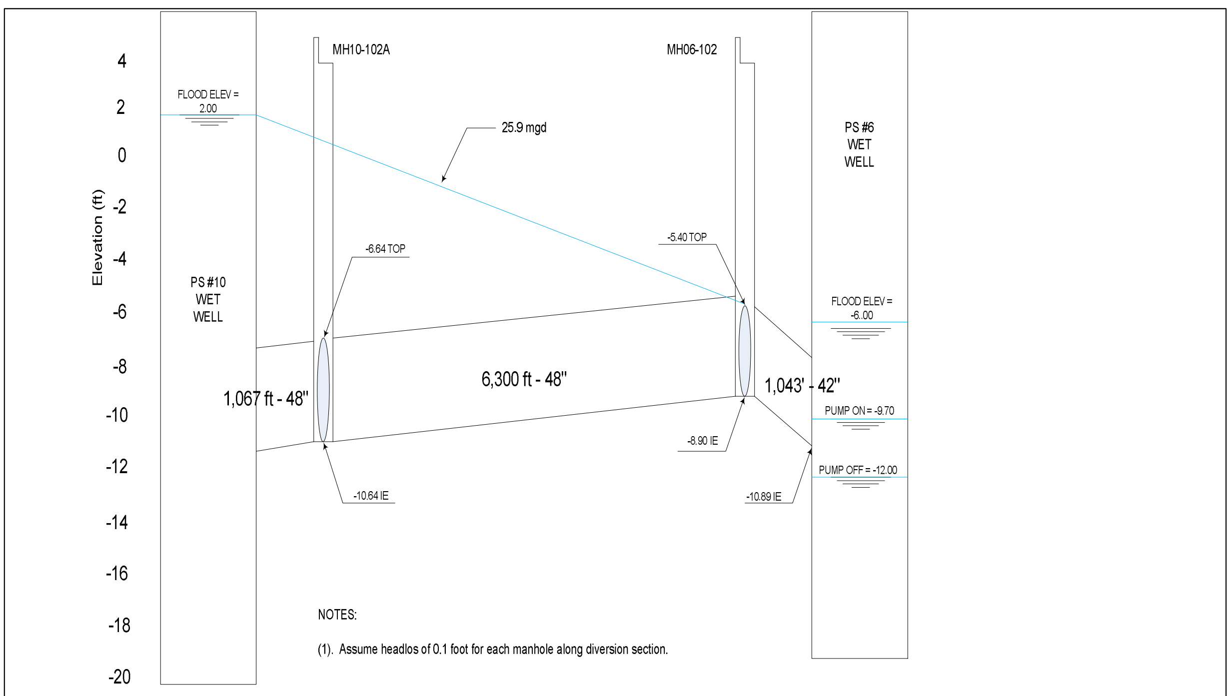
Assumed water surface elevation at discharge (MH06-102) = -5.40

II. CALCULATE FLOW BY MANNING'S EQUATION

$Q = (1.49/n) * A*R^{2/3} *S^{1/2}$

$\Delta H = ((Q*n)/(1.49*A*R^{2/3}))^2 * L$

Diversion Flow, Q (mgd)	Diversion Flow, Q (cfs)	Head Loss, ΔH, in 48" Diversion Section (ft)	Head Loss, ΔH, in 48" Diversion Section (ft)	Water Surface Elevation at PS 10 (ft)
30.00	46.41	1.31	8.04	3.95
28.00	43.32	1.17	7.20	2.96
27.00	41.77	1.10	6.80	2.50
25.90	40.07	1.03	6.38	2.00
24.00	37.13	0.91	5.69	1.20
22.00	34.03	0.80	5.02	0.41
20.00	30.94	0.69	4.41	-0.30
18.00	27.85	0.60	3.85	-0.95
16.00	24.75	0.52	3.36	-1.52



EMERGENCY DIVERSION – PS 10 TO PS 6

APPENDIX 10 - PS 6-10 CONNECTOR CHARACTERISTICS

Proposed Conditions -Alternate 1

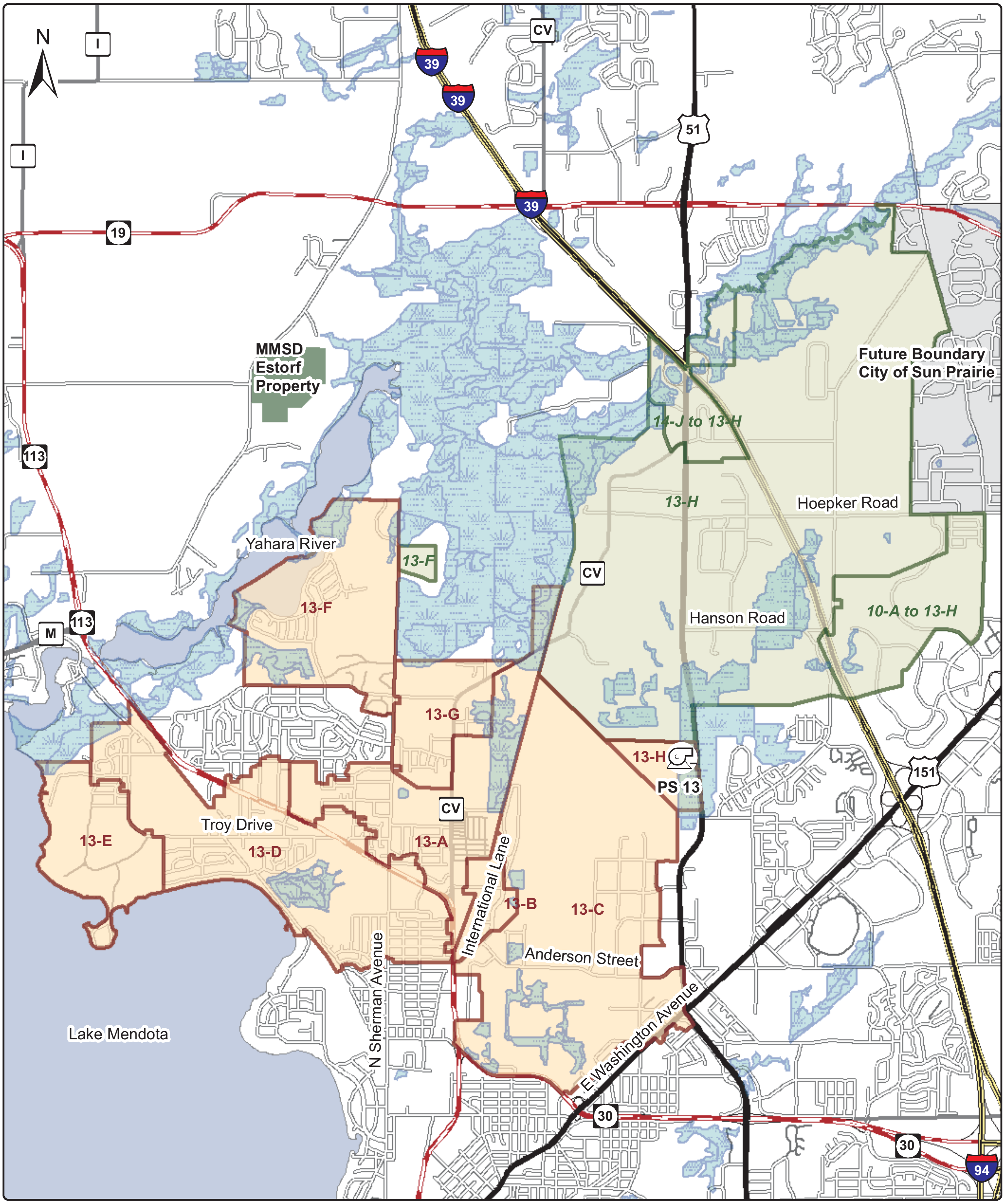
From Manhole	To Manhole	Upstream EI	Downstream EI	Pipe Length (ft)	Manhole Rim (ft)	Manhole Depth (ft)
MH06-102	Div1	-8.90	-8.98	261	3.0	12.0
Div1	Div2	-8.98	-9.07	308	5.0	14.1
Div2	Div3	-9.07	-9.17	340	4.5	13.7
Div3	Div4	-9.17	-9.32	498	4.0	13.3
Div4	Div5	-9.32	-9.46	488	5.0	14.5
Div5	Div6	-9.46	-9.59	434	9.0	18.6
Div6	Div7	-9.59	-9.67	294	17.2	26.9
Div7	Div8	-9.67	-9.77	325	19.2	29.0
Div8	Div9	-9.77	-9.91	470	20.0	29.9
Div9	Div10	-9.91	-10.01	358	22.5	32.5
Div10	Div11	-10.01	-10.12	352	13.0	23.1
Div11	Div12	-10.12	-10.27	513	16.0	26.3
Div12	Div13	-10.27	-10.44	590	3.0	13.4
Div13	Div14	-10.44	-10.61	552	2.0	12.6
Div14	MH10-102A	-10.61	-10.75	484	-2.0	8.8
TOTAL FOOTAGE =	6,267					
GRADE =	0.0295%					

Proposed Conditions - Alternate 2

From Manhole	To Manhole	Upstream El	Downstream El	Pipe Length (ft)	Manhole Rim (ft)	Manhole Depth (ft)
PS 6	Div1	-10.80	-10.81	292	8.5	19.3
Div1	Div2	-10.81	-10.83	565	10.2	21.0
Div2	Div3	-10.83	-10.86	696	11.0	21.9
Div3	Div4	-10.86	-10.88	572	12.0	22.9
Div4	Div5	-10.88	-10.89	400	14.0	24.9
Div5	Div6	-10.89	-10.92	658	13.0	23.9
Div6	Div7	-10.92	-10.94	587	13.0	23.9
Div7	Div8	-10.94	-10.95	330	10.2	21.2
Div8	Div9	-10.95	-10.96	319	9.0	20.0
Div9	Div10	-10.96	-10.98	516	12.0	23.0
Div10	PS 10	-10.98	-11.00	514	8.0	19.0
TOTAL FOOTAGE =	5,449					
GRADE =	0.0037%					

APPENDIX 11

10/23/2008



	Sub-Basins in 2000
	Potential by 2030
	Potential by 2060
	DNR Wetlands

PUMPING STATION 13 SUB-BASINS

MADISON METROPOLITAN SEWERAGE DISTRICT
COLLECTION SYSTEM EVALUATION
2008

0 0.5 1 2 Miles

FIGURE 3-34

APPENDIX 12

PUMP STATION 13 FLOW DIVERSION - EFFEFCTS ON DOWNSTREAM PUMP STATIONS

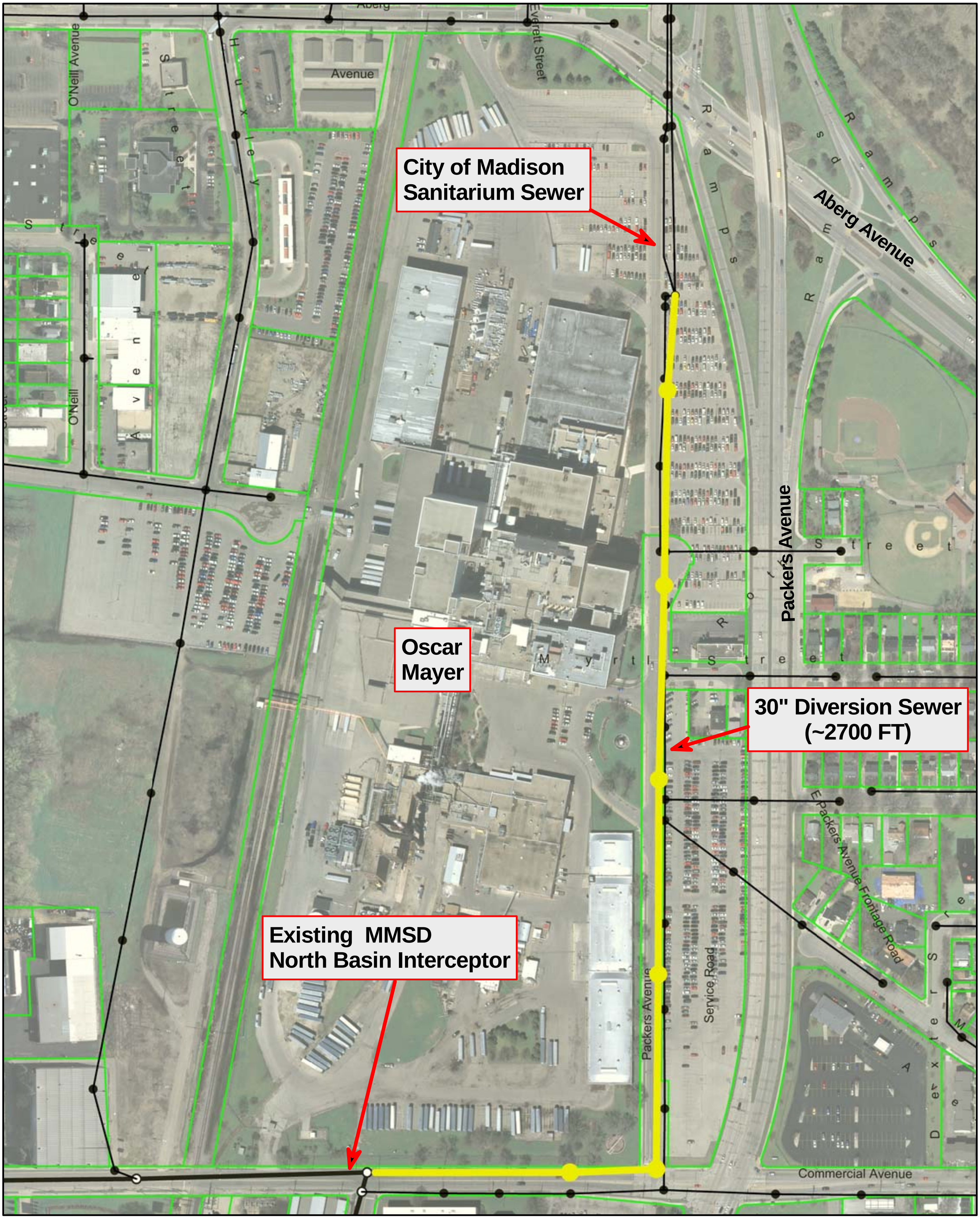
Average Daily Flows at Pump Stations

		No Diversion			With Diversion		
Pumping Station	Firm Pumping Capacity (mgd)	2030 TAZ Average Daily Flow (mgd)	2030 UF Average Daily Flow (mgd)	2060 Average Daily Flow (mgd)	2030 TAZ Average Daily Flow (mgd)	2030 UF Average Daily Flow (mgd)	2060 Average Daily Flow (mgd)
1	35.3	5.22	5.54	6.14	6.45	6.83	7.44
2	41.0	9.69	10.74	12.56	10.92	12.03	13.85
13	20.0	7.40	9.14	10.71	6.16	7.85	9.41
10	42.2	10.62	13.26	14.83	9.39	11.97	13.54
7	39.0	18.14	23.94	28.92	16.91	22.65	27.63

Peak Hourly Flows at Pump Stations

		No Diversion			With Diversion		
Pumping Station	Firm Pumping Capacity (mgd)	2030 TAZ Peak Hourly Flow (mgd)	2030 UF Peak Hourly Flow (mgd)	2060 Peak Hourly Flow (mgd)	2030 TAZ Peak Hourly Flow (mgd)	2030 UF Peak Hourly Flow (mgd)	2060 Peak Hourly Flow (mgd)
1	35.3	16.08	16.90	18.44	19.22	20.16	21.66
2	41.0	27.06	29.53	33.69	29.93	32.49	36.58
13	20.0	***21.56***	***25.77***	***29.44***	18.50	***22.67***	***26.42***
10	42.2	29.25	35.26	38.74	26.37	32.35	35.88
7	39.0	***45.90***	***59.85***	***72.30***	***43.26***	***56.63***	***69.07***

21.56 Peak Hourly Flow Exceeds Firm Capacity



MMSD	**Madison Metropolitan Sewerage District**	**Appendix 13- Pump Station 13 Diversion Sewer**	Prepared by: JP Scale: 1" = 300' Date: 5/14/10 N

APPENDIX 14

40-Year Present Worth Cost Analysis for Pump Station 13 Diversion

MAJOR CAPITAL COSTS FOR PS 13 DIVERSION

No.	Description	Footnote	Unit Cost	Units	Quantity	Total Cost
1	PS13 to PS1 Connector Sewer	(1)	$700	L.F.	2,700	$1,890,000
2	Upgrade PS13 Firm Capacity	(2)	$150,000	LUMP SUM	1	$150,000
3	NEI (Truax Extension) Relief Sewer	(3)	$850	L.F.	10,000	$8,500,000

Notes:

(1). MMSD 50-Year Master Plan (TM3) cites unit cost of $500 per foot for 30" gravity sewer. Unit cost for PS13 diversion sewer adjusted to account for utility congestion in construction corridor and for construction of new diversion structures at International Lane and Packers Avenue.

(2). Includes cost for one centrifugal pump and drive, analogous to existing Pump 13C, in empty slot at PS13. No other rehabilitation work is included in estimate. Earth Tech's Design Report for PS 13 & 14 Firm Capacity Improvements (2007) suggests that future firm capacity of 30.7 MGD can be achieved by providing a new pump similar to capacity of existing Pump 13C.

(3). Project limits are MH10-145 to MH10-121. Estimated construction cost is based on actual unit cost of $950/ft for MMSD's NEI (PS 10 to Lien Road) project and unit cost of $800/ft as taken from TM 3 of *MMSD 50-Year Master Plan* .

(4). All costs in 2010 dollars.

APPENDIX 14

40-Year Present Worth Cost Analysis for Pump Station 13 Diversion

Project Description	Year	Capital Cost		O&M Costs		Salvage Value		Total 2010 Present Worth
		Cost in Year Constructed	2010 Present Worth	Annual Cost in Year Constructed	2010 Present Worth	Year 2050	2010 Present Worth	
No Diversion								
PS 13 to PS 1 connector	N/A	0	0	0	0	0	0	0
Upgrade to PS 13 Firm Capacity	2020	206,000	150,000	710	14,430	52,000	13,000	151,430
NEI (Truax Extension) Relief Sewer	2032	16,997,000	8,500,000	5,300	43,200	12,918,000	3,263,000	5,280,200
Total Pumping Costs for Diversion Flow	2010	0	0	49,300	3,195,000	0	0	3,195,000
TOTALS								**8,630,000**
With Diversion to PS 1								
PS 13 to PS 1 connector	2020	2,590,000	1,890,000	987	21,000	1,554,000	392,000	1,519,000
Upgrade to PS 13 Firm Capacity	2036	340,000	150,000	1,220	7,000	221,000	56,000	101,000
NEI (Truax Extension) Relief Sewer	2043	24,035,000	8,500,000	7,800	16,800	21,792,000	5,504,000	3,012,800
Total Pumping Costs for Diversion Flow	2010	0	0	52,900	3,429,000	0	0	3,429,000
TOTALS								**8,060,000**

Assumptions and Notes:

(1). Base interest rate = 3.5%
(2). Construction cost escalation rate = 3.2%
(3). Interceptor Service Life (yrs) = 75
(4). Pump/Drive Service Life (yrs) = 40
(4). Annual O&M interceptor cost ($/ft) = 0.25
(5). O&M costs increase at the base interest rate.
(6). Energy escalaction rate = 6.0%
(7). Pumping Costs for PS 13, 10 & 7 = $110/Mgal.
(8). Pumping Costs for PS 1 & 2 = $118/Mgal.
(9). Timing for replacement of facilities determined from CARPC's flow projections utilizing Traffic Analysis Zone data *(2009 MMSD Collection System Evaluation).*

Appendix A4
Interceptor Maintenance Guidelines

MADISON METROPOLITAN SEWERAGE DISTRICT

Protecting Public Health and the Environment

INTERCEPTOR MAINTENANCE PROGRAM GUIDELINES

Original: July, 1999
1st Revision: January, 2000
2nd Revision: April, 2008
3rd Revision: November, 2009

Table of Contents

List of Tables

Introduction

MMSD's wastewater collection system currently includes 17 regional pumping stations, 95 miles of gravity interceptors, 44 miles of forcemains (which includes 15 miles of effluent forcemains and 29 miles of raw wastewater forcemains), and 1,594 manholes. The statistics of the MMSD collection system are summarized on the following page.

The MMSD collection system is an important part of the public works infrastructure in the metropolitan area, and is continuously responsible for transmitting over 40 mgd of raw wastewater to the Nine Springs Wastewater Treatment Plant. The collection system also represents a large investment, with an estimated replacement value over $200 million for the pipeline facilities and over $100 million for the pumping stations.

The purpose of this document is to present a set of guidelines for the maintenance of MMSD’s 139-mile system of interceptors and forcemains. These guidelines represent an updated version of the MMSD Interceptor and Forcemain Maintenance Plan that was originally prepared in November of 1992. These guidelines incorporate much of the original plan, but also reflect various changes and strategies that have occurred at MMSD since 1992. Updated aspects of these guidelines include improved methods for systematic workflow & recordkeeping, availability of computerized maintenance management, contracted field locating services, Diggers Hotline membership, development of MMSD's GIS program, and promotion of cross-training.

MMSD Collection System Statistics

Note that statistics apply only to active structures and main segments

Number of Pumping Stations 17 **Today's Date: 10/30/2009**

Length of Main Segments

	Feet	Miles
Gravity	501,667	95.01
Forcemains	234,625	44.44
All Mains	736,292	139.45

	Feet	Miles
Badfish Creek Effl Diversion Forcemain	26,269	4.98
Badgermill Creek Effl Diversion Forcemain	53,765	10.18
Total All Effluent Diversion Forcemains	80,034	15.16
Total All Forcemains w/o Effluent Diversion	154,591	29.28
Total All Mains w/o Effluent Diversion	656,258	124.29

Number of Structures

Gravity Manholes	1,501
Forcemain Manholes	93
All Manholes	1,594
Shut-off Valves	23
Air-Release w/ Automatic Valve	43
Air-Release w/o Automatic Valve	9

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As of October 30, 2009

Table 1: Interceptor Maintenance Program At-A-Glance							
	1 Interceptor Evaluations (TV & Cleaning)	**2** Forcemain Gate Valve Exercising	**3** Air Valve Inspection & Maintenance	**4** Siphons	**5** Stoplog & Flapgate Structures	**6** Special Projects, Events and Repairs (Individual items)	**7** Program Coordination & Management
Scope	• Locate MHs • Evaluate flow • Prepare specs • Bid & award • Monitor work • Review inspection results • Log condition into database	Exercise each isolation valve on MMSD forcemains	Inspect each air valve location twice per year. More often if an active valve is problematic. Clean & repair active air release valves as needed.	Clean each siphon via contracted services and inspect access structures on each end of siphon.	Inspect each stoplog, flapgate, structure, etc. Repair as needed.	• MH repairs • Emerg. repairs • High flow events • I/I work • Complaints • Utility coordination • Inspections • Field measurements • Surface Route Insp.	• Planning • Budgeting • Inventories • Contract services • Diggers & Locator • Cross-training • UTILITY log • Preparedness
Quantity	95 miles total in gravity system	21 isolation valves currently in-service (as of Nov. 2009)	• 52 locations total: • 36 active • 11 manual valve only (not auto). • 2 removed • 3 vent pipe only	11 active siphons	• 20 locations total: • 16 active • 4 removed	As needed. Create individual w/o for each specific event	Involves numerous people from different Departments.
Frequency	Approx. 10% of system each year = 8 to 10 miles/yr.	Each valve twice per year	Each active valve twice per year	Each location twice per year	Each active location twice per year	As needed	As needed and on-going
Lead Responsibilities	• CS Supervisor • Sewer Maint. Crew for field work	Sewer Maint. Field Crew, w/direction from CS Supervisor	Sewer Maint. Field crew, w/direction from CS Supervisor	Contracted services, w/direction from CS Supervisor	Sewer Maint. Field crew, w/direction from CS Supervisor	CS Supervisor. & Sewer Maint. Crew as needed. Additional help from Engr. and O&M as required.	Diggers Hotline, Locating Services & UTILITY log managed by Engr. Dept.
Estimated Crew Time	240 manhours, assuming 2 men for 3 weeks, once/yr.	160 manhours, assuming 2 men for 2 hours per valve, twice/yr.	300 manhours, assuming 2 men for 2 hours per valve, twice/yr.	Work bid on a 2 or 3 year basis. Sewer Maint. Crew to assist as req'd.	120 manhours, assuming 2 men for 2 hrs per valve, twice/yr.	480 manhours. Rough estimate. Individual projects and events will vary from year to year.	Budgeting by CS Supervisor, O&M Dir. & Engr. Dir. Other tasks by CS Sup. & staff as needed.
Work Order Comments	• One WO each year for all work related to TV'ing and Cleaning.	• Two WO's each year. • 21 tasks on each WO. • See Table 2	• Two WO's each year. • 36 tasks on each WO. • See Table 3	• One WO each year • 11 tasks on each WO • See Table 4	• Two WO's each year • 16 tasks on each WO • See Table5	• Create WO's for each event. • Track costs to asset • Costs and time will vary from year to year.	• Create WO's for each task. • 9901005 UTILITY screening • 9901006 Diggers Hotline & Locating

Scope of the Work

Table 1 is a summary of the overall interceptor maintenance program at a glance. As shown, the program has been divided into seven areas or subprojects. Each of these areas is outlined below. Program staffing and recordkeeping are discussed in later sections of this document.

Area 1: Interceptor Evaluations

- MMSD formalized its annual Interceptor Evaluation program in the early 1990's.
- The purpose of the evaluations is to keep MMSD current on the physical condition and hydraulic adequacy of its individual gravity interceptors, and to allow informed decisions regarding the need for significant rehabilitation or replacement projects.
- The program includes televising, cleaning, manhole inspection, flow documentation, and various other work. See the detailed work outline attached as an appendix to this document.
- Interceptor evaluations have been performed on roughly 10% of MMSD's gravity mileage each year (i.e. an average of about 9 miles per year).
- The program has been successful in identifying system needs prior to their becoming emergencies, and has allowed MMSD to more efficiently plan, budget and carry out the necessary repairs and rehab projects.
- Project examples have included MMSD's East Interceptor Replacements Phase III and IV, East Interceptor Rehab/Relining Phase V, South Interceptor Replacement, West Interceptor Replacement at UW Campus, PS2 Forcemain Replacement, Crosstown Forcemain Replacement, North Basin Interceptor, and numerous cured-in-place lining projects.
- The interceptor evaluation program seems to work for MMSD, and should be continued at the rate of approximately 10% per year. An average evaluation interval of about 10 years is a reasonable time frame for a gravity interceptor facility.
- The main new strategies are aimed at the systematic recordkeeping and organization of the work. See program staffing and recordkeeping sections of these guidelines.

Area 2: Forcemain Isolation Valve Exercising

- Table 2 summarizes the exterior isolation valves which formerly existed or which currently exist in MMSD's collection system (not including valves located inside pumping stations). Of the 27 valves listed, 6 have been abandoned/removed and 21 are active (i.e., in-service).
- Some MMSD forcemains were designed with isolation valves just outside of the station, with the primary function to limit possible pumproom flooding in the event of a burst header inside the station.
- In other special cases, isolation valves were added at specific forcemain junction points to allow diversion of flow as part of a construction project.

- Many of the older MMSD isolation valves are double-disc gate valves. As discussed in Sanks and MMSD's Technical Memo, the double disc gate valve is not a particularly good choice for wastewater, since the seating area can become filled with grit and solids, preventing full seating of the valve. At their time of installation, however, double disc valves were the accepted standard for water and wastewater.
- Newer isolation valves (those typically installed after the mid-1990's) are either resilient wedge gate valves or plug valves. These are designed to close better in the presence of grit and solids contained in wastewater.
- Each valve should be regularly exercised and inspected by twice per year.
- Valve exercising verifies that the stem and gearing remain accessible and the valve is in working order.
- Note that valve exercising does not automatically verify that the valve is fully sealing off the flow. Some valves may leak, even though their valve stem exercises freely to closure, and may require additional work to fully close the valve.
- In 1998, MMSD purchased a hydraulic valve operator that is permanently mounted on one of MMSD's trucks. Most buried MMSD valves are accessible by this truck-mounted valve operator, thus allowing the valve to be exercised via power. However, a few valves still require manual operation (i.e., turning by hand).

Table 2: Force Main Isolation Valves

#	Forcemain	MH Station	Comments	Map Sheet
1	Old PS2 FM (30") at Brittingham Park	2-0207 ("Valve 1")	30" double disc gate valve, 1963. **ABANDONED DURING PS2FM REPLACEMENT IN AUGUST 2001.**	23.3 Madison
2	Crosstown FM at Brittingham Park	2-0035 ("Valve 2")	20" double disc gate valve, 1914. **ABANDONED DURING CROSSTOWN FM REPLACEMENT IN 2003.**	23.3 Madison
3	Crosstown FM at Brittingham Park	XT-0095R ("Valve 3")	20" resilient wedge gate valve, 1997. **ABANDONED DURING CROSSTOWN FM REPLACEMENT IN 2003.**	23.3 Madison
4	Crosstown FM at Bedford Street	XT-3420	20" double disc gate valve, 1914. **ABANDONED DURING CROSSTOWN FM REPLACEMENT IN 2003.**	23.4 Madison
5	Old PS3 FM before junction with old 30" PS2 FM	2-17010	8" hand-operated gate valve. **ABANDONED DURING PS2FM REPLACEMENT IN AUGUST 2001.**	30.3 Bl. Grove
6	Old PS4 FM before junction with old 30" PS2 FM	4-0120	16" gate valve, 1967. **ABANDONED DURING PS2FM REPLACEMENT IN AUGUST 2001.**	25.3 Madison
7	PS5 FM near PS5	5-22885	16" Val-Matic plug valve in valvebox, 1996. Normally open. Closes cw, 20 turns.	18.4 Madison
8	PS5 FM at junction with PS15FM	5-22384	16" double disc gate valve, 1959. Normally open. **Closes ccw**, 78 turns. NOTE: This valve is broke in the open position. It is not routinely exercised.	18.4 Madison
9	PS7 FM (1963) in vault in front of PS7	7-8526	36" double disc gate valve, 1963. Normally open. **Closes ccw**.	20.3 Bl. Grove

Table 2: Force Main Isolation Valves… continued				
#	**Forcemain**	**MH Station**	**Comments**	**Map Sheet**
10	PS7 FM (1963) at NSWTP near Storage Building No. 1.	7-1551	36" double disc gate valve, 1963. Normally open. Closes cw.	30.3 Bl. Grove
11	PS7 FM (1948) at NSWTP near Storage Building No. 1.	7-1546A	36" double disc gate valve, 1963. Normally open. Closes cw.	30.3 Bl. Grove
12	PS9 New FM (1987) in valve box at PS9	9-20582	14" double disc gate valve, 1987. Normally open. Closes cw, 43 turns.	3.2 Dunn
13	PS9 Old FM (1961) in manhole at PS9	9-20594	10" double disc gate valve, 1961. Normally closed. Opens ccw, 28 turns.	3.2 Dunn
14	PS15 Old FM (to West Interceptor/PS8) at Allen Blvd.	15-1360	24" double disc gate valve, 1974. Keep valve open for flow to WI / PS8. Close valve to divert flow to PS16. Closes cw, 74 turns.	12.4Middleton
15	PS15 New FM (diversion to PS16) at Allen Blvd.	15-5587	30" double disc gate valve, 1982. Open for flow to PS16. Closes cw, 70 turns. Note: this valve can be left open even when pumping to WI / PS8.	12.4Middleton
16	New PS2 FM. Behind PS2, closest to bldg. (Valve 1)	10+00	24” Val-Matic plug valve, 2001. Normally open. Closes cw, 60 turns.	23.3 Madison
17	New PS2 FM. Behind PS2, further from bldg. (Valve 2)	10+00	24” Val-Matic plug valve, 2001. Normally closed. Opens ccw, 60 turns.	23.3 Madison
18	PS4 to PS2 bypass. SW of PS2, near air release MH.	11+32	16” Val-Matic plug valve, 2001. Normally closed. Opens ccw, 20 turns.	23.3 Madison
19	New PS2 FM, prior to PS4 tee (behind PS4, near RR).	109+25	36” Val-Matic plug valve, 2001. Normally open. Closes cw, 87 turns.	25.3 Madison
20	New PS2 FM, after PS4 tee (behind PS4, near RR).	109+41	36” Val-Matic plug valve, 2001. Normally open. Closes cw, 87 turns.	25.3 Madison
21	PS4 FM, prior to connection with new 36” PS2 FM.	109+33	16” Val-Matic plug valve, 2001. Normally open. Closes cw, 20 turns.	25.3 Madison
22	PS3 FM, prior to connection with new 36” PS2 FM.	173+28	8” resilient wedge gate valve, 2001. Normally open. Closes cw, 26 turns.	30.3 Bl. Grove
23	New XTFM. Behind PS2, furthest from bldg. (Valve 3)	0+20 (On connection)	30” Val-Matic plug valve, 2003. Normally open. Closes cw, 80 turns.	23.3 Madison
24	New XTFM. At SW corner of PS1.	9+69	24” resilient wedge gate valve, 2000. Normally open. Closes cw, 73 turns.	6.3 Bl. Grove
25	PS15 FM at junction with PS 5 FM	15-7264	24” resilient wedge gate valve. Normally open. **Closes ccw**, 78 turns. NOTE: This valve is broke in the open position. It is not routinely exercised.	18.4 Madison
26	PS10FM drain valve (at low-point of forcemain)	10-23080	6” plug valve with blind flange. ¼ -turn to open or close.	9.1 BlGr.
27	BM Creek Effluent Return	305+05	6” Waterous resilient wedge gate valve, 19 turns. Used for golf course irrigation trial.	3.3 Fitchburg

Area 3: Air Valve Inspection and Maintenance

- Table 3 summarizes the air valves previously within or currently active within the MMSD collection system. These include 52 valves total: 36 “active” locations with automatic air valves; 11 “active” locations with manual gates valves (not automatic); 2 locations that have been removed; and 3 locations with standpipes (vents) that are open all the time.
- Most of MMSD’s air valves are “combination” valves, i.e. they perform both a vacuum breaking function and an air release function.
- The vacuum breaking function admits air into the forcemain during low pressure conditions (such as during pump shutdowns), thus preventing possible vapor cavity formation & water column separations which could lead to waterhammer failures.
- The air release function prevents air pockets from accumulating and potentially restricting the flow at forcemain high points.
- To ensure that each valve remains in working order, each air valve should be inspected and cleaned twice each year. In some cases it may be possible to clean and repair the valve in the field. In most cases, the valve should be removed and returned to the shop where it can be inspected and cleaned prior to reinstallation at the site.

Table 3: Air Valve Locations

#	Forcemain	MH Station	Location & Comments	Map Sheet
1	PS02	2-17710	NSWTP near Metrogro Storage Tank odor beds. ***No air valve at this site. MH and valve removed during 10th addition.***	30.3 BlGr.
2	PS07 (1963)	7-6750	Engel St. near WPS. ***MH with 2” gate valve and ARI automatic valve. 2” gate valve N.C. Opened only as-needed.***	29.2 BlGr.
3	PS08	8-4009	Under Beltline Nob Hill viaduct. ***Manual valve only. No automatic valve at this site.***	36.1 Mad.
4	PS08	8-8079	Bram St. near Coliseum. ***Removed in 2008. Manual valve only. No automatic valve.***	25.3 Mad.
5	PS08	8-11264	1722 Kenward St. ***Removed in 2008. Manual valve only. No automatic valve.***	26.4 Mad.
6	PS09	9-1500	Between Paulson Road & Railroad	34.3 BlGr.
7	PS10	10-24760	Hwy 51 East R.O.W. south of Robertson Rd.	4.4 BlGr.
8	PS11	11-1073	NSWTP near Metrogro Storage Tank odor beds. ***No air valve at this site. MH and standpipe removed during 10th addition.***	30.3 BlGr.
9	PS15 (to West Int.)	15-1525	2045 Allen Blvd. near Univ. Ave. ***No automatic air valve at this site. 2” gate valve in MH for manual air release.***	12.4 Midltn
10	PS15 (to West Int.)	15-2411	Thorstrand Rd. @ University Ave. ***No automatic air valve at this site. 2” gate valve in MH for manual air release.***	13.1 Midltn
11	PS15 (to West Int.)	15-4827	Capital Drive @ University Ave. ***No automatic air valve at this site. 2” gate valve in MH for manual air release.***	18.2 Mad.
12	PS15 Diversion to PS16	16-106	St. Dunstan's Drive. ***MH with 2” gate valve and automatic valve. 2” gate valve N.C. Opened only as-needed.***	13.1 Midltn

Table 3: Air Valve Locations… continued				
#	**Forcemain**	**MH Station**	**Location & Comments**	**Map Sheet**
13	PS17	17-2050	Bruce Street	22.3 Ver.
14	PS17	17-3050	Locust Drive	22.3 Ver.
15	PS17	17-4113	Hwy. M east of Locust Drive	22.4 Ver.
16	PS17	17-8900	South of Verona Rd. and West of Hwy PB	14.3 Ver.
17	BM Creek Effluent	6650	Near Goose Lake. South of USH 18/151 and West of Fitchrona Road.	12.4 Ver.
18	BM Creek Effluent	10200	4' Dia MH. 2" ball valve and 2" galvanized steel ***standpipe***. There is also a 1" corporation stop in the MH. No automatic air valve.	7.3 Fitch
19	BM Creek Effluent	12900	4' Dia MH. 2" ball valve and 2" galvanized steel ***standpipe***. There is also a 1" corporation stop in the MH. No automatic air valve.	7.2 Fitch
20	BM Creek Effluent	29050	Longford Terrace	4.4 Fitch.
21	BM Creek Effluent	42000	McCoy Rd. near RR	2.4 Fitch.
22	BM Creek Effluent	44450	McCoy Rd. near Hwy 14	1.2 Fitch.
23	BM Creek Effluent	46500	Clayton Road	1.2 Fitch.
24	BM Creek Effluent	53720	NSWTP north of Moorland Road	30.3 BlGr.
25	Effluent 54"	2300	NSWTP north of Moorland Road	30.3 BlGr.
26	Effluent 54"	7090	North of Meadowview Road	31.3 BlGr.
27	Effluent 54"	11800	North of Goodland Park Road	6.3 Dunn
28	Effluent 54"	13478	Lalor Road south of Goodland Park Road	7.2 Dunn
29	Effluent 54"	16575	Lalor Road	7.3 Dunn
30	Effluent 54"	20250	Lalor Road	18.2 Dunn
31	Effluent 54"	25808	Back of 2399 White Oak Trail. ***Standpipe only. No air valve at this site.***	19.1 Dunn
32	New 36" PS02	11+24	50' SW of PS2	23.3 Mad.
33	New 36" PS02	69+36	Corner of Van Deusen & Rowell Streets	26.1 Mad.
34	New 36" PS02	111+81	South of PS4, along RR tracks. ***Trial in-progress in 2009 to determine if automatic valve can be removed. Gate valve only. Inspected for air every two weeks.***	25.3 Mad.
35	New 36" PS02	151+52	South of Nob Hill Road, near bike path	36.1 Mad.
36	New 30" XT	7+41	Brittingham Park at bike path intersection	23.3 Mad.
37	New 30" XT	33+26	Next to Boathouse at Bedford Street	23.4 Mad.
38	New 30" XT	38+17	Between bike path and North Shore Drive	23.4 Mad.
39	New 30" XT	45+27	Near tennis courts, south of Broom Street	24.2 Mad.
40	New 30" XT	103+61	RR embankment north of Monona Terrace	24.2 Mad.
41	New 30" XT	113+90	Median of E. Wilson, in front of Essen Haus	13.3 Mad.
42	New 30" XT	117+43	Between MG&E and RR tracks, north of Blair	13.3 Mad.
43	New 30" XT	121+61	MG&E parking lot south of Blount Street	13.3 Mad.
44	New 30" XT	127+13	Bike path, between Blount & Livingston	13.4 Mad
45	New 30" XT	135+72	Bike path, between Livingston & Patterson	13.4 Mad.
46	New 30" XT	139+60	Bike path, between Patterson & Brearly	13.4 Mad.
47	New 30" XT	146+75	Bike path, between Brearly & Ingersol	13.1 Mad.
48	New 30" XT	157+29	East Wilson Street at Few Street	13.1 Mad.

Table 3: Air Valve Locations… continued				
#	**Forcemain**	**MH Station**	**Location & Comments**	**Map Sheet**
49	New 30” XT	179+85	Median of E. Wash. Ave, south of Thornton	7.2 BlGr
50	New 30” XT	174+98	Between E. Wash. Ave. and Dickinson St.	7.2 BlGr
51	PS07 (1948)	7-5385	Automatic 6” Air Release Valve installed 2002. Adjacent to 7-6750 MH. ***6” gate valve and Vent-O-Mat automatic valve. 6” gate valve N.C. Opened only as-needed.***	29.2 BlGr.
52	PS01	09300 +/-	30”x 4” tapping sleeve, 4” companion flange, 2” SS nipple, and 2” ball valve installed in 2006. East Wash Ave @ 2nd Street. ***No automatic valve. Manual air release only.***	6.3 BlGr.

Area 4: Siphon Cleaning

- Table 4 summarizes the 11 active inverted siphons currently owned by MMSD.
- As of 2009, nine of the eleven MMSD siphons are cleaned twice per year. Due to its’ length, the WI West Point Extension siphon at Pheasant Branch Creek is not routinely cleaned (i.e., it is classified as a forcemain). The WI Campus Relief siphon on Randall Avenue is also not routinely cleaned.
- The purpose of a siphon is to carry the wastewater flow beneath an obstacle (such as a streambed or a major utility line) which would otherwise block the interceptor’s gravity profile.
- One disadvantage of a siphon is that it typically carries a lower velocity (since it always flows full) and thus creates greater potential for solids deposition. Newer siphons with multiple barrels are designed to minimize the potential for solids deposition.
- MMSD has generally not experienced significant problems with its siphons, except for the Shorewood Hills siphon. That siphon has needed numerous cleanings over the years due to grease accumulation, and has been the responsibility of the City of Madison since it was constructed in conjunction with a City storm sewer project.
- MMSD began contracting out the regular cleaning of its siphons in 1998. Prior to 1998, siphons were cleaned only if specific problems occurred. These services are typically contracted for a two or three year period.
- It is recommended that MMSD continue its’ current program of contracted siphon cleaning. This should help to catch any problems before they become serious.
- The contractor’s cleaning operations should be observed, and the adjacent siphon manholes should be visually inspected at the time of cleaning to determine if any additional work is needed.

Table 4: Siphons						
#	**Interceptor**	**Location**	**Manholes**	**Year**	**Comments**	**Map Sheet**
1	WI West Point Ext.	Pheasant Branch Creek at Hwy. M	5-116 to 5-115A	1966 & 1957	2094 ft. of 14" AC pipe. ***Due to length, classified as a forcemain. Not routinely cleaned.***	1.4 Middleton
2	West Int. Relief	Walnut Street Underpass at Campus Drive	2-517 to 2-516	1959	105 ft. of 36" RCP	21.1 Madison
3	Old West Interceptor	Midvale Blvd. at University Ave.	2-054A to 2-053B	1958	31 ft. of 16" CI pipe installed in 1958 to clear new storm sewer box conduit	20.1 Madison
4	Old West Interceptor	Shorewood Blvd. north of University Ave.	2-047B to 2-047A	1972	21 ft. of 15" RCP installed in 1972 to clear City storm sewer. City agreed to maintain siphon.	20.1 Madison
5	West Int. Replacement at UW Campus	Randall Avenue at Wendt Engineering Library	No manholes	1999	120 ft. of 30” DI installed in 1999 to clear twin UW chilled water lines and MGE gas line. ***No manholes…not routinely cleaned.***	22.1 Madison
6	West Int. Spring Street Relief	Brooks Street at College Court	2-309B to 2-309A	1975	46 ft. of 24" CI pipe installed in 1975 to clear 5’x12’ storm box	22.1 Madison
7	West Int. Spring Street Relief	Brooks Street at Regent Street	2-309 to 2-308	1940	91 ft. of 24" CI pipe	22.1 Madison
8	West Int. Spring Street Relief	Brooks Street at Milton Street, near Meriter Hospital	2-307 to 2-306	1965	63 ft. of 24" CI pipe	23.3 Madison
9	South Int. Baird Street Relief	Wingra Creek at Baird Street	4-312 to 4-311	1995	Two barrels, 156 ft. of 14" and 10" DI pipe inside of 36" steel casing, grouted in place.	26.4 Madison
10	Southeast Int.	Siggelkow Road underpass at USHwy 51	7-218A21 to A20 to A19	1961 & 1992	185 ft. of 8" DI and CI pipe (145 ft. replaced with DI in 1992)	34.3 Bl. Grove
11	East Monona Interceptor	Fair Oaks Avenue at Starkweather Creek	6-108F to 6-108E	1925	85 ft. of 14" CI pipe, crossing Starkweather Creek	5.4 Bl. Grove
NA	INACTIVE: Old West Int.	Regent Street at Murray Street	2-005A to 2-005	1968	50 ft. of 24" CI pipe. Flow diverted to City sewer in 1995	23.3 Madison

Area 5: Stoplog & Gate Structures

- Table 5 lists the 20 stoplog and gate structures located within the MMSD collection system. Of these, 16 are currently in-service and 4 have been removed/abandoned.
- Some of these structures are overflows to nearby streams or lakes. These should be inspected during high flow events to make sure the nearby waterway is not overflowing into the collection system.
- Some of these structures were constructed at junction points between adjacent interceptor projects and are used to divert flow from one interceptor to another.
- Others were originally constructed as flushing manholes (no longer used) for the purpose of periodic flushing of the interceptor with adjacent surface water.

To ensure that the stoplog and flapgate structures remain in good condition, are at the correct elevation, and not leaking, MMSD should inspect each structure twice per year and provide any stoplog or gate replacements or repairs that are needed.

Table 5: Stoplog and Gate Locations

#	Facility	MH	Location & Comments	Map Sheet
1	Bedford Street Stoplogs.	CT-3420	Northshore Drive at end of Bedford Street, adjacent to Monona Bay.	23.4 Mad.
2	Burke Outfall Stoplog for diversion to 30”	93+10	Pennsylvania Ave south of Commercial Ave. ***Abandoned/removed during North Basin Interceptor project.***	31.3 Burke
3	PS5 Stoplog	5-403	Mendota Drive across from PS5	18.4 Mad.
4	PS6 Flapgate	6-102	Drainage ditch near PS6	5.4 Bl. Gr.
5	PS7 Stoplog	PS7	Entrance chamber behind PS7	20.3 Bl. Gr.
6	PS8 Stoplog at Wingra Creek	8-100	North side of Wingra Creek across from PS8	26.3 Mad.
7	SWI Junction MH for emergency diversion from PS2 to PS8.	8-106	Haywood Street at Wingra Drive, near entrance to Arboretum. ***Slide gate normally removed, allowing overflow to PS2. Gate stored in MH.***	26.2 Mad.
8	SEI Flushing Valve (upstream of PS9)	9-108	East side of Hwy. 51, north of Yahara River, south of Yahara Drive. ***Gate valve to remain closed always.***	3.2 Dunn
9	NEI Flapgate upstream of PS10	10-114	At Starkweather Creek, south of Sycamore Ave and west of Walsh Rd. ***Removed in 2009 during NEI-PS10 to Lien Road Project.***	33.4 Burke
10	PS11 Flapgate	PS11	PS11 near entrance chamber	31.3 Bl. Gr.
11	NSVI MP Ext. Flapgate upstream of PS12	12-113	Along Badger Mill Creek, north of Nesbitt Road and west of Maple Grove Road. ***Flap gate removed in 2004 during City Greenway Modification Project. MH remains.***	12.3 Verona

Table 5: Stoplog and Gate Locations… continued				
#	**Facility**	**MH**	**Location & Comments**	**Map Sheet**
12	NEI Truax Ext Flapgate upstream of PS13	13-105	Along drainage ditch, west of Hwy 51 at Dane County Airport access road. ***Inside airport perimeter fence.***	20.1 Burke
13	PS15 Slidegate with hole for gravity diversion to PS5	5-102A	130 feet south of PS15 along Allen Blvd., in Marshall Park.	12.4 Middl.
14	WI Relief junction with Old WI, allowing overflow to old WI d/s	2-513	South side of Campus Drive across from Veterinary Science ***Abandoned/removed during WI-Campus Relief Phase 4 Project***	22.2 Mad
15	WI Campus Relief Phase 1 junction with WI Relief.	8-207	At UW Met. Engineering Bldg. Stopgates allow stopping either leg d/s. ***Gates normally removed and open to flow both ways.***	22.1 Mad
16	WI Campus Relief Phase I junction with Old WI	8-206	Randall Ave just south of RR. Stopgates allow stopping either leg d/s. ***Gates normally removed and open to flow both ways.***	22.1 Mad
17	WI Relief junction with Old WI	2-014A	Randall Ave. south of Dayton St. Slide gate blocks flow to Old WI d/s. ***Gate always in-place and flow is always blocked to Old WI.***	22.1 Mad
18	WI Randall Relief cross-connect with Old WI at MH 2-012B	8-122	Randall Ave. between Spring Street and Regent Street. ***Gate always in-place, but if flow is 2.5' +/- above invert of MH 8-122 it will overflow to MH02-012B in the Old WI.***	22.1 Mad
19	WI Spring Street Relief cross-connect with Old WI	2-316B	Randall Ave. south of Monroe Street. ***Gate always in place. Diverts flow from Old WI (Monroe Street) into the WI Spring Street Relief.***	22.1 Mad
20	PS16 Overflow to Gammon Extension	5-230	Gammon Road, just west of PS16. ***Brick dam to divert gravity flow from PS16 to PS5 via the WI Gammon Ext.***	13.2 Middl.

Area 6: Special Projects, Repairs and Events

- Areas 1 through 5 above represent the regular planned maintenance activities.
- Area 6 includes the numerous specific projects, repairs and events that occur every year in the operation and maintenance of interceptors and forcemains.
- Examples include high flow events, emergency repairs, connection inspections, odor complaints, backup events, I/I work, specific manhole repairs, surface route inspections, and other events.

- As discussed later under the Recordkeeping section, a separate workorder should be created for each specific event as it comes up.
- These specific events are an important aspect of an interceptor maintenance program, and maintaining a record of these events will be helpful for future decisions and management of the MMSD program.

Area 7: Coordination and Management Functions

Coordination and management of the interceptor maintenance program includes numerous functions needed to make the program successful. Examples include the following.

- Preparing annual program budget and tracking it during the year. This is typically performed by the Collection System Supervisor and Director of O&M.
- Tracking and documenting work performed and work outstanding. This is typically performed by the Collection System Supervisor.
- Updating interceptor GIS database and maps. This is typically performed by GIS personnel in the Engineering Department.
- Managing inventory. This is typically performed by the Collection System Supervisor.
- Managing annual siphon cleaning and TV & Clean contracts. This is typically performed by the Collection System Supervisor.
- Managing Diggers' Hotline membership and locating services. This is typically performed by the Engineering Department.
- Organization of emergency preparedness. This is typically performed by the Collection System Supervisor
- Screening projects being done by other utilities and municipalities via the UTILITY log (spreadsheet). This is performed by the Engineering Department.
- Organizing cross-training activities.
- Recommending periodic improvements to the program.

Program Staffing

The proposed staffing plan outlined below is a team approach, and a joint effort of several departments, employees and outside resources.

Collection System Supervisor

- The interceptor maintenance program is to be managed primarily by the Collection System Supervisor. Oversight of the program will be provided by the Director of Operations & Maintenance and Director of Engineering. Assistance will be provided by the Engineering Department staff whenever necessary.
- Planning, budgeting, prioritizing, tracking, and management of the program will be accomplished via a joint effort between the Collection System Supervisor, Director of O&M, Director of Engineering, and Engineering Department staff. Work will be tracked and documented through the Computerized Maintenance Management System.
- The role of Collection System Supervisor focuses on organizing and supervising the day-to-day field operations and seeing that they are successfully carried out.
- The Collection System Supervisor personally conducts much of the field "reconnaissance" work, i.e. monitoring contractors, attending preconstruction meetings, inspecting connections, addressing complaints, meeting with property owners, etc.
- The Collection System Supervisor should consult with the Director of O&M, the Director of Engineering, and Engineering Department staff on a regular basis to keep others informed of day-to-day operations, decisions, and observations made in the field.
- The Collection System Supervisor should schedule work for the field crew, monitor the results of the field work, hire outside contractors, and other transfer knowledge to MMSD staff as needed. All are essential to the program's success.
- The Collection System Supervisor will organize the work, create the necessary workorders, and recruit help as required from the Buildings & Grounds Supervisor.

Field Crew

- Personnel from the Monitoring Services/Sewer Maintenance Crew will carry-out the day-to-day field work needed for specific interceptor maintenance activities.
- If necessary, the Building and Grounds Crew will provide members to assist the Monitoring Services/Sewer Maintenance Crew when needed for specific interceptor maintenance activities.
- Regular planned activities requiring field crew participation are as follows:
  a) Manhole field locations prior to annual televising/cleaning.
  b) Semiannual gate valve exercising.
  c) Semiannual air valve inspection & maintenance.
  d) Semiannual inspection and maintenance of special structures.
  e) Various special projects and emergencies, as required.

- Per Table 1, the anticipated Field Crew commitment is estimated at roughly1300 manhours/yr., but this may vary from year to year.
- Through cross-training, involving different personnel, and assigning hands-on projects to different people, it is desired to build up a significant knowledge of the MMSD interceptor system in members of the Monitoring Services/Sewer Maintenance field crew.

Outside Services

- Heavy construction work, major repairs, excavation, and specialty services should typically be contracted out to private firms. The Collection System Supervisor or Engineering Department will coordinate this work.
- Contracting out such work frees MMSD from the cost of owning and maintaining extensive specialty equipment (i.e., backhoes, vactor trucks, etc.) and allows MMSD to focus on what is does best: Managing the overall collection system.
- Examples of efficient outside services for MMSD’s interceptor maintenance have included televising & cleaning work, surveying work, field marking, excavation work, emergency excavation & repairs, significant construction work, etc.

Other Staff Resources

- The Collection System Supervisor should recruit the participation of other MMSD staff whenever needed for specific advice, engineering evaluation, emergencies, etc.
- Examples include map updates by the GIS/CAD specialist, UTILITY project screening, assistance by the Engineering Department during emergency events, etc.
- Major projects that become identified through interceptor maintenance will need to be budgeted and assigned to a project manager. This will be done by the Director of Engineering though the annual capital budgeting process.

Recordkeeping and CMMS

General Organization

- The overall interceptor maintenance program has been packaged as "INT MAINT" within the Project module of MMSD's CMMS system.
- The Project "INT MAINT" is subdivided into seven Subprojects corresponding to the seven work areas shown on Table 1.

Creating Workorders

- When creating an interceptor maintenance workorder, it should typically be linked to one of the seven subprojects under "INT MAINT".
- When entering the work order description, the name of the facility involved, e.g. NEI, PS8 Forcemain, etc., should typically be included in the description.
- **Subproject 1: Interceptor Evaluations**. One work order should be created each year for all work associated with the TV/Clean/Evaluation project that year.
- **Subproject 2: FM Gate Valve Exercising.** A semiannual activity, two work orders should be created each year. Each work order should have tasks for each valve location that requires valve exercising.
- **Subproject 3: Air Valve Inspections.** A semiannual activity, two work orders should be created each year. Each work order should have tasks for each air valve location that requires inspection.
- **Subproject 4: Siphons.** A semiannual activity, one work order should be created for the entire year (both cleanings). The workorder should be "tied" to the purchase order for the contractor hired to clean the siphons. The workorder should have eleven tasks, one for each siphon location.
- **Subproject 5: Stoplog & Flapgate Structures.** A semiannual activity, two work orders should be created each year. Each work order should have tasks for each structure that requires inspection.
- **Subproject 6: Special Projects, Events and Repairs.** Most of the workorder activity will take place in this subproject. Individual work orders should be created for each significant project, event or repair. If a specific event will involve more than a few hours of time, or if it's simply an event that's worth documenting, a separate work order should be created to track the work.
- **Subproject 7: Program Coordination & Management.** For work not related to one specific event or asset (i.e., the overall collection system) or work that takes less than a few hours to complete, the standing workorders shown in Table 1 should be used. These include for General Coordination, UTILITY Log and Diggers Hotline/Locating Services. Note: These should be used as little as possible. Specific workorders related to the event or asset should be created and used whenever possible.

Finishing and Closing Workorders

- The Collection System Supervisor should frequently search through the list of all active "INT MAINT" workorders and all workorders related to the collection system maintenance to determine what work is outstanding and to guide daily workflow.
- Whenever an item has been completed, the Sewer Maintenance Crew and/or Collection System Supervisor should enter a comment under the "Notes" field. The Note should briefly indicate what was done, who did it, and the date it took place. These "Notes" are one of the main benefits of having a CMMS and are a great way to document observations, problems, and fixes.
- After the work is finished and "notes" have been entered, the Collection System Supervisor should change the TASK to "finished".
- After all tasks on a work order have been finished (most work orders will have just one task), the Collection System Supervisor should change the work order to "closed". Note: The CMMS will not allow the work order to be "closed" until the day following the "finishing" of the last task.

Generating Lists and Reports

- Various reports and search capabilities are available or are being developed within the CMMS.
- The CMMS Work Order Selection Search provides on-screen lists of work orders. The user can designate desired workorders by status (active, closed, etc.), by Account No., by Subproject, by date, etc.
- The ACCESS database "Employee Timekeeping" report shows staff hours and $ amounts for a specified calendar year.
- The ACCESS database "Total Cost of WO's by Crew" is a departmental report listing all workorders in chronological order, along with total costs for each.
- The ACCESS database "Employee Hours by WO" shows each individual employee's time charged for a specific selected workorder.
- The ACCESS database "WO Total Cost w/ Hours & Mtls" report shows detailed costs for a specific selected workorder.
- The CMMS report writing and usage is still a developing area at MMSD. Personnel should look for the reports that are most useful to Interceptor Maintenance Program, and provide suggestions for any modifications that would be helpful.

Reference Documents for Interceptor Maintenance

Numerous documents and sources of information are available for reference when working with the MMSD interceptor system. Some of the most useful references are listed below.

- MMSD Collection System Map Book (hard copies)
- MMSD GIS and Mapping
- MMSD Collection System Database. This database provides valuable information concerning the details of the MMSD collection system.
- MMSD Collection System Inspection Database. This database provides detailed results of the annual televising and cleaning of MMSD interceptors
- MMSD Emergency Response Manual provides important emergency contacts, phone numbers, and forcemain emergency repair information
- MMSD Forcemain Profiles. These drawings provide detailed profiles at-a-glance for each forcemain. (Electronic files are located on the network and hard copies are located in the maintenance files. Numerous personnel also have hard copies of the profiles).
- Interceptor Maintenance Files (hard copy) are in the file room maintenance section, organized by interceptor and pumping station. These include hard copies of correspondence, memos, etc.
- Original as-built project construction plans (hard copy) are located in the file room on the plan racks.
- The Computerized Maintenance Management System (CMMS) database (see discussion above).
- Shared network drives, which include project documentation and various documents related to maintenance, including these guidelines.
- The "MMSD Collection System Evaluation", prepared by the staff of the Capital Area Regional Plan Commission. This was last completed in 2008.
- The MMSD "Collection System Facilities Plan". This includes a comprehensive look at the entire MMSD Collection system, from both a capacity and condition aspect. The original plan was completed in 2002, with an update scheduled for completion in 2010.

As paper copies become superseded by electronic information, an ongoing goal will be to consolidate the relevant information in the most effective way for easy access. The document management system, CMMS reporting system, GIS mapping, and databases will be warehouses for much of the interceptor maintenance information. Use of the network drives and OnBase should also be encouraged to store key spreadsheets, documents, tables, etc. for easy access and sharing.

Summary

This document provides guidelines for MMSD's interceptor maintenance program. It is an updated version of MMSD's original 1992 Interceptor Maintenance Plan. The interceptor maintenance program has been organized as a separate project called "INT MAINT" within MMSD's CMMS system, and is divided into seven main work areas as summarized in Table 1. The program is staffed as a team effort of several departments and employees, including the Collection System Supervisor, the Monitoring Services/Sewer Maintenance Crew, personnel from Buildings and Grounds as required, outside contractors, and other MMSD staff as needed. The program is intended to be a flexible and cost-efficient approach to interceptor maintenance. The program managers are encouraged to look for opportunities to improve the program whenever possible.

APPENDIX NO. 1

Work Sequence Guidelines
For
Interceptor Evaluations

<u>Appendix No. 1</u>

Interceptor Evaluations

Detailed Work Sequence Guidelines

a) Budget. Recommend and budget for the particular interceptor system(s) desired to be evaluated in the following year. Aim for an overall average of about 10% per year, but allow this to vary from year to year in order to evaluate entire interceptor systems as a unit wherever possible.

b) Pre-inspect. Pre-inspect the entire route of the proposed evaluation project. Identify any manhole access problems, special property issues or other conditions that might affect the proposed contractor televising and cleaning operations.

c) Document actual flows. The two main objectives of the interceptor evaluations are to evaluate the *physical condition* and the *hydraulic adequacy* of the interceptor system. To address hydraulic adequacy, it is important to document actual measured flow rates in the key branches of the system. In some cases, flow information may be directly available from an upstream or downstream pumping station flow meter. Due to multiple interceptor branches, however, pumping station records alone will often be insufficient to determine the desired interceptor flows. Contracted installation of temporary flow vs. time meters has been used successfully by MMSD and should be considered for key interceptor branch locations. One week contracted installations are fairly inexpensive and have provided both the average and the time distribution of flow, depth and velocity.

Use the documented *average* flow and the Greeley and Hansen formula to compute the peak flow. Compare this to the nominal pipe capacity (based on the Manning equation) to determine the hydraulic adequacy of the interceptor. Also use the measured flow information to determine whether or not special flow control measures (for example, diversion pumping, night-time televising) will be needed for proper cleaning and televising.

d) TV and Cleaning Specs. Prepare specifications for contracting the cleaning and televising of the interceptor system to be evaluated. Use MMSD's standard format, and keep this standard spec up-to-date with desired new features (for example, pan and tilt camera technology). In preparing the specs, give special consideration to any access problems or easement issues. Also, specifically indicate any flow control or diversion requirements and any night-time work requirements.

e) Advertise, Bid and Award. Advertise, bid and award the televising and cleaning contract work.

f) Contractor's Field Work. Prior to the start of the field work, notify any property owners and municipal public works departments that may be affected by the work. Monitor the contractor's field operations to ensure that the work is proceeding in accordance with the specifications.

g) Map Edits. Review MMSD's collection system maps during the pre-inspection and during the field work. Do the MMSD maps correctly show the interceptor? Is the information shown on the maps accurate? Make note of any changes or corrections needed (for example, sewer lengths, incoming connections, etc.) and route these to MMSD's GIS/CAD specialist for incorporation.

h) Tape review and Pipe Condition Log. Review the contractor's completed televising tapes and summarize the pipe condition using MMSD's pipe rating log. Enter the rating data into the Collection System Inspection database (see attached).

i) Evaluation memo. Prepare a summary evaluation memo which documents the results of the above items and which provides specific recommendations for any follow-up action. The memo should be concise, but should cover each of the following:

- Is the interceptor pipe structurally adequate? Or does rehabilitation or replacement need to be considered?
- Document the average interceptor flows and address the interceptor's hydraulic adequacy.
- Are the manholes in satisfactory condition, or are specific repairs needed?
- Document the estimated total gpm of clearwater infiltration, and recommend whether or not the specific sources are cost effective to repair.
- Note any corrections or additions to be made to the GIS collection system maps or data. Attach marked-up map copies and forward to the GIS/CAD Technician for incorporation.
- Provide recommendations for any action and/or work required.

Interceptor Condition Evaluation Form

Date Evaluated: _______________

Recorded by: ______________

Ranking System (Choose One)
- **N: None**
- **I: Insignificant**
- **M: Moderate**
- **S: Severe**

Type of Pipe
- **(RCP) Reinf. Concrete Pipe**
- **(CI) Cast Iron Pipe**
- **(DI) Ductile Iron Pipe**
- **(VC) Vitrified Clay Pipe**
- **(AC) Asbestos Cement Pipe**
- **(PVC) Plastic Pipe**

Pipe Defects
- **(BP) Broken Pipe**
- **(CP) Cracked Pipe**
- **(CO) Corrosion**
- **(FO) Flow Obstructions**
- **(GA) Gasket Problems**
- **(GR) Grease Buildup**
- **(IB) Iron Buildup**
- **(JB) Joint Buildup**
- **(OJ) Offset Joint**
- **(R) Roots (before cleaning)**

I / I (gpm)
Estimate the total gpm of leakage in the entire pipe section

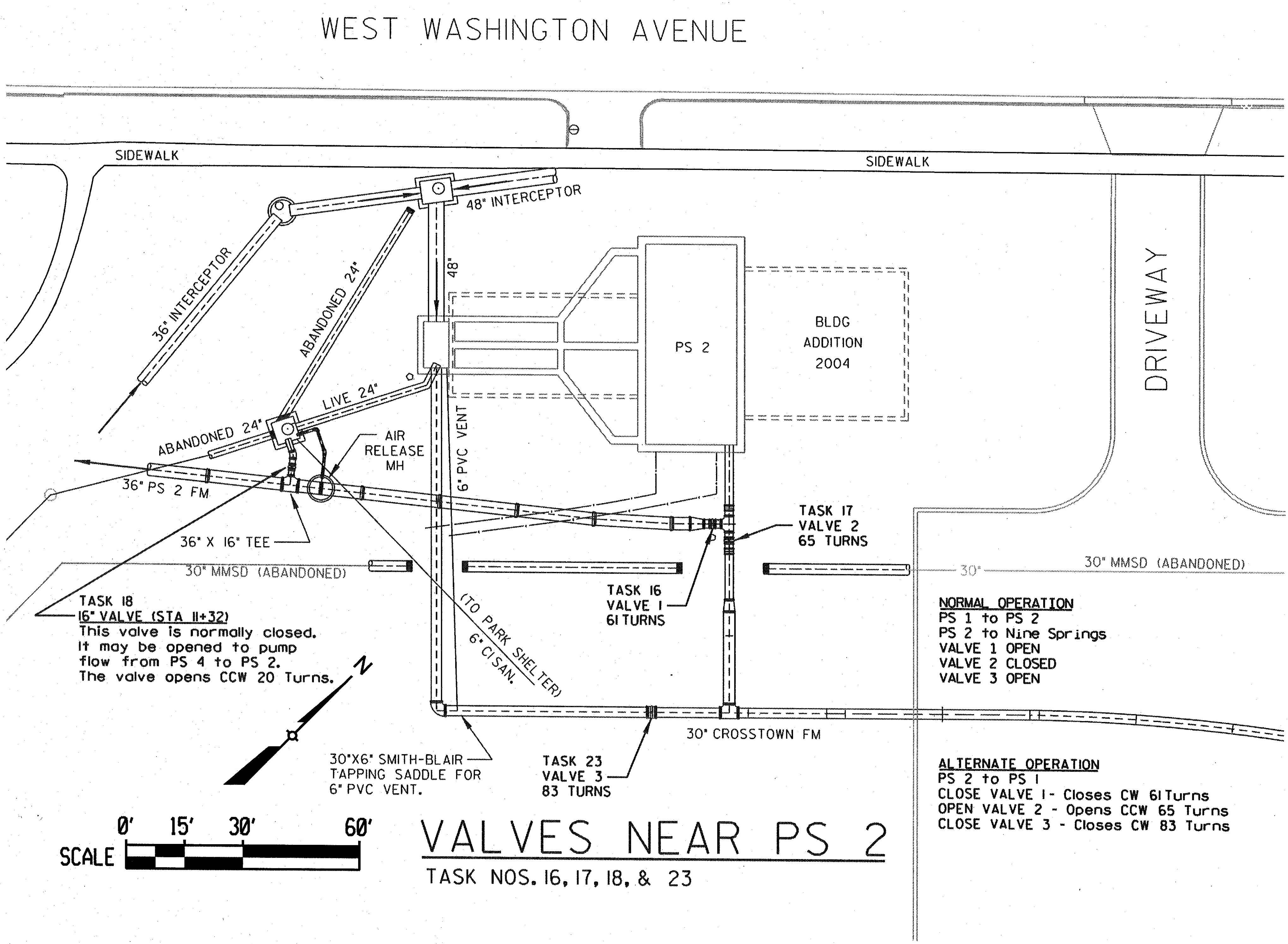
MH Defects
Using N, I M, S system, rank the Manhole at the *Downstream end of each pipe section*

Interceptor Name	Pipe Section		Date	Pipe	Pipe Defects										I / I	MH	Other Remarks
	From	To	Televised	Type	BP	CP	CO	FO	GA	GR	IB	JB	OJ	R	gpm	d / s	

Interceptor Reporting Form

APPENDIX NO. 2

Maps of Certain Valve Clusters



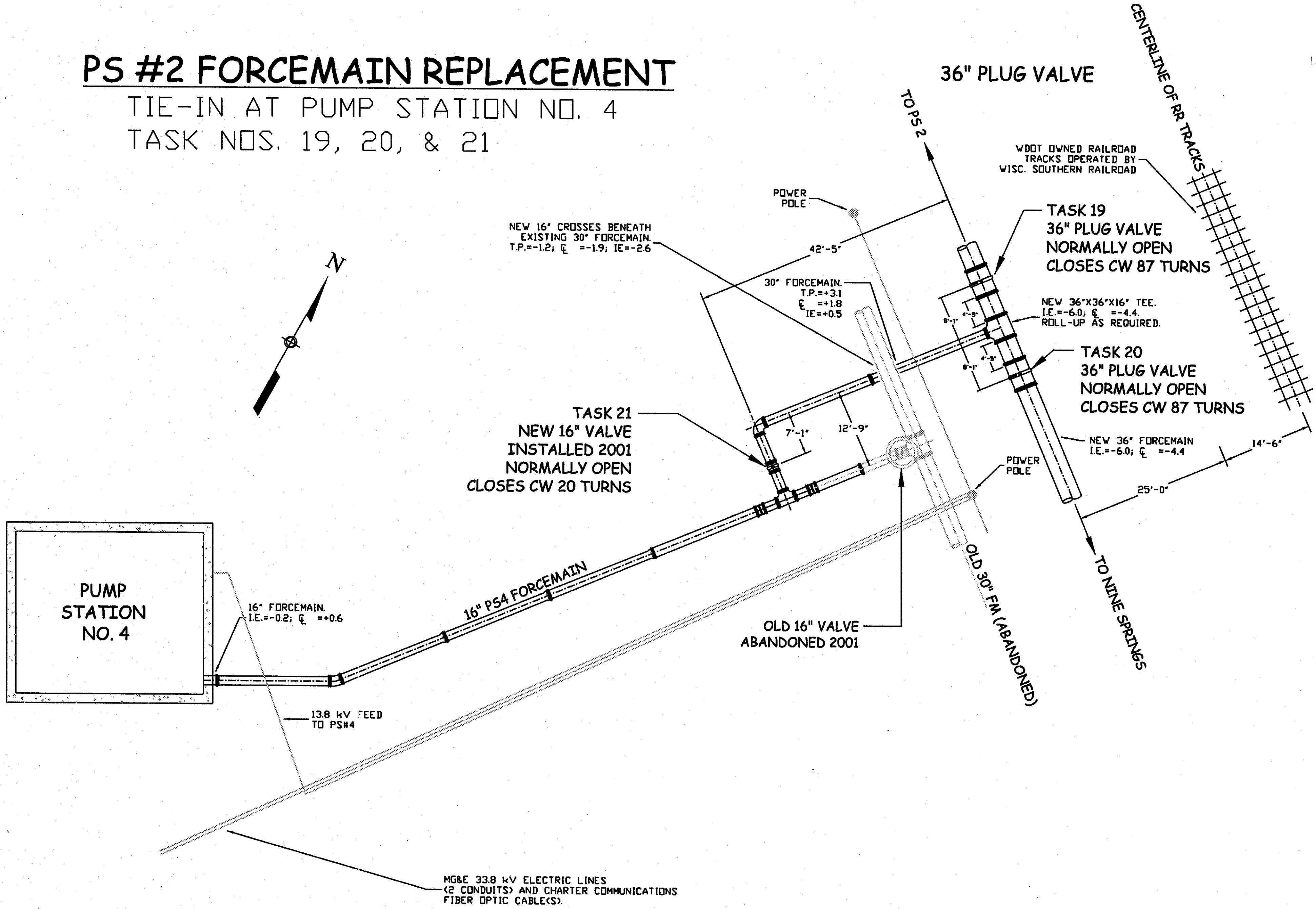
VALVES NEAR PS 2

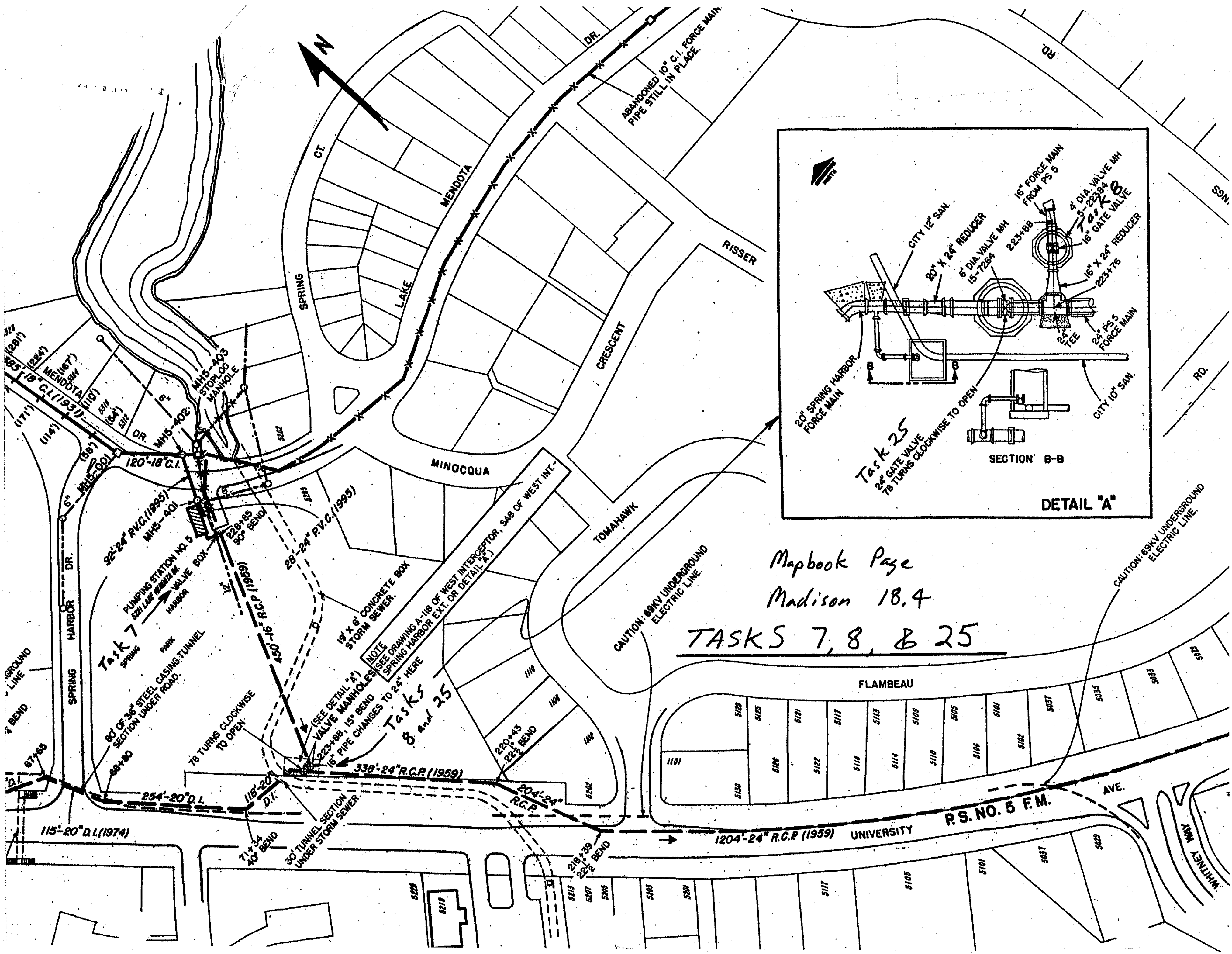
TASK NOS. 16, 17, 18, & 23

PS #2 FORCEMAIN REPLACEMENT

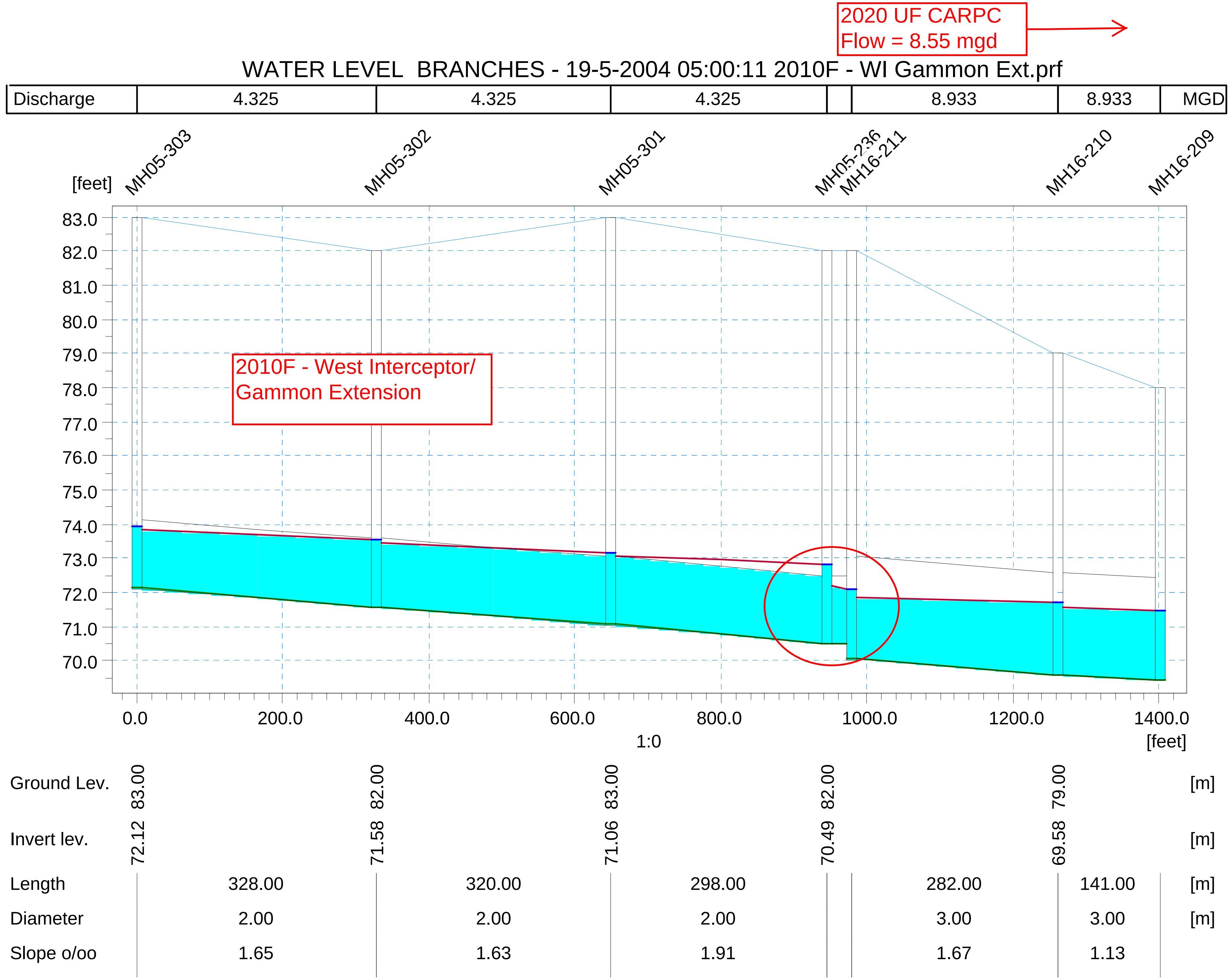
TIE-IN AT PUMP STATION NO. 4

TASK NOS. 19, 20, & 21

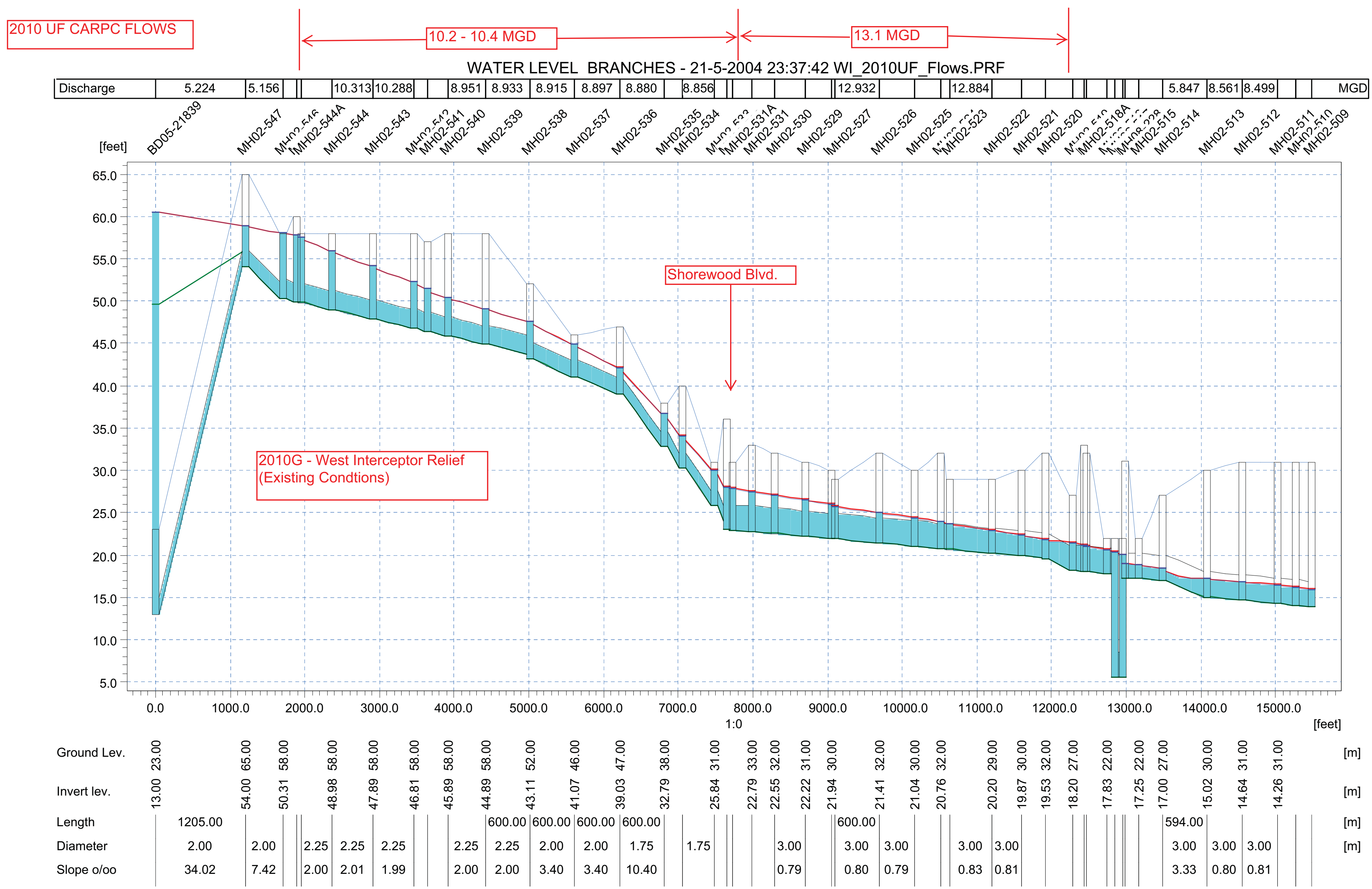



Mapbook Page
Madison 18.4
TASKS 7, 8, & 25
N
MENDOTA
LAKE
DR.
SPRING
CT.
ABANDONED 10" C.I. FORCE MAIN PIPE STILL IN PLACE.
RISSER
RD.
CRESCENT
MINOCQUA
TOMAHAWK
FLAMBEAU
UNIVERSITY
AVE.
WHITNEY WAY
SPRING HARBOR DR.
MH5-403 STOPLOG MANHOLE
MH5-402
MH5-401
MH5-001
120'-18" C.I.
18" C.I. (1931)
92'-24" P.V.C.(1995)
28'-24" P.V.C.(1995)
228+85 90° BEND
PUMPING STATION NO. 5
VALVE BOX
Task 7
SPRING HARBOR PARK
450'-16" R.C.P. (1959)
19' x 6' CONCRETE BOX STORM SEWER.
NOTE SEE DRAWING A-118 OF WEST INTERCEPTOR, SAB OF WEST INT.-SPRING HARBOR EXT. OR DETAIL "A".)
(SEE DETAIL "A") VALVE MANHOLES
223+88, 15° BEND
16" PIPE CHANGES TO 24" HERE
Tasks 8 and 25
78 TURNS CLOCKWISE TO OPEN
80' OF 36" STEEL CASING TUNNEL SECTION UNDER ROAD.
67+65
68+80
115'-20" D.I.(1974)
254'-20" D.I.
118'-20" D.I.
71+34 40° BEND
30' TUNNEL SECTION UNDER STORM SEWER.
338'-24" R.C.P. (1959)
220+43 22½° BEND
204'-24" R.C.P.
218+39 22½° BEND
1204'-24" R.C.P (1959)
P.S. NO. 5 F.M.
CAUTION: 69KV UNDERGROUND ELECTRIC LINE.
CAUTION: 69KV UNDERGROUND ELECTRIC LINE.
CITY 12" SAN.
20" X 24" REDUCER
6' DIA. VALVE MH 15-7264
223+88
16" FORCE MAIN FROM PS 5
4' DIA. VALVE MH 5-22384
Task 8
16" GATE VALVE
16" X 24" REDUCER
223+76
24" TEE
24" PS 5 FORCE MAIN
20" SPRING HARBOR FORCE MAIN
CITY 10" SAN.
Task 25
24" GATE VALVE 78 TURNS CLOCKWISE TO OPEN
SECTION B-B
DETAIL "A"

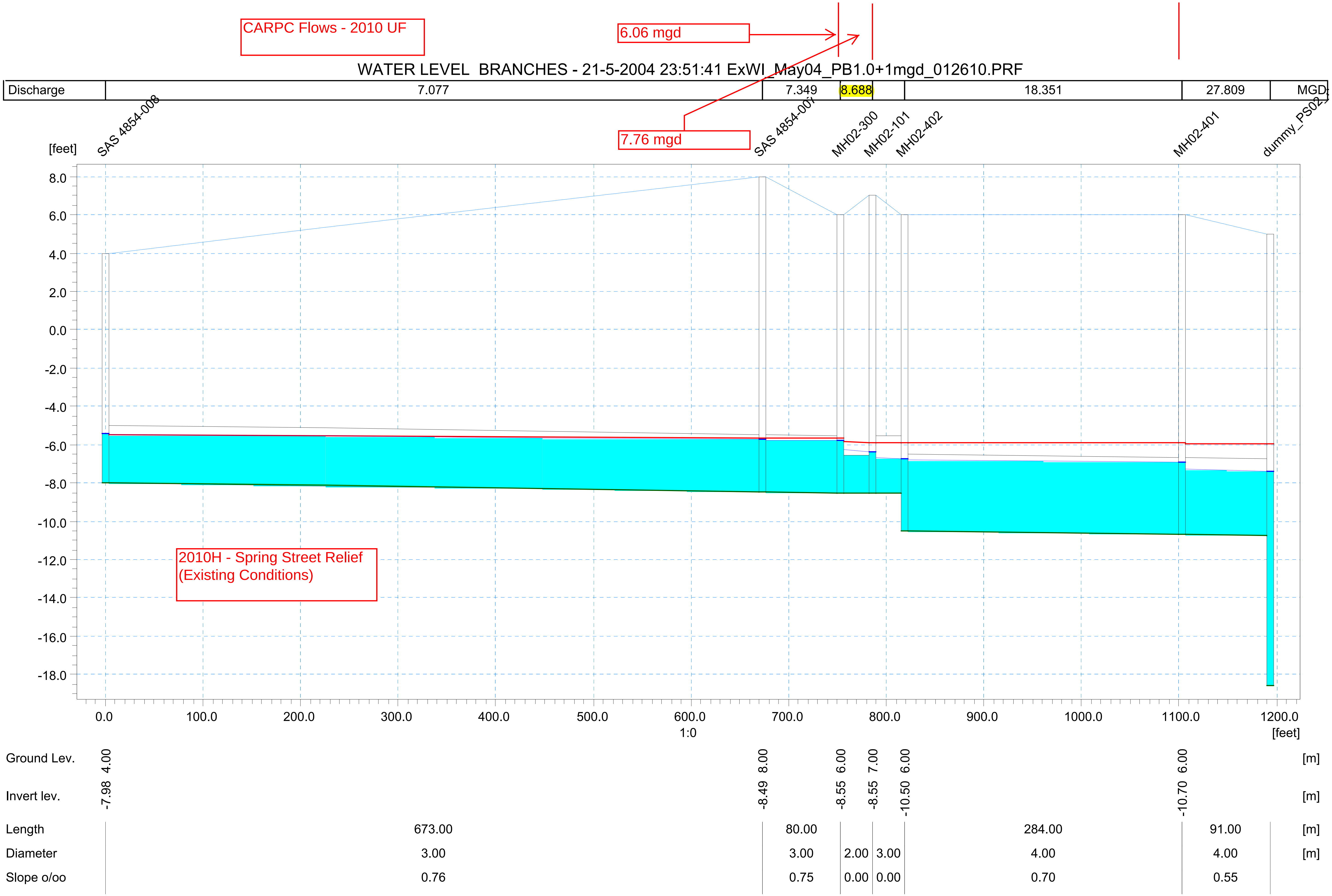
Appendix A5
Hydraulic Modeling Results



2020 UF CARPC
Flow = 8.55 mgd
WATER LEVEL BRANCHES - 19-5-2004 05:00:11 2010F - WI Gammon Ext.prf
Discharge 4.325 4.325 4.325 8.933 8.933 MGD
MH05-303
MH05-302
MH05-301
MH05-236
MH16-211
MH16-210
MH16-209
[feet]
83.0
82.0
81.0
80.0
79.0
78.0
77.0
76.0
75.0
74.0
73.0
72.0
71.0
70.0
2010F - West Interceptor/
Gammon Extension
0.0
200.0
400.0
600.0
800.0
1000.0
1200.0
1400.0
1:0
[feet]
Ground Lev. 83.00 82.00 83.00 82.00 79.00 [m]
Invert lev. 72.12 71.58 71.06 70.49 69.58 [m]
Length 328.00 320.00 298.00 282.00 141.00 [m]
Diameter 2.00 2.00 2.00 3.00 3.00 [m]
Slope o/oo 1.65 1.63 1.91 1.67 1.13

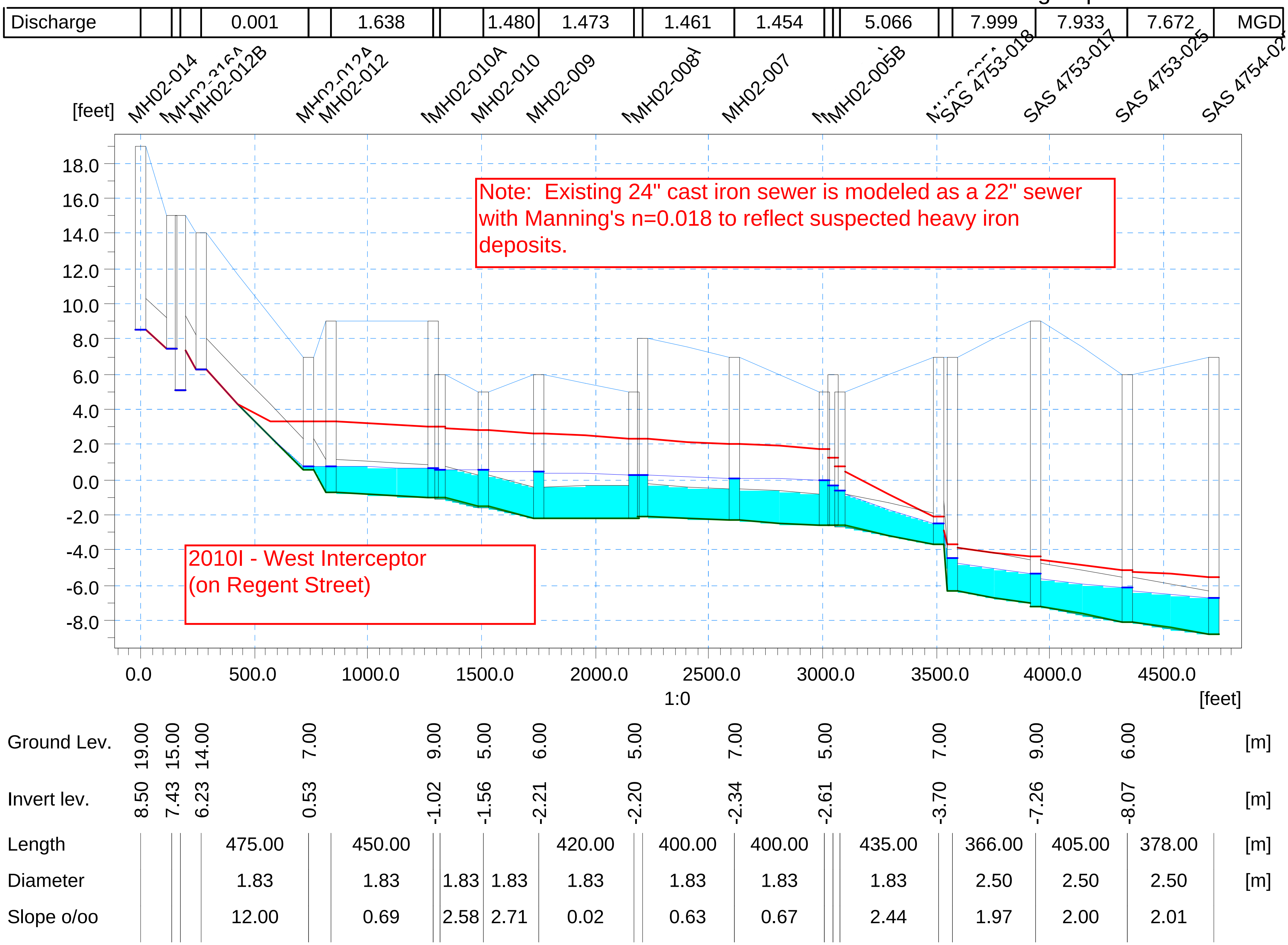


2010 UF CARPC FLOWS
10.2 - 10.4 MGD
13.1 MGD
WATER LEVEL BRANCHES - 21-5-2004 23:37:42 WI_2010UF_Flows.PRF
Discharge
5.224
5.156
10.313
10.288
8.951
8.933
8.915
8.897
8.880
8.856
12.932
12.884
5.847
8.561
8.499
MGD
BD05-21839
MH02-547
MH02-544A
MH02-544
MH02-543
MH02-541
MH02-540
MH02-539
MH02-538
MH02-537
MH02-536
MH02-535
MH02-534
MH02-531A
MH02-531
MH02-530
MH02-529
MH02-527
MH02-526
MH02-525
MH02-523
MH02-522
MH02-521
MH02-520
MH02-518A
MH02-515
MH02-514
MH02-513
MH02-512
MH02-511
MH02-509
[feet]
Shorewood Blvd.
2010G - West Interceptor Relief
(Existing Condtions)
1:0
Ground Lev.
Invert lev.
Length
Diameter
Slope o/oo
[m]

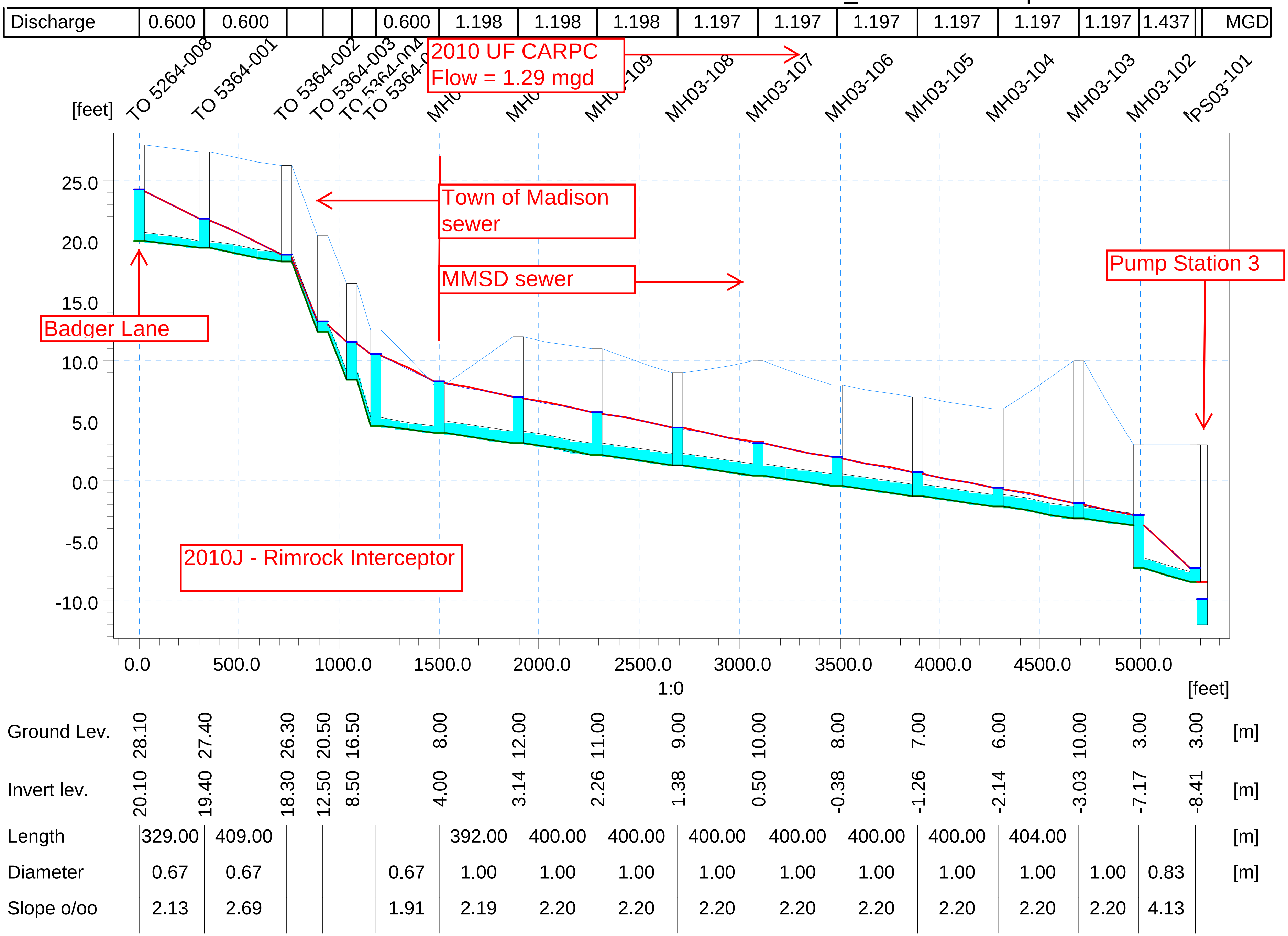


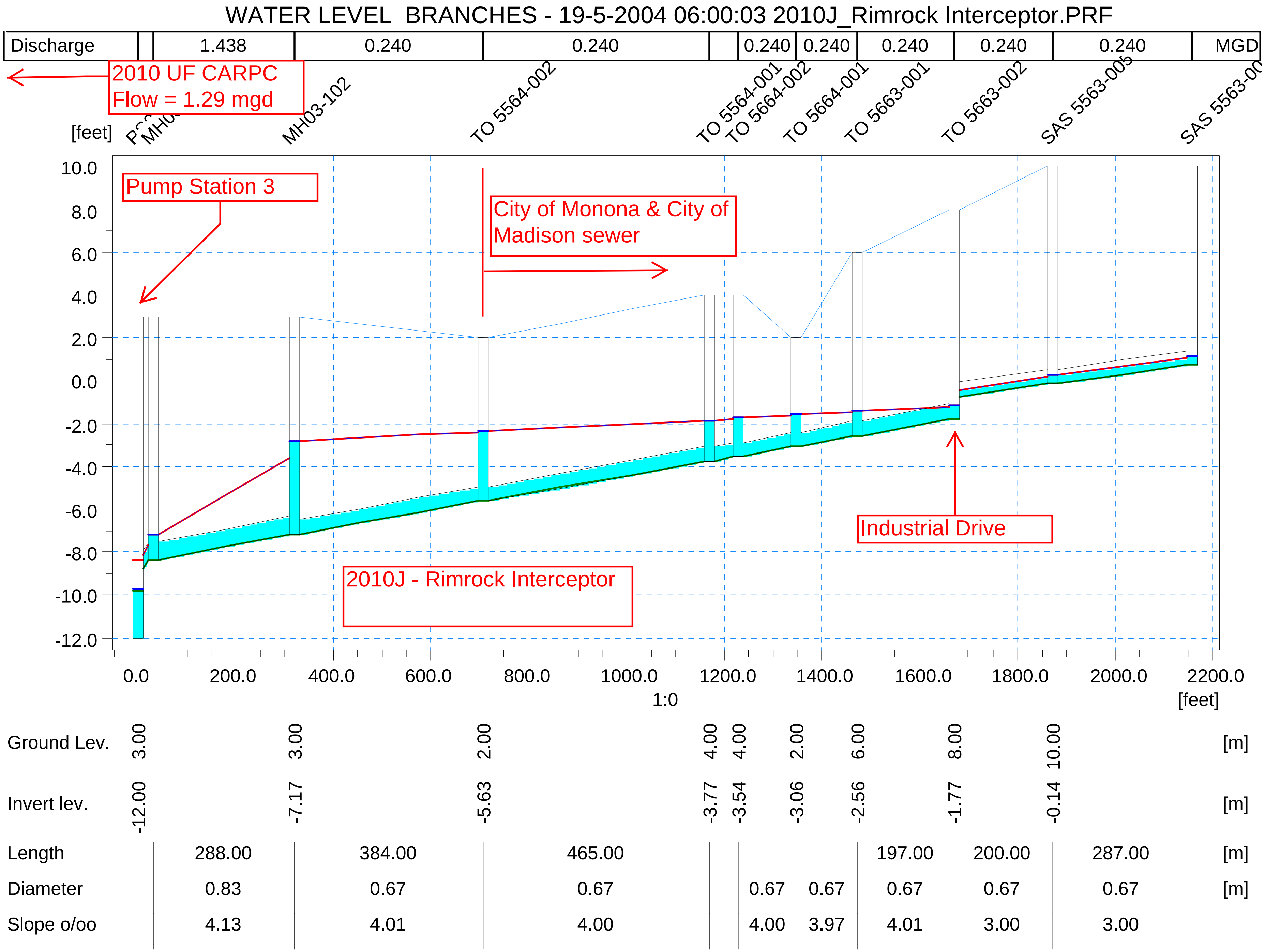
CARPC Flows - 2010 UF
6.06 mgd
7.76 mgd
WATER LEVEL BRANCHES - 21-5-2004 23:51:41 ExWI_May04_PB1.0+1mgd_012610.PRF
Discharge 7.077 7.349 8.688 18.351 27.809 MGD
SAS 4854-008
SAS 4854-007
MH02-300
MH02-101
MH02-402
MH02-401
dummy_PS02
[feet]
2010H - Spring Street Relief
(Existing Conditions)
1:0
Ground Lev. 4.00 8.00 6.00 7.00 6.00 6.00 [m]
Invert lev. -7.98 -8.49 -8.55 -8.55 -10.50 -10.70 [m]
Length 673.00 80.00 284.00 91.00 [m]
Diameter 3.00 3.00 2.00 3.00 4.00 4.00 [m]
Slope o/oo 0.76 0.75 0.00 0.00 0.70 0.55

WATER LEVEL BRANCHES - 21-5-2004 07:51:56 2010I - WI - 22 inch & RoughPipe.PRF

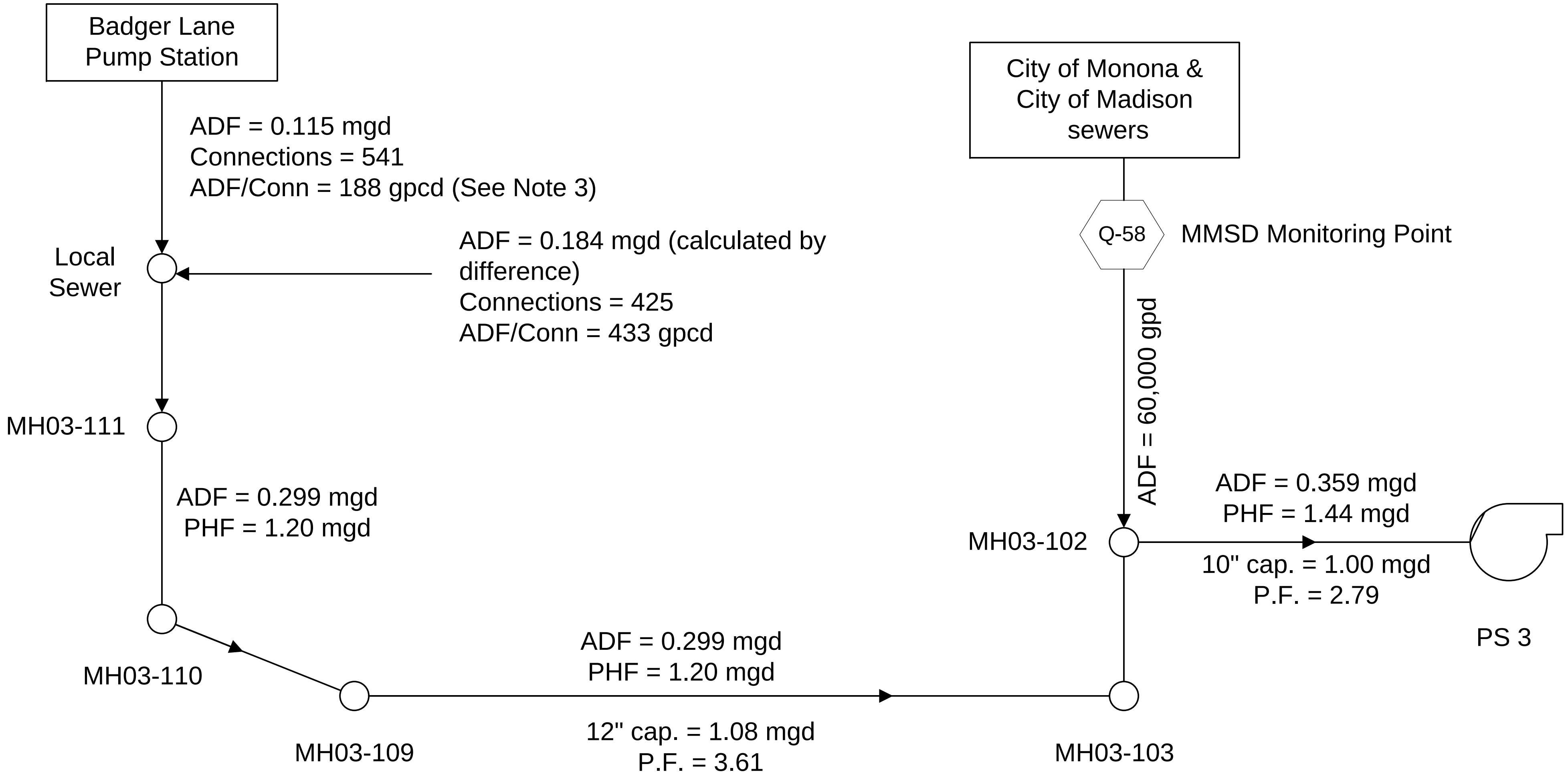


WATER LEVEL BRANCHES - 19-5-2004 05:00:29 2010J_Rimrock Interceptor.PRF





WATER LEVEL BRANCHES - 19-5-2004 06:00:03 2010J_Rimrock Interceptor.PRF
Discharge
1.438
0.240
0.240
0.240
0.240
0.240
0.240
0.240
MGD
2010 UF CARPC
Flow = 1.29 mgd
MH03-102
TO 5564-002
TO 5564-001
TO 5564-002
TO 5664-001
TO 5663-001
TO 5663-002
SAS 5563-005
[feet]
Pump Station 3
City of Monona & City of
Madison sewer
Industrial Drive
2010J - Rimrock Interceptor
10.0
8.0
6.0
4.0
2.0
0.0
-2.0
-4.0
-6.0
-8.0
-10.0
-12.0
0.0
200.0
400.0
600.0
800.0
1000.0
1200.0
1400.0
1600.0
1800.0
2000.0
2200.0
1:0
[feet]
Ground Lev. 3.00 3.00 2.00 4.00 4.00 2.00 6.00 8.00 10.00 [m]
Invert lev. -12.00 -7.17 -5.63 -3.77 -3.54 -3.06 -2.56 -1.77 -0.14 [m]
Length 288.00 384.00 465.00 197.00 200.00 287.00 [m]
Diameter 0.83 0.67 0.67 0.67 0.67 0.67 0.67 0.67 [m]
Slope o/oo 4.13 4.01 4.00 4.00 3.97 4.01 3.00 3.00



NOTES:

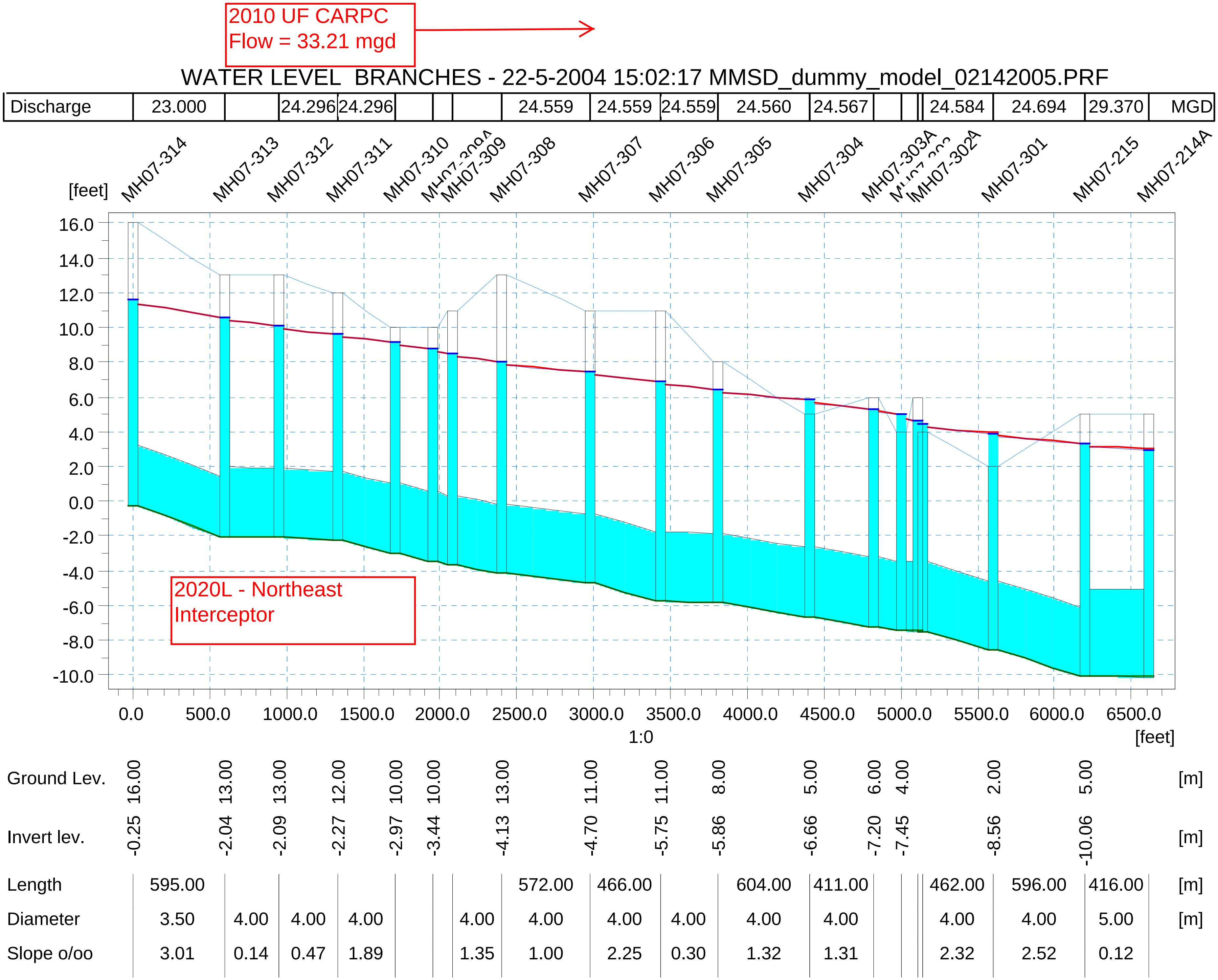
(1). All average daily flows (ADF) shown are for Year 2009. PS 3 flows are metered via magmeter. Badger Lane Pump Station flows are estimated from run time hours. Flows at Q-58 are monitored by MMSD personnel on a quarterly basis for one week durations.

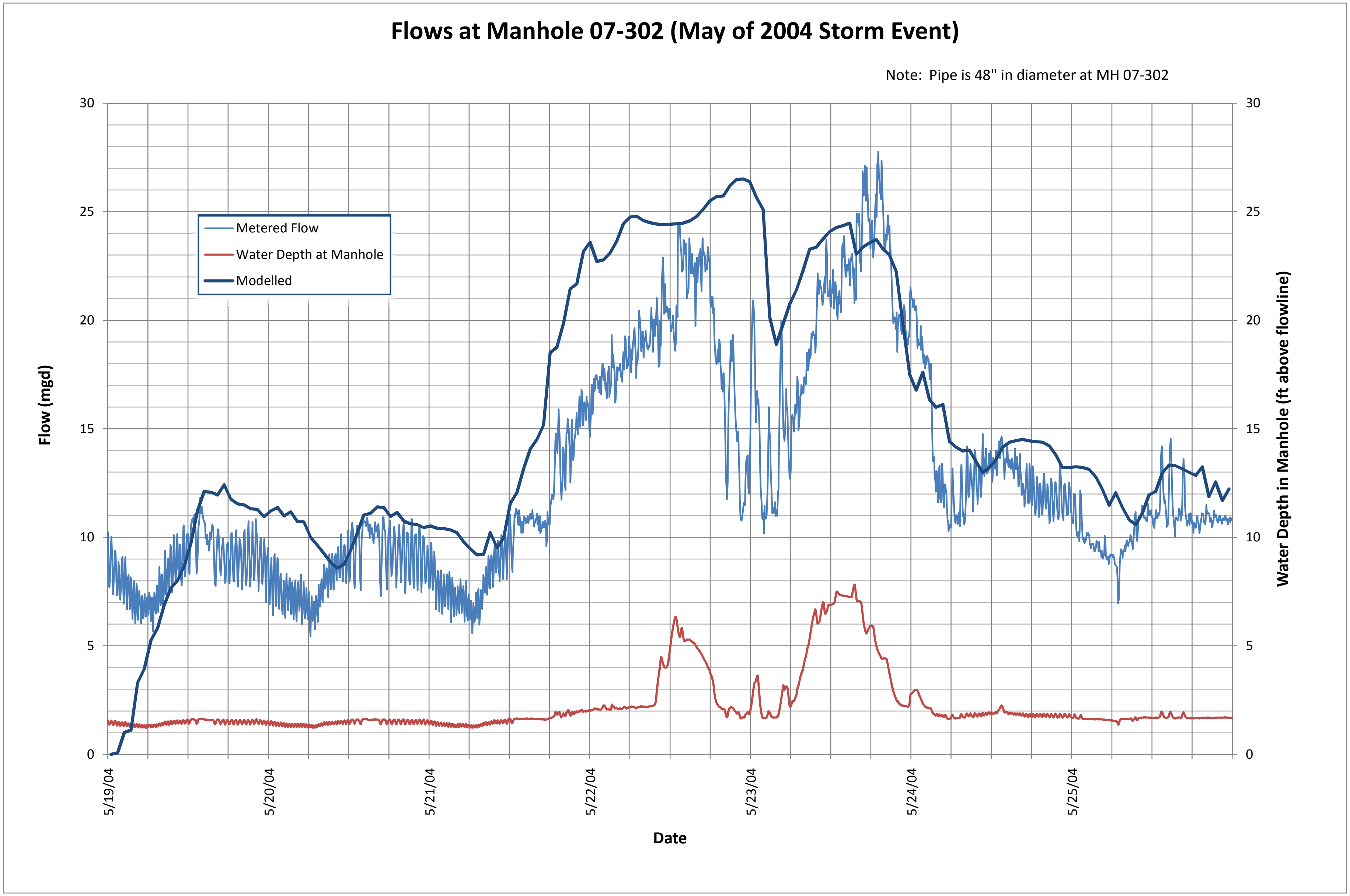
(2). Connection counts obtained from MMSD's User Charge billing system (last revised 2000).

(3). Large flow generators, Clarion Hotel (6,700 gpd) and Department of Revenue Building (6,900 gpd), not included in "flow/connection" calculation.

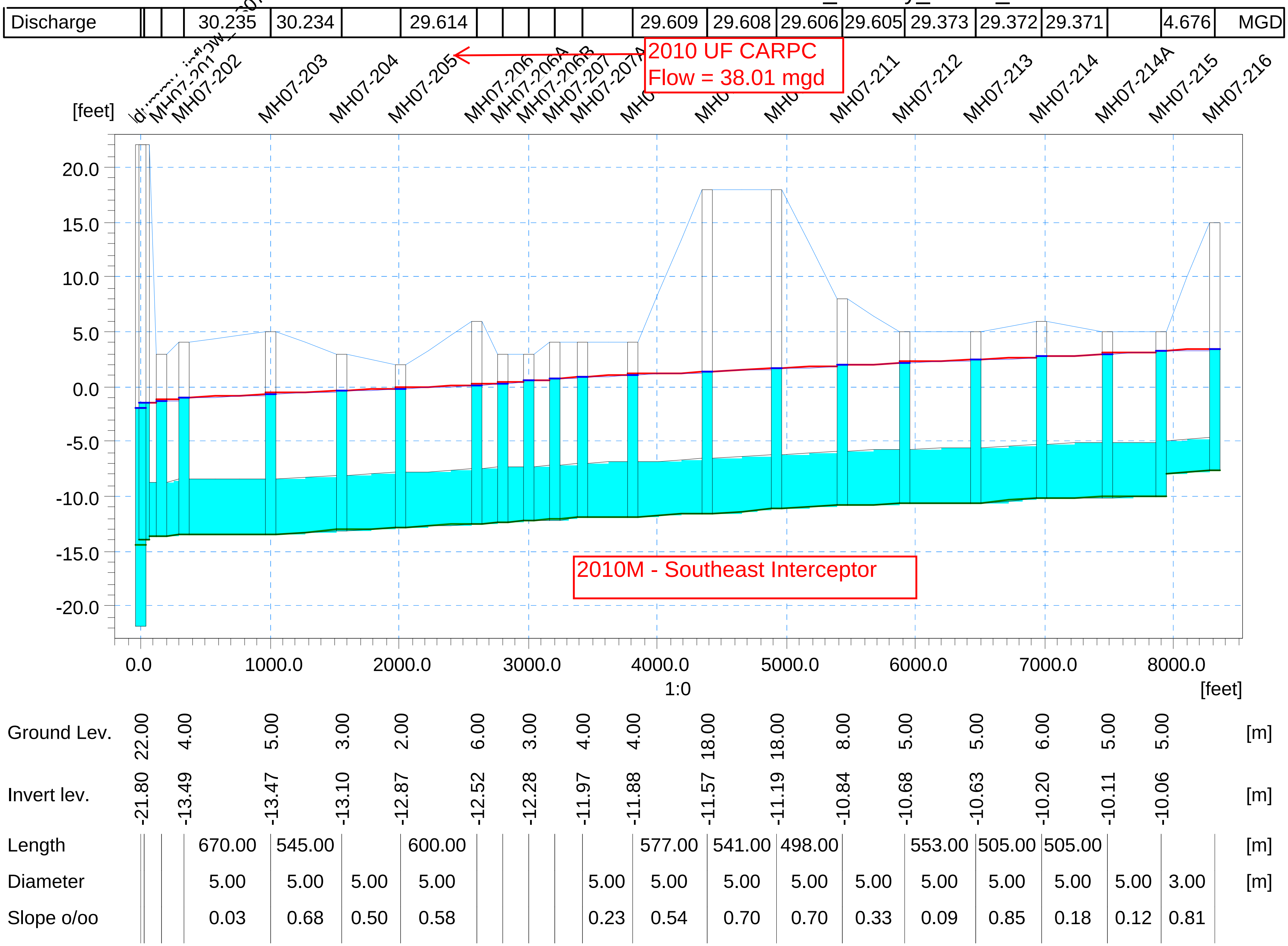
(4). CARPC's 2010 UF flow estimate for average daily flow at PS 3 is 0.322 mgd.

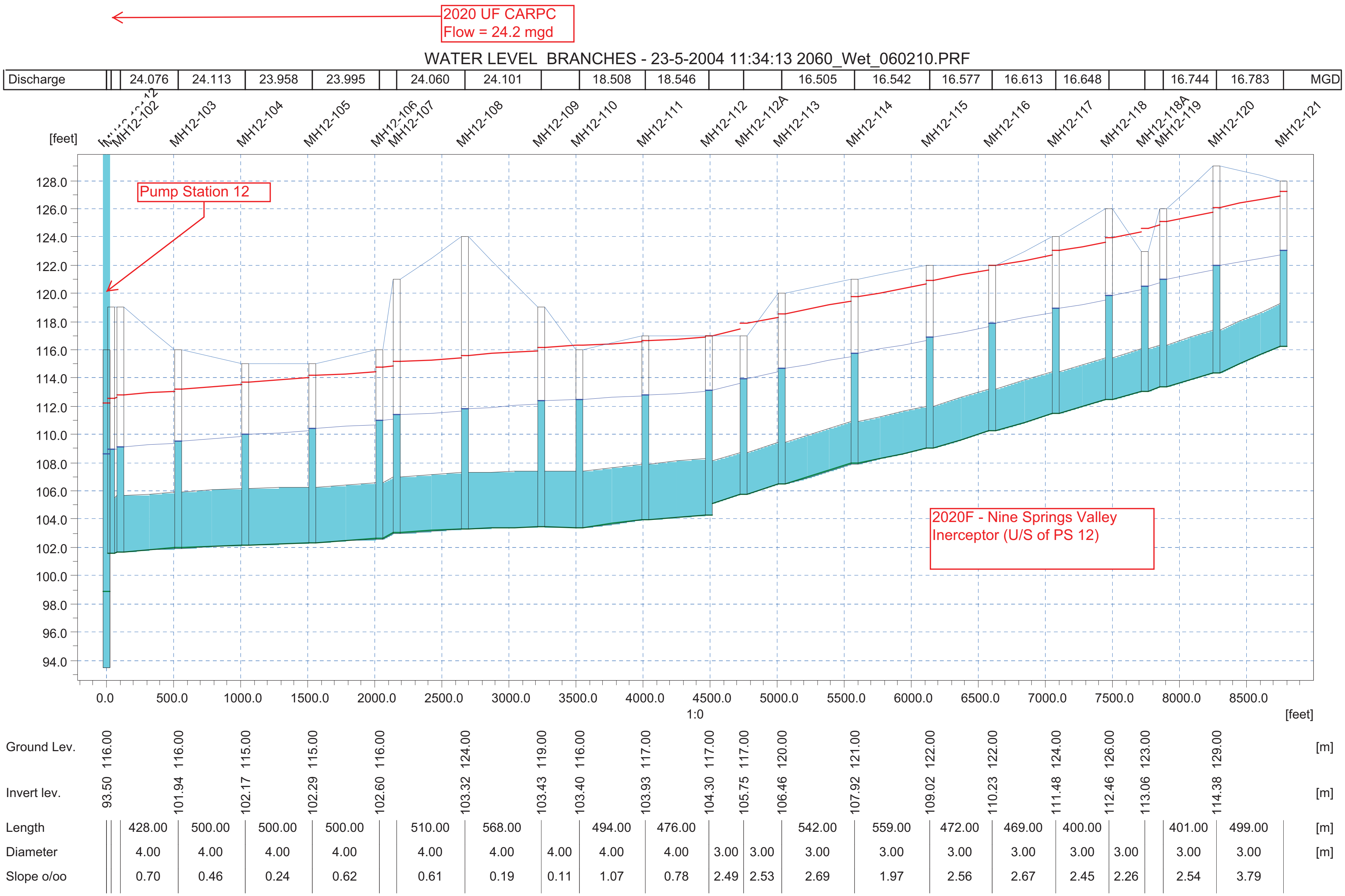
2010J – RIMROCK INTERCEPTOR – FLOW SCHEMATIC

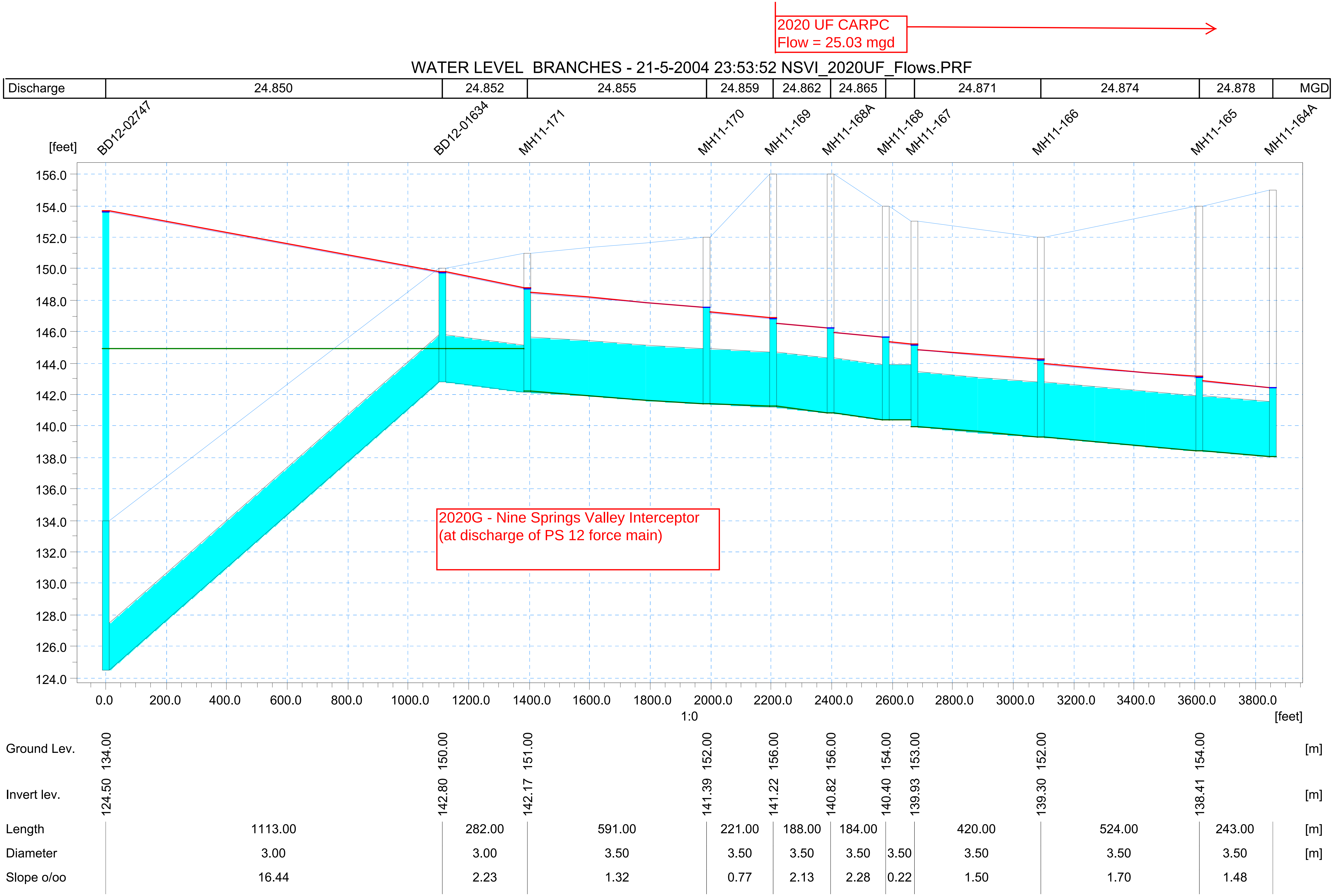

2010 UF CARPC
Flow = 33.21 mgd
WATER LEVEL BRANCHES - 22-5-2004 15:02:17 MMSD_dummy_model_02142005.PRF
Discharge 23.000 24.296 24.296 24.559 24.559 24.559 24.560 24.567 24.584 24.694 29.370 MGD
MH07-314
MH07-313
MH07-312
MH07-311
MH07-310
MH07-309
MH07-308
MH07-307
MH07-306
MH07-305
MH07-304
MH07-303A
MH07-302A
MH07-301
MH07-215
MH07-214A
[feet]
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
-2.0
-4.0
-6.0
-8.0
-10.0
2020L - Northeast Interceptor
0.0 500.0 1000.0 1500.0 2000.0 2500.0 3000.0 3500.0 4000.0 4500.0 5000.0 5500.0 6000.0 6500.0
1:0
[feet]
Ground Lev. 16.00 13.00 13.00 12.00 10.00 10.00 13.00 11.00 11.00 8.00 5.00 6.00 4.00 2.00 5.00 [m]
Invert lev. -0.25 -2.04 -2.09 -2.27 -2.97 -3.44 -4.13 -4.70 -5.75 -5.86 -6.66 -7.20 -7.45 -8.56 -10.06 [m]
Length 595.00 572.00 466.00 604.00 411.00 462.00 596.00 416.00 [m]
Diameter 3.50 4.00 4.00 4.00 4.00 4.00 4.00 4.00 4.00 4.00 4.00 4.00 5.00 [m]
Slope o/oo 3.01 0.14 0.47 1.89 1.35 1.00 2.25 0.30 1.32 1.31 2.32 2.52 0.12

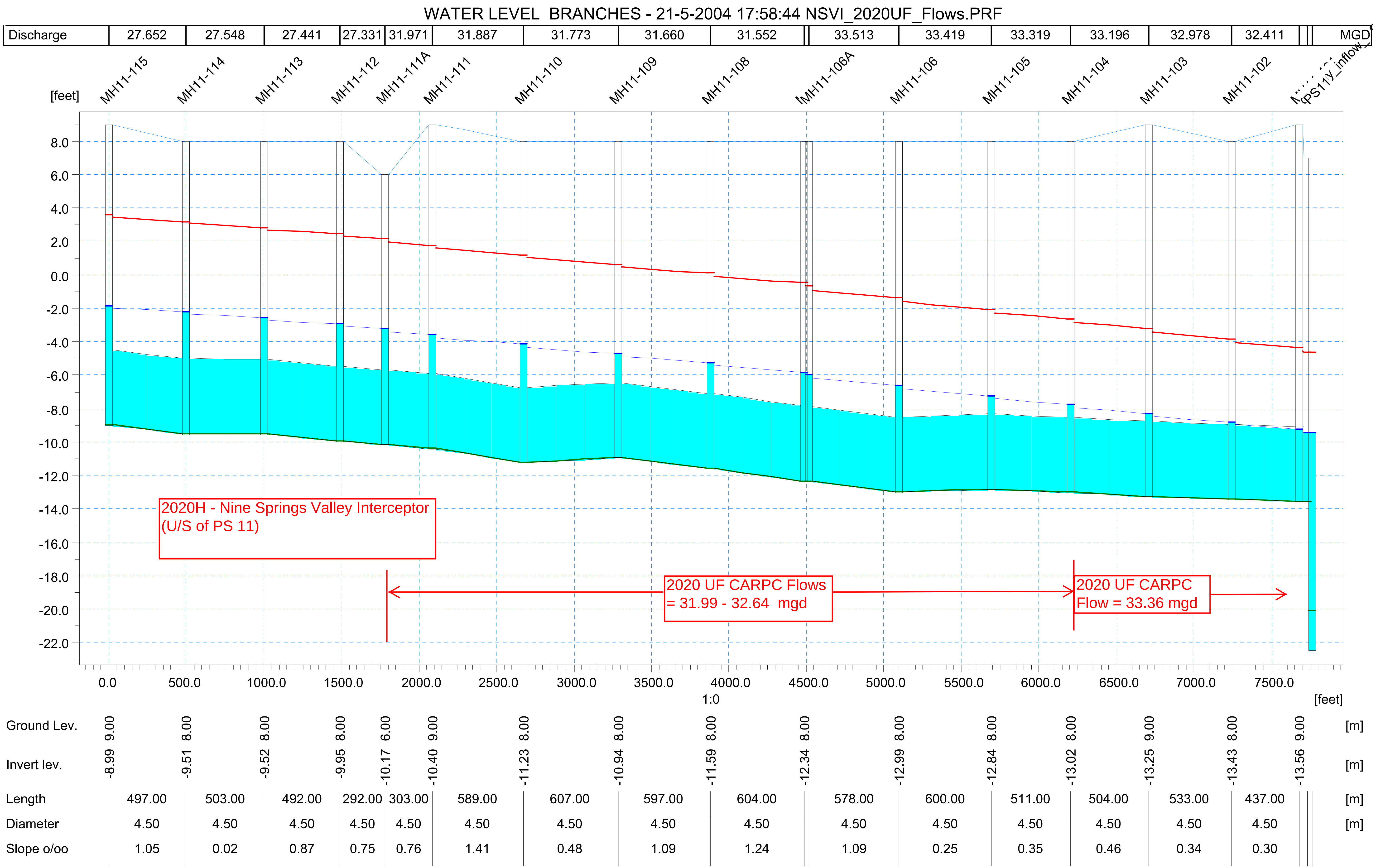

Flows at Manhole 07-302 (May of 2004 Storm Event)
Note: Pipe is 48" in diameter at MH 07-302
Metered Flow
Water Depth at Manhole
Modelled
Flow (mgd)
Water Depth in Manhole (ft above flowline)
Date
5/19/04
5/20/04
5/21/04
5/22/04
5/23/04
5/24/04
5/25/04
0
5
10
15
20
25
30

WATER LEVEL BRANCHES - 22-5-2004 15:02:17 MMSD_dummy_model_02142005.PRF

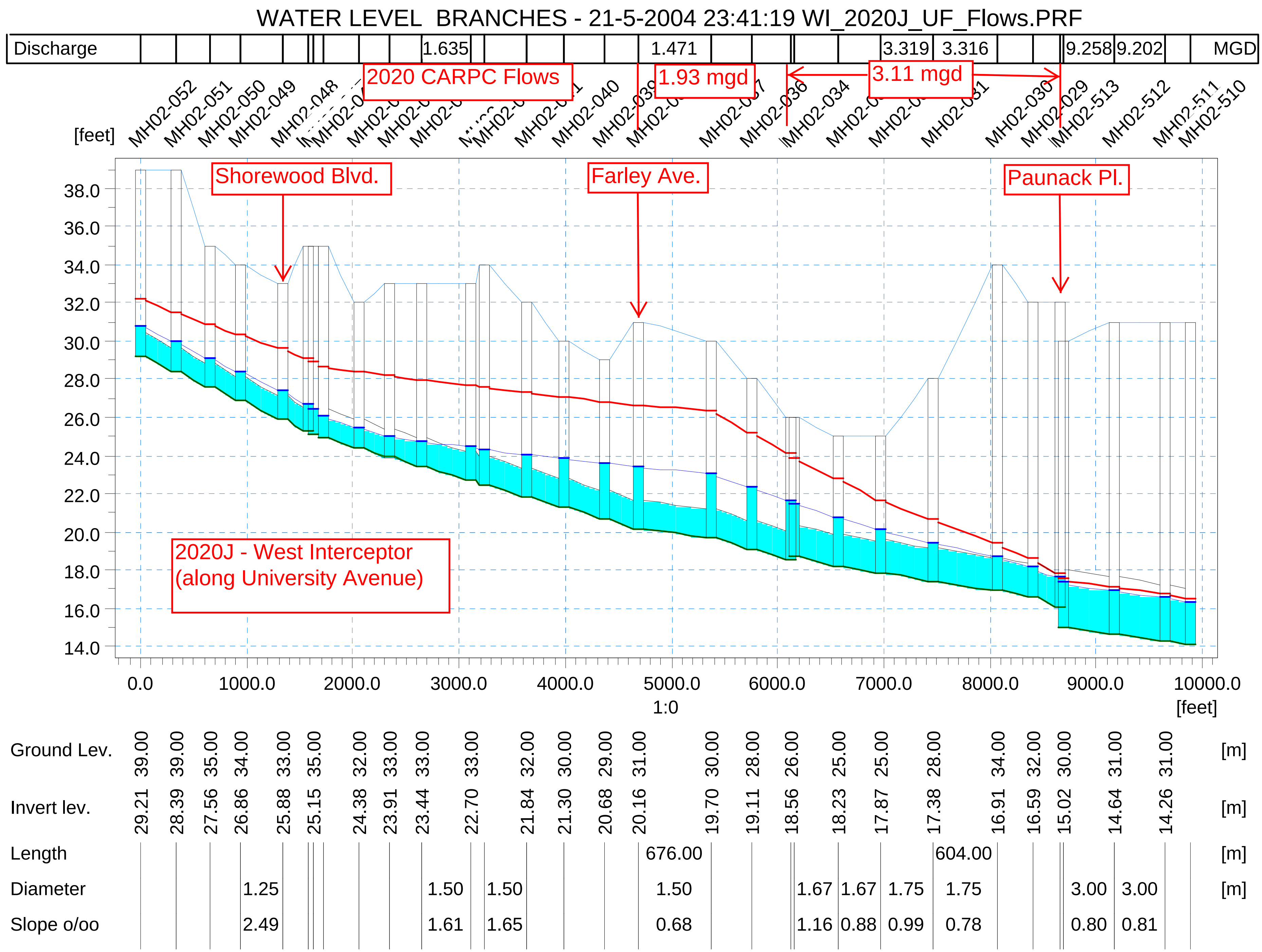



2020 UF CARPC
Flow = 24.2 mgd
WATER LEVEL BRANCHES - 23-5-2004 11:34:13 2060_Wet_060210.PRF
Discharge 24.076 24.113 23.958 23.995 24.060 24.101 18.508 18.546 16.505 16.542 16.577 16.613 16.648 16.744 16.783 MGD
MH12-102 MH12-103 MH12-104 MH12-105 MH12-106 MH12-107 MH12-108 MH12-109 MH12-110 MH12-111 MH12-112 MH12-112A MH12-113 MH12-114 MH12-115 MH12-116 MH12-117 MH12-118 MH12-118A MH12-119 MH12-120 MH12-121
[feet]
128.0 126.0 124.0 122.0 120.0 118.0 116.0 114.0 112.0 110.0 108.0 106.0 104.0 102.0 100.0 98.0 96.0 94.0
Pump Station 12
2020F - Nine Springs Valley Inerceptor (U/S of PS 12)
0.0 500.0 1000.0 1500.0 2000.0 2500.0 3000.0 3500.0 4000.0 4500.0 5000.0 5500.0 6000.0 6500.0 7000.0 7500.0 8000.0 8500.0
1:0
[feet]
Ground Lev. 116.00 116.00 115.00 115.00 116.00 124.00 119.00 116.00 117.00 117.00 117.00 120.00 121.00 122.00 122.00 124.00 126.00 123.00 129.00 [m]
Invert lev. 93.50 101.94 102.17 102.29 102.60 103.32 103.43 103.40 103.93 104.30 105.75 106.46 107.92 109.02 110.23 111.48 112.46 113.06 114.38 [m]
Length 428.00 500.00 500.00 500.00 510.00 568.00 494.00 476.00 542.00 559.00 472.00 469.00 400.00 401.00 499.00 [m]
Diameter 4.00 4.00 4.00 4.00 4.00 4.00 4.00 4.00 4.00 3.00 3.00 3.00 3.00 3.00 3.00 3.00 3.00 3.00 3.00 [m]
Slope o/oo 0.70 0.46 0.24 0.62 0.61 0.19 0.11 1.07 0.78 2.49 2.53 2.69 1.97 2.56 2.67 2.45 2.26 2.54 3.79

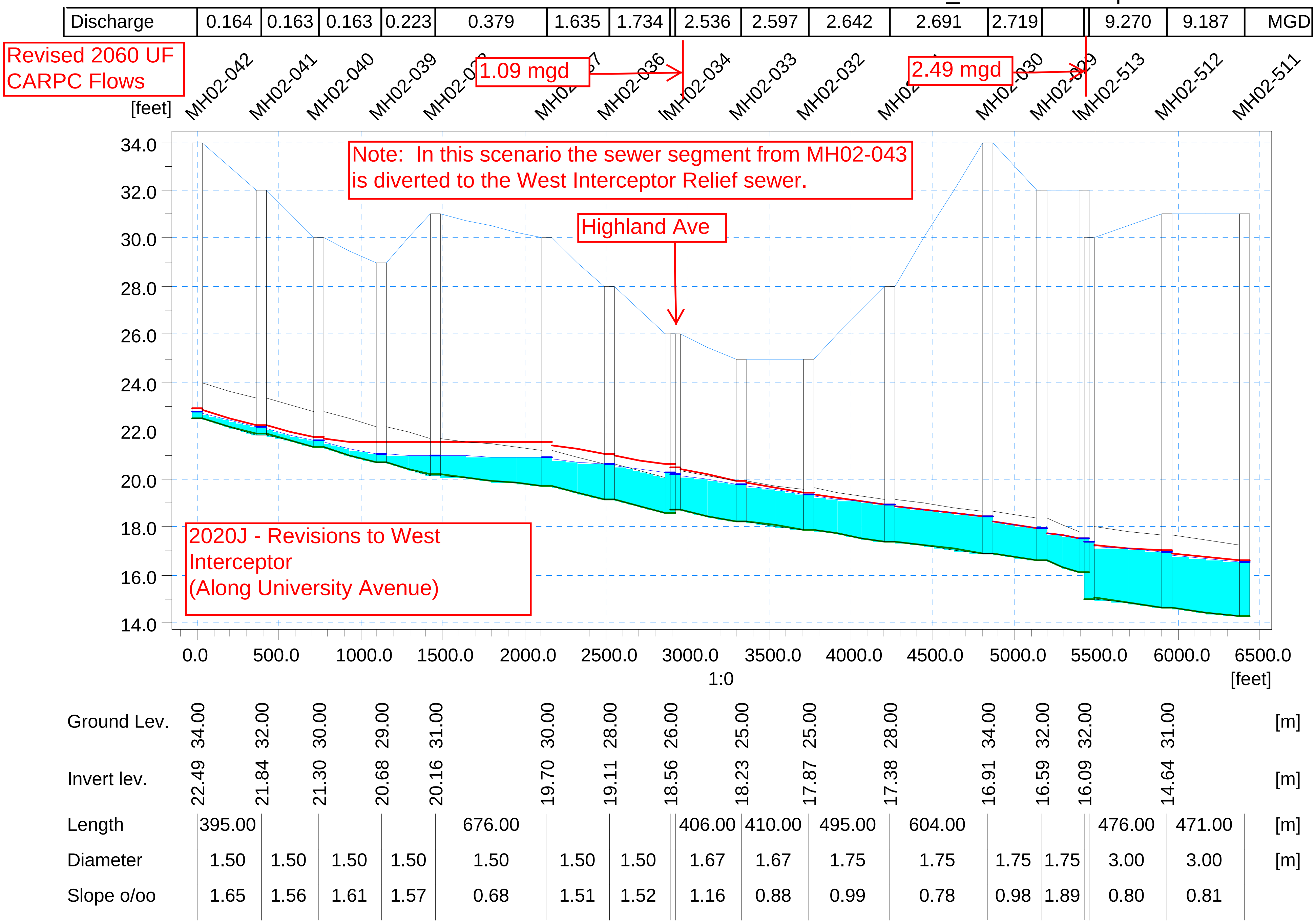

2020 UF CARPC
Flow = 25.03 mgd
WATER LEVEL BRANCHES - 21-5-2004 23:53:52 NSVI_2020UF_Flows.PRF
Discharge
24.850
24.852
24.855
24.859
24.862
24.865
24.871
24.874
24.878
MGD
BD12-02747
BD12-01634
MH11-171
MH11-170
MH11-169
MH11-168A
MH11-168
MH11-167
MH11-166
MH11-165
MH11-164A
[feet]
2020G - Nine Springs Valley Interceptor
(at discharge of PS 12 force main)
1:0
Ground Lev.
134.00
150.00
151.00
152.00
156.00
156.00
154.00
153.00
152.00
154.00
Invert lev.
124.50
142.80
142.17
141.39
141.22
140.82
140.40
139.93
139.30
138.41
Length
1113.00
282.00
591.00
221.00
188.00
184.00
420.00
524.00
243.00
Diameter
3.00
3.00
3.50
3.50
3.50
3.50
3.50
3.50
3.50
3.50
Slope o/oo
16.44
2.23
1.32
0.77
2.13
2.28
0.22
1.50
1.70
1.48
[m]

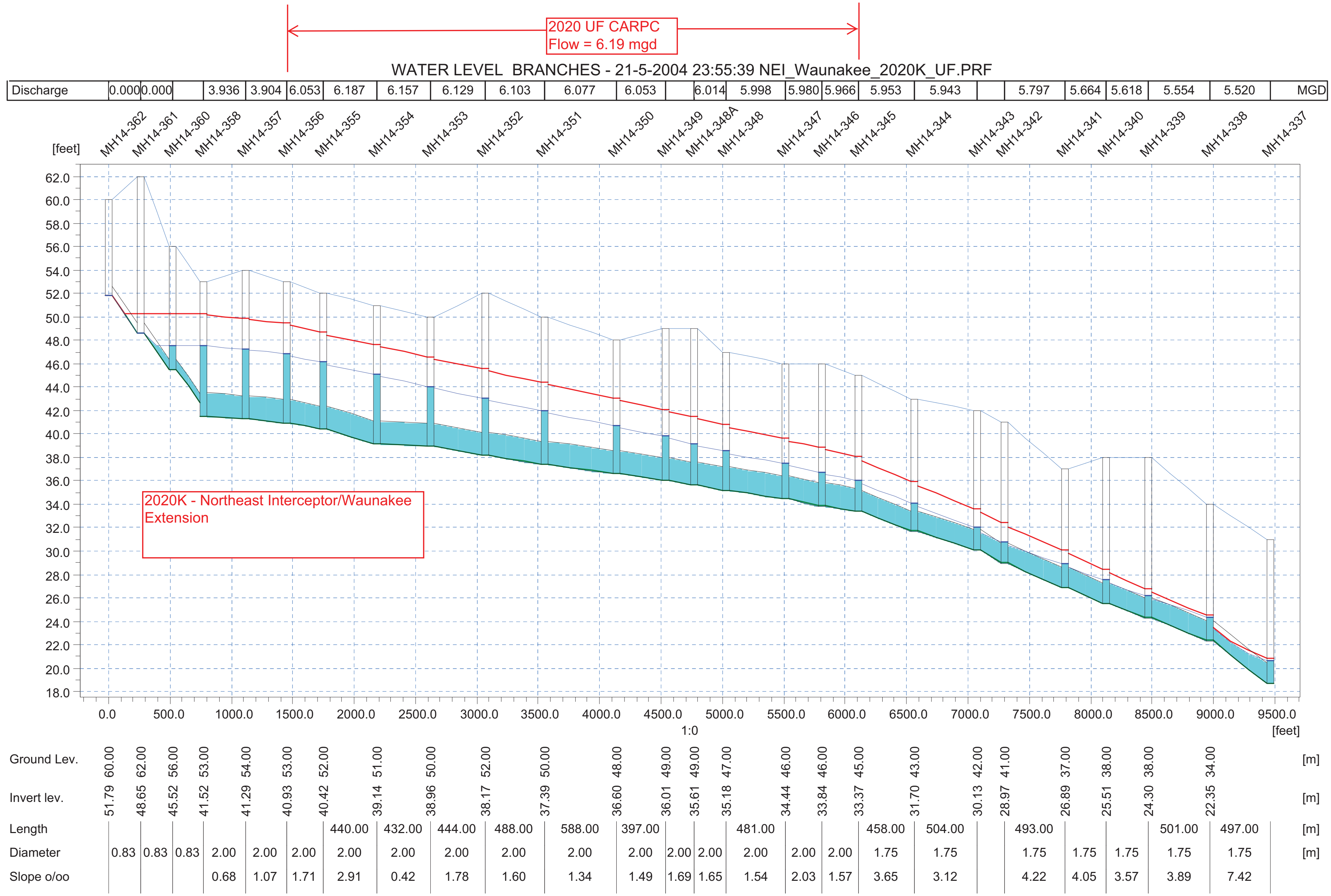


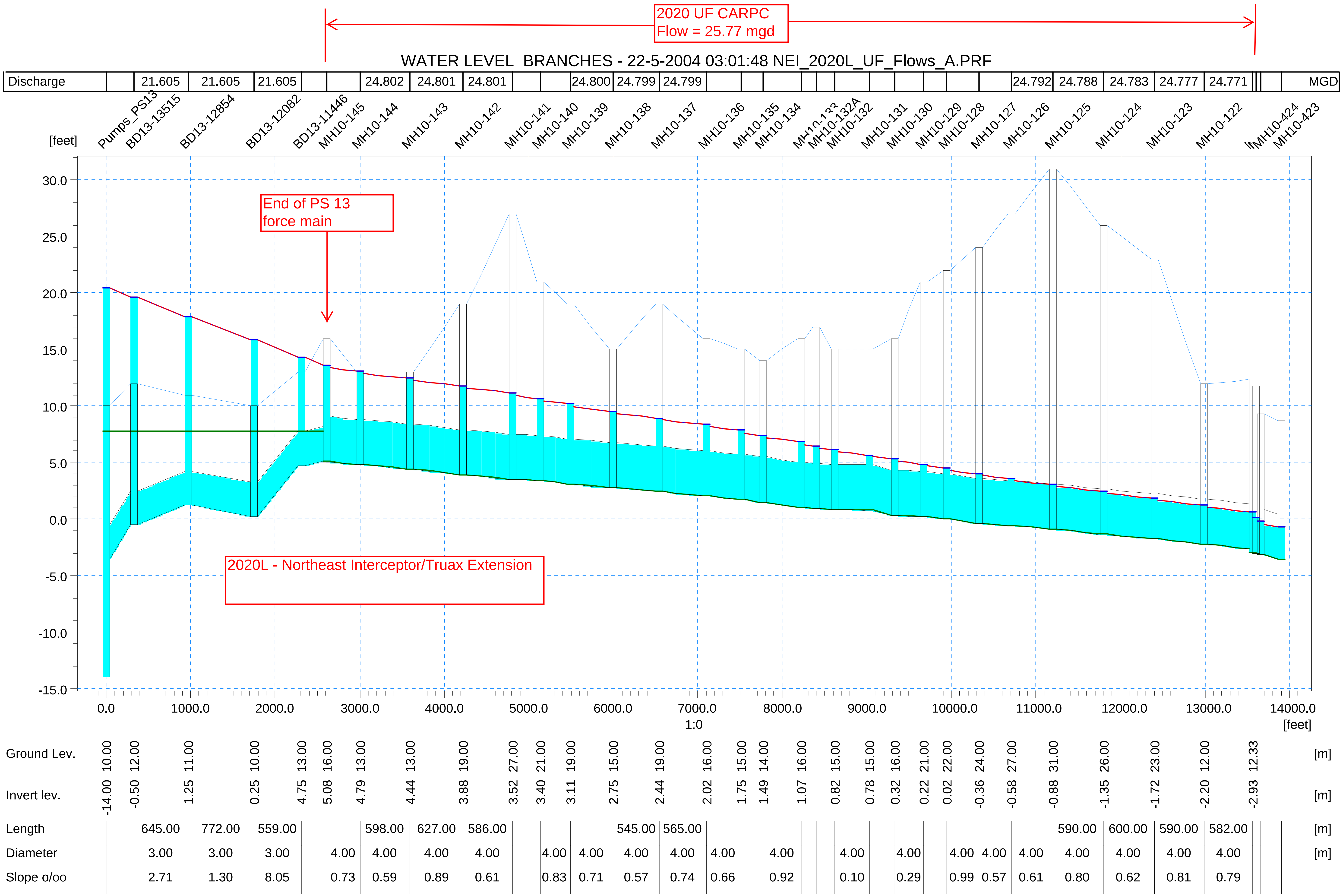
WATER LEVEL BRANCHES - 21-5-2004 17:58:44 NSVI_2020UF_Flows.PRF
Discharge 27.652 27.548 27.441 27.331 31.971 31.887 31.773 31.660 31.552 33.513 33.419 33.319 33.196 32.978 32.411 MGD
MH11-115 MH11-114 MH11-113 MH11-112 MH11-111A MH11-111 MH11-110 MH11-109 MH11-108 MH11-106A MH11-106 MH11-105 MH11-104 MH11-103 MH11-102
[feet]
2020H - Nine Springs Valley Interceptor
(U/S of PS 11)
2020 UF CARPC Flows
= 31.99 - 32.64 mgd
2020 UF CARPC
Flow = 33.36 mgd
1:0
[feet]
Ground Lev. 9.00 8.00 8.00 8.00 6.00 9.00 8.00 8.00 8.00 8.00 8.00 8.00 8.00 9.00 8.00 9.00 [m]
Invert lev. -8.99 -9.51 -9.52 -9.95 -10.17 -10.40 -11.23 -10.94 -11.59 -12.34 -12.99 -12.84 -13.02 -13.25 -13.43 -13.56 [m]
Length 497.00 503.00 492.00 292.00 303.00 589.00 607.00 597.00 604.00 578.00 600.00 511.00 504.00 533.00 437.00 [m]
Diameter 4.50 4.50 4.50 4.50 4.50 4.50 4.50 4.50 4.50 4.50 4.50 4.50 4.50 4.50 4.50 [m]
Slope o/oo 1.05 0.02 0.87 0.75 0.76 1.41 0.48 1.09 1.24 1.09 0.25 0.35 0.46 0.34 0.30


WATER LEVEL BRANCHES - 21-5-2004 23:41:19 WI_2020J_UF_Flows.PRF
Discharge
1.635
1.471
3.319
3.316
9.258
9.202
MGD
2020 CARPC Flows
1.93 mgd
3.11 mgd
MH02-052
MH02-051
MH02-050
MH02-049
MH02-048
MH02-040
MH02-036
MH02-034
MH02-030
MH02-029
MH02-513
MH02-512
MH02-511
MH02-510
[feet]
Shorewood Blvd.
Farley Ave.
Paunack Pl.
2020J - West Interceptor
(along University Avenue)
38.0
36.0
34.0
32.0
30.0
28.0
26.0
24.0
22.0
20.0
18.0
16.0
14.0
0.0
1000.0
2000.0
3000.0
4000.0
5000.0
6000.0
7000.0
8000.0
9000.0
10000.0
1:0
[feet]
Ground Lev.
39.00 39.00 35.00 34.00 33.00 35.00 32.00 33.00 33.00 33.00 32.00 30.00 29.00 31.00 30.00 28.00 26.00 25.00 25.00 28.00 34.00 32.00 30.00 31.00 31.00
[m]
Invert lev.
29.21 28.39 27.56 26.86 25.88 25.15 24.38 23.91 23.44 22.70 21.84 21.30 20.68 20.16 19.70 19.11 18.56 18.23 17.87 17.38 16.91 16.59 15.02 14.64 14.26
[m]
Length
676.00
604.00
[m]
Diameter
1.25 1.50 1.50 1.50 1.67 1.67 1.75 1.75 3.00 3.00
[m]
Slope o/oo
2.49 1.61 1.65 0.68 1.16 0.88 0.99 0.78 0.80 0.81

WATER LEVEL BRANCHES - 21-5-2004 23:41:17 OWI Rev_No WIR Relief.prf

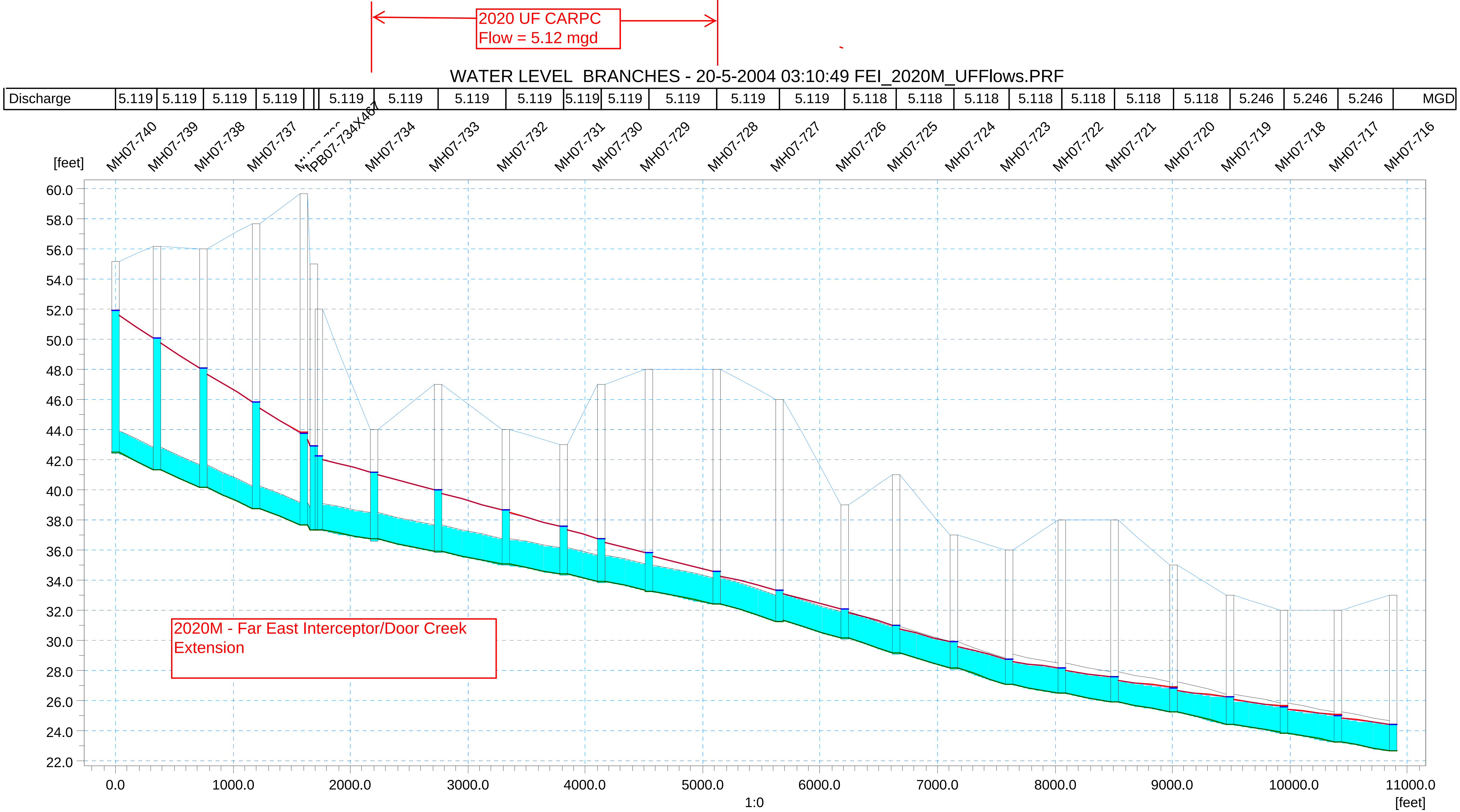



2020 UF CARPC
Flow = 6.19 mgd
WATER LEVEL BRANCHES - 21-5-2004 23:55:39 NEI_Waunakee_2020K_UF.PRF
Discharge 0.000 0.000 3.936 3.904 6.053 6.187 6.157 6.129 6.103 6.077 6.053 6.014 5.998 5.980 5.966 5.953 5.943 5.797 5.664 5.618 5.554 5.520 MGD
MH14-362 MH14-361 MH14-360 MH14-358 MH14-357 MH14-356 MH14-355 MH14-354 MH14-353 MH14-352 MH14-351 MH14-350 MH14-349 MH14-348A MH14-348 MH14-347 MH14-346 MH14-345 MH14-344 MH14-343 MH14-342 MH14-341 MH14-340 MH14-339 MH14-338 MH14-337
[feet]
2020K - Northeast Interceptor/Waunakee Extension
1:0
[feet]
Ground Lev. 60.00 62.00 56.00 53.00 54.00 53.00 52.00 51.00 50.00 52.00 50.00 48.00 49.00 49.00 47.00 46.00 46.00 45.00 43.00 42.00 41.00 37.00 38.00 38.00 34.00 [m]
Invert lev. 51.79 48.65 45.52 41.52 41.29 40.93 40.42 39.14 38.96 38.17 37.39 36.60 36.01 35.61 35.18 34.44 33.84 33.37 31.70 30.13 28.97 26.89 25.51 24.30 22.35 [m]
Length 440.00 432.00 444.00 488.00 588.00 397.00 481.00 458.00 504.00 493.00 501.00 497.00 [m]
Diameter 0.83 0.83 0.83 2.00 2.00 2.00 2.00 2.00 2.00 2.00 2.00 2.00 2.00 2.00 2.00 2.00 2.00 1.75 1.75 1.75 1.75 1.75 1.75 1.75 [m]
Slope o/oo 0.68 1.07 1.71 2.91 0.42 1.78 1.60 1.34 1.49 1.69 1.65 1.54 2.03 1.57 3.65 3.12 4.22 4.05 3.57 3.89 7.42


2020 UF CARPC
Flow = 25.77 mgd
WATER LEVEL BRANCHES - 22-5-2004 03:01:48 NEI_2020L_UF_Flows_A.PRF
Discharge 21.605 21.605 21.605 24.802 24.801 24.801 24.800 24.799 24.799 24.792 24.788 24.783 24.777 24.771 MGD
Pumps_PS13 BD13-13515 BD13-12854 BD13-12082 BD13-11446 MH10-145 MH10-144 MH10-143 MH10-142 MH10-141 MH10-140 MH10-139 MH10-138 MH10-137 MH10-136 MH10-135 MH10-134 MH10-132A MH10-132 MH10-131 MH10-130 MH10-129 MH10-128 MH10-127 MH10-126 MH10-125 MH10-124 MH10-123 MH10-122 MH10-424 MH10-423
[feet]
30.0
25.0
20.0
15.0
10.0
5.0
0.0
-5.0
-10.0
-15.0
End of PS 13
force main
2020L - Northeast Interceptor/Truax Extension
0.0 1000.0 2000.0 3000.0 4000.0 5000.0 6000.0 7000.0 8000.0 9000.0 10000.0 11000.0 12000.0 13000.0 14000.0
1:0
[feet]
Ground Lev. 10.00 10.00 12.00 11.00 10.00 13.00 16.00 13.00 13.00 19.00 27.00 21.00 19.00 15.00 19.00 16.00 15.00 14.00 16.00 15.00 15.00 16.00 21.00 22.00 24.00 27.00 31.00 26.00 23.00 12.00 12.33 [m]
Invert lev. -14.00 -0.50 1.25 0.25 4.75 5.08 4.79 4.44 3.88 3.52 3.40 3.11 2.75 2.44 2.02 1.75 1.49 1.07 0.82 0.78 0.32 0.22 0.02 -0.36 -0.58 -0.88 -1.35 -1.72 -2.20 -2.93 [m]
Length 645.00 772.00 559.00 598.00 627.00 586.00 545.00 565.00 590.00 600.00 590.00 582.00 [m]
Diameter 3.00 3.00 3.00 4.00 4.00 4.00 4.00 4.00 4.00 4.00 4.00 4.00 4.00 4.00 4.00 4.00 4.00 4.00 4.00 4.00 4.00 4.00 [m]
Slope o/oo 2.71 1.30 8.05 0.73 0.59 0.89 0.61 0.83 0.71 0.57 0.74 0.66 0.92 0.10 0.29 0.99 0.57 0.61 0.80 0.62 0.81 0.79

WATER LEVEL BRANCHES - 20-5-2004 03:10:49 FEI_2020M_UFFlows.PRF

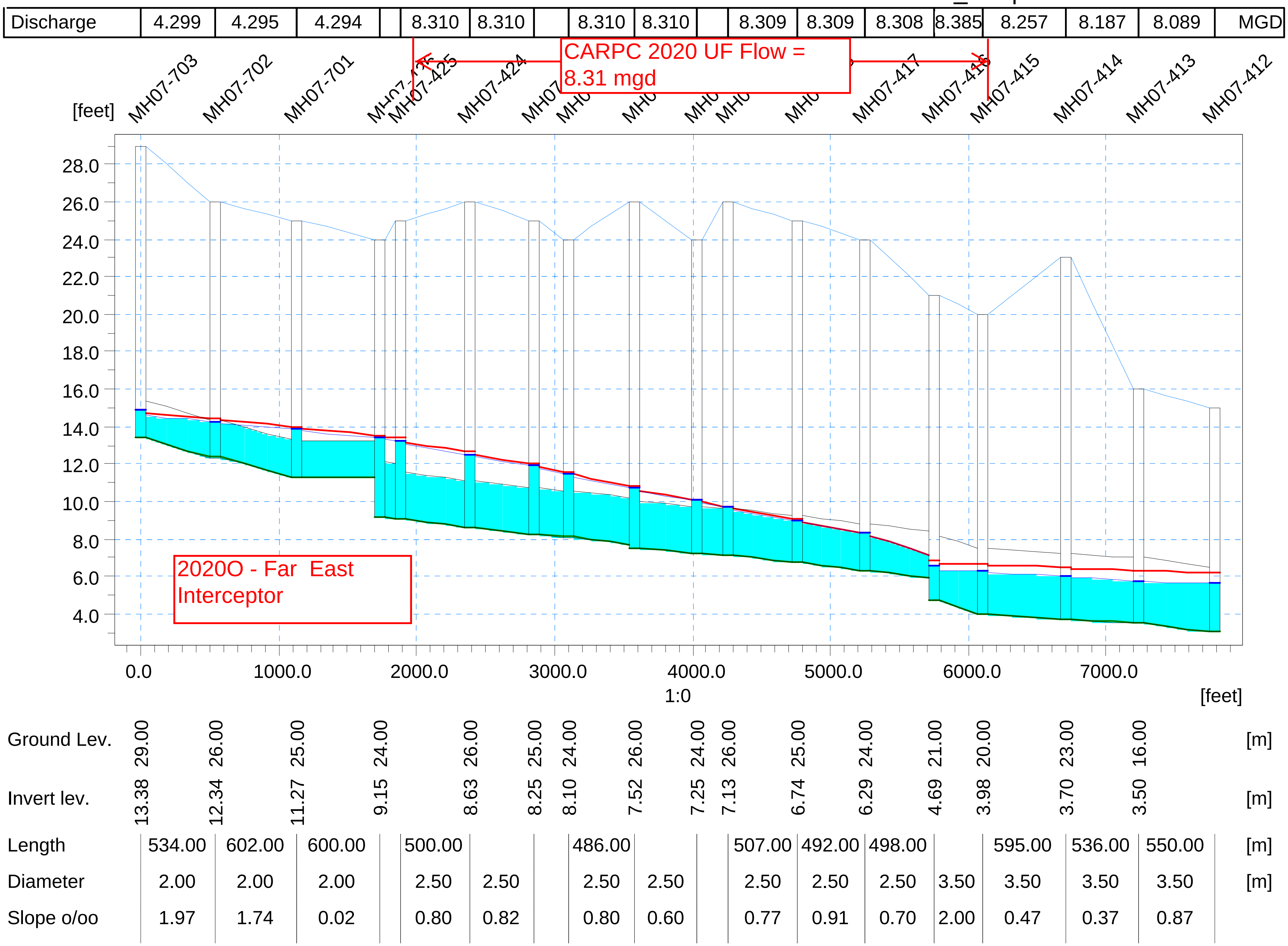
Discharge	5.119	5.119	5.119	5.119			5.119	5.119	5.119	5.119	5.119	5.119	5.119	5.119	5.119	5.118	5.118	5.118	5.118	5.118	5.118	5.118	5.246	5.246	5.246	MGD



Ground Lev.	55.20	56.20	56.00	57.70	59.70	44.00	47.00	44.00	43.00	47.00	48.00	48.00	46.00	39.00	41.00	37.00	36.00	38.00	38.00	35.00	33.00	32.00	32.00	[m]
Invert lev.	42.47	41.35	40.18	38.80	37.72	36.60	35.83	35.03	34.35	33.87	33.30	32.45	31.31	30.12	29.15	28.12	27.09	26.50	25.92	25.26	24.49	23.88	23.29	[m]

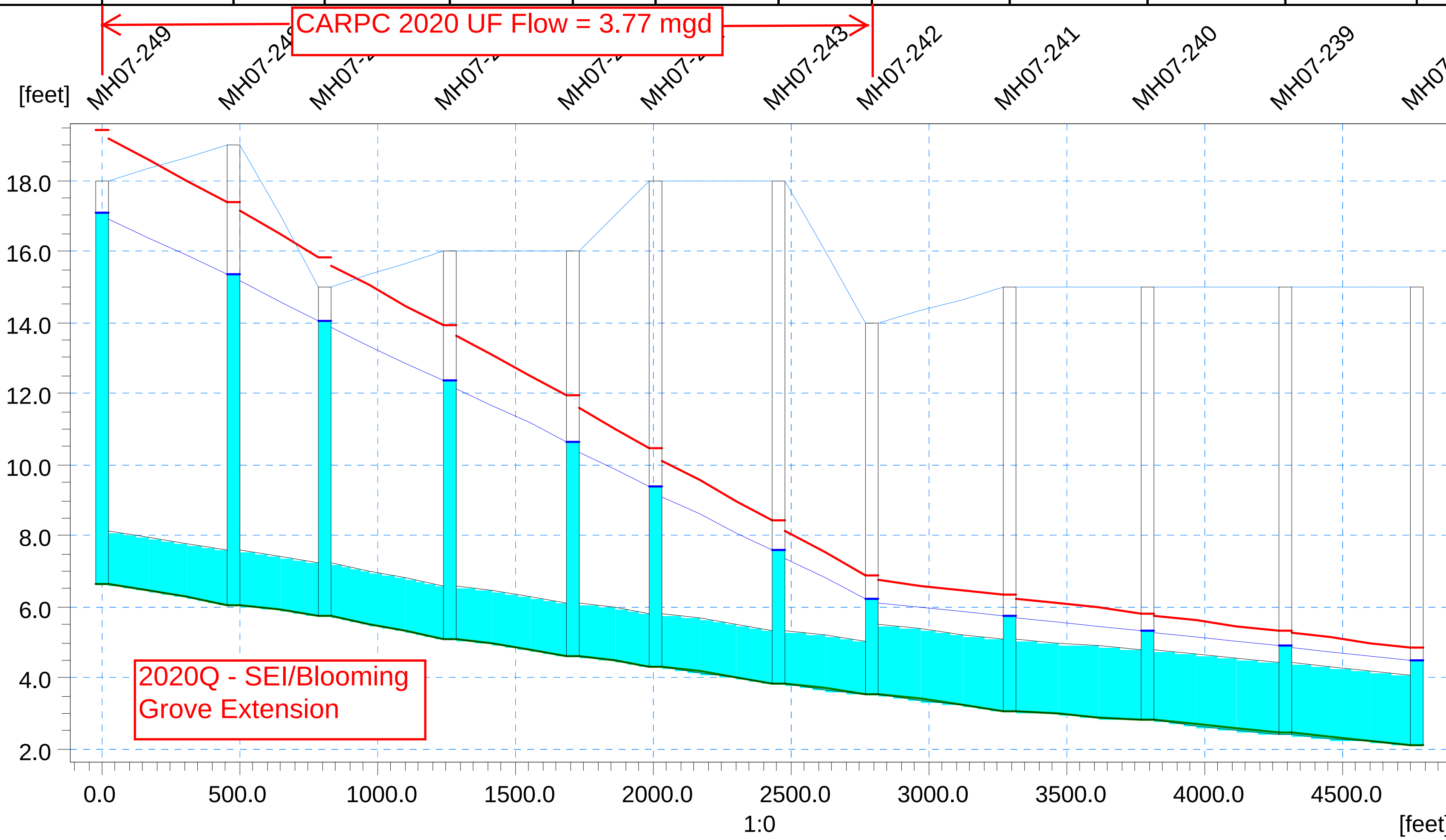
Length			450.00	405.00			467.00	552.00	578.00	490.00		401.00	575.00	543.00	551.00	434.00	493.00	476.00	450.00	441.00	505.00	480.00	466.00	449.00	476.00	[m]
Diameter	1.50	1.50	1.50	1.50			1.75	1.75	1.75	1.75	1.75	1.75	1.75	1.75	1.75	1.75	1.75	1.75	2.00	2.00	2.00	2.00	2.00	2.00	2.00	[m]
Slope o/oo	3.20	2.93	3.07	2.67			1.31	1.49	1.49	1.33	1.49	1.37	1.48	2.10	2.11	2.30	2.01	2.23	1.31	1.32	1.21	1.62	1.24	1.31	1.26	

WATER LEVEL BRANCHES - 21-5-2004 17:33:11 2020O_FEI.prf



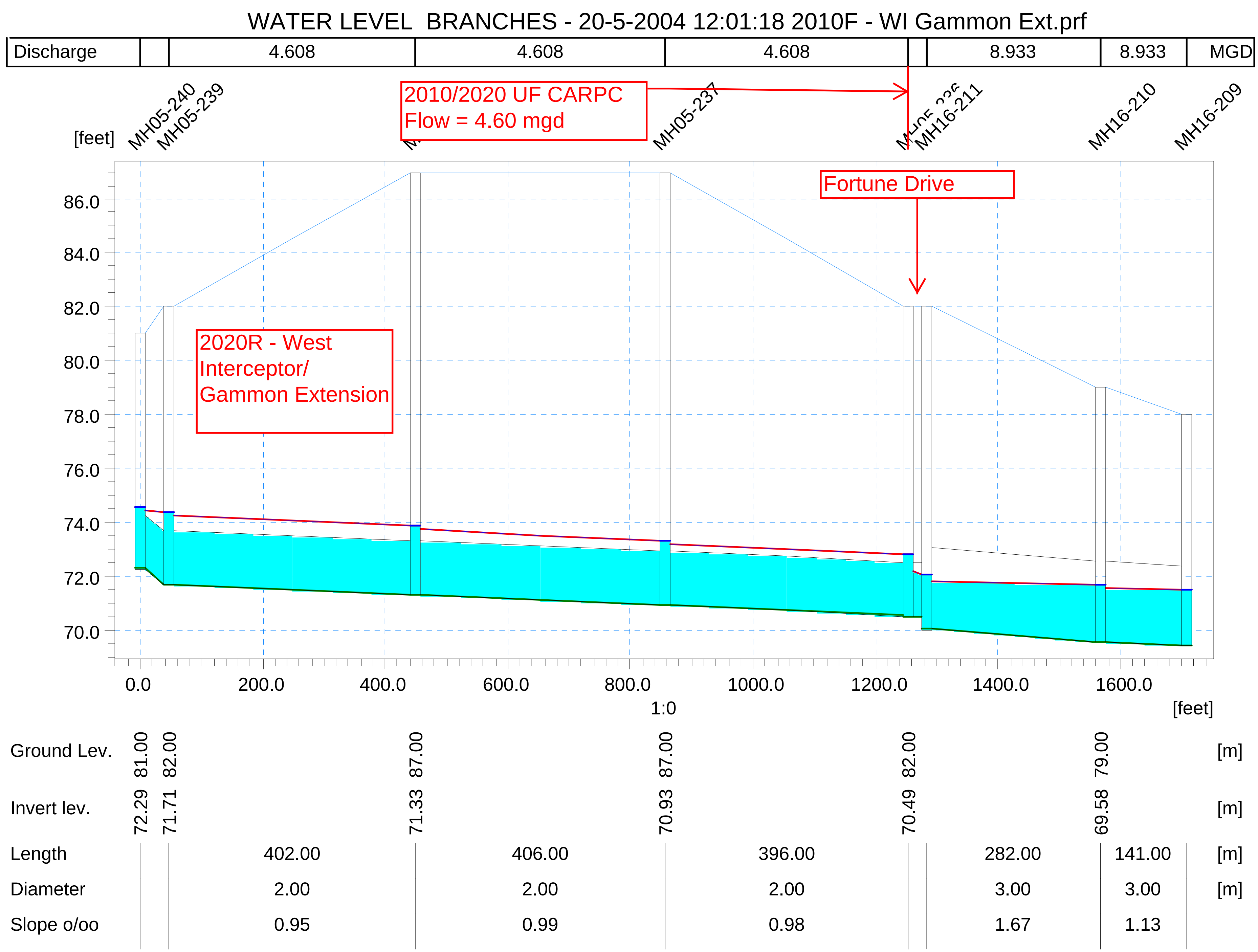
WATER LEVEL BRANCHES - 21-5-2004 02:10:20 2020Q - BG Ext.PRF

Discharge	3.748	3.763	3.777	3.789	3.799	3.806	3.813	3.817	3.822	3.826	3.829	MGD



Ground Lev.	18.00	19.00	15.00	16.00	16.00	18.00	18.00	14.00	15.00	15.00	15.00	[m]
Invert lev.	6.66	6.07	5.73	5.11	4.63	4.33	3.83	3.54	3.07	2.80	2.43	[m]

Length	474.00	337.00	450.00	450.00	300.00	445.00	338.00	502.00	497.00	498.00	479.00	[m]
Diameter	1.50	1.50	1.50	1.50	1.50	1.50	1.50	2.00	2.00	2.00	2.00	[m]
Slope o/oo	1.24	1.01	1.38	1.07	1.00	1.12	0.86	0.94	0.54	0.74	0.71	


WATER LEVEL BRANCHES - 20-5-2004 12:01:18 2010F - WI Gammon Ext.prf
Discharge
4.608
4.608
4.608
8.933
8.933
MGD
2010/2020 UF CARPC
Flow = 4.60 mgd
MH05-240
MH05-239
MH05-237
MH16-211
MH16-210
MH16-209
[feet]
Fortune Drive
2020R - West
Interceptor/
Gammon Extension
86.0
84.0
82.0
80.0
78.0
76.0
74.0
72.0
70.0
0.0
200.0
400.0
600.0
800.0
1000.0
1200.0
1400.0
1600.0
1:0
[feet]
Ground Lev.
81.00
82.00
87.00
87.00
82.00
79.00
[m]
Invert lev.
72.29
71.71
71.33
70.93
70.49
69.58
[m]
Length
402.00
406.00
396.00
282.00
141.00
[m]
Diameter
2.00
2.00
2.00
3.00
3.00
[m]
Slope o/oo
0.95
0.99
0.98
1.67
1.13

Appendix A6
Lower Badger Mill Creek Interceptor

MADISON METROPOLITAN SEWERAGE DISTRICT

Lower Badger Mill Creek Sewer Service Report

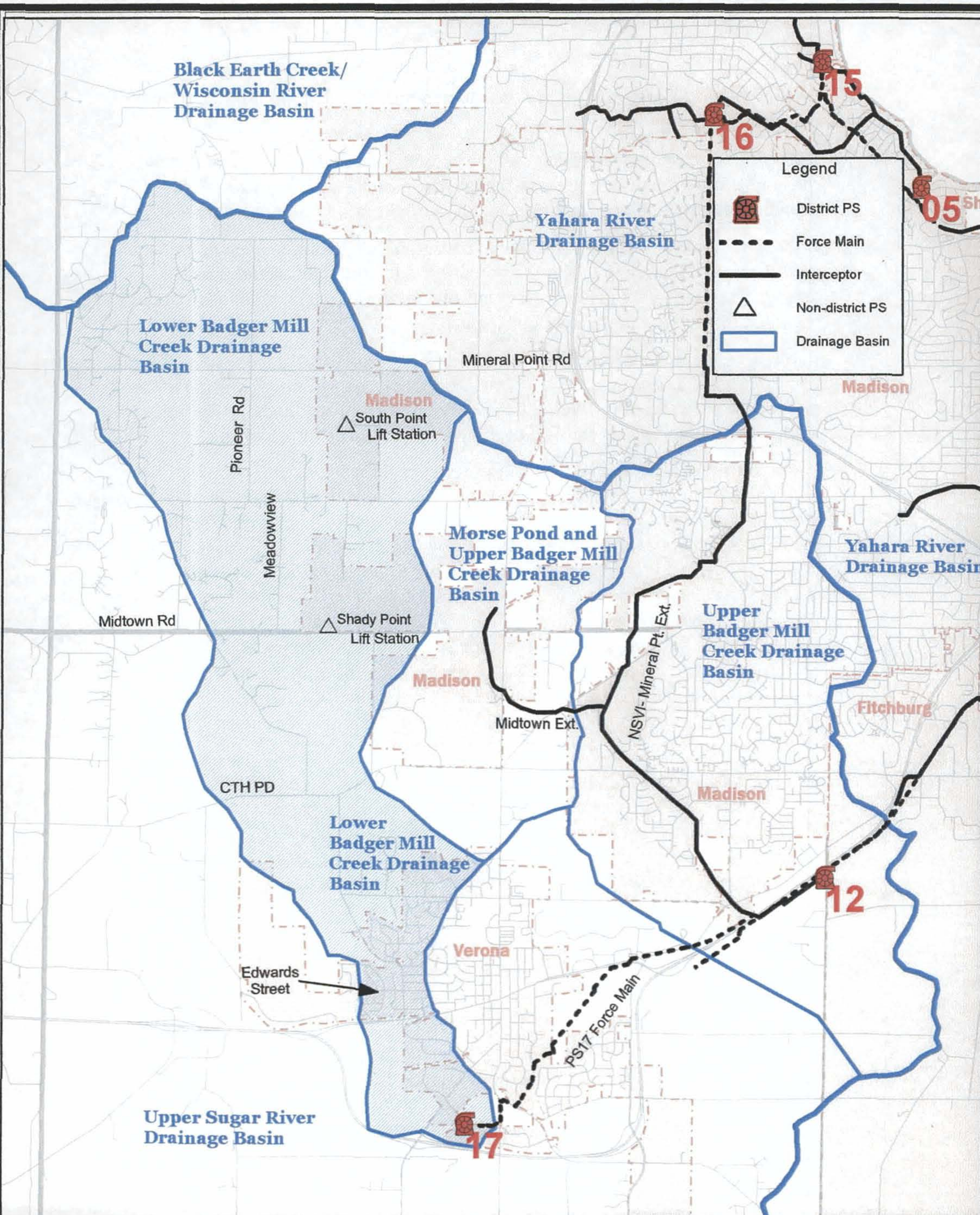
Lead Author: Jon Schellpfeffer

January 2005

Introduction

The purpose of this report is to present an analysis of options to provide sewer service in the Lower Badger Mill Creek watershed. The location of this watershed is shown in Figure 1. There are four municipal entities in this watershed, and the timing of sewer service availability is somewhat different in each area.

1. The Town of Middleton is the furthest upstream municipality in the watershed. There is currently no municipal sewer service in the town although there is significant residential development on large rural lots. These homes are served by private septic tank and drain field systems, and the town has no plans to replace these systems through the provision of public sewerage. However, it would be prudent to include capacity in any long-term sewer system facilities for these areas and other vacant land in the town.
2. The City of Madison portion of this watershed is north of Midtown Road and is expected to develop rapidly over the next decade. The first City of Madison development in this watershed occurred in 2000. The area is currently served by two small lift stations. The South Point Lift Station serves a proposed industrial/commercial area south of Mineral Point Road and discharges to the City of Madison's Wexford Interceptor that flows to Pumping Station No. 16. The Shady Point Lift Station serves the southern portion of the city's development north of Midtown Road and pumps easterly to the Nine Springs Valley Interceptor – Midtown Extension.
3. The Town of Verona portion of this watershed is located between Midtown Road on the north and the City of Verona to the south. The Town of Verona has no plans to provide sewer service in the area. The City of Madison's 1990 Peripheral Development Plan also calls for this area to be permanent open space. However, this area is adjacent to developing areas in the City of Madison and the City of Verona, and unless an inter-municipal boundary agreement is developed to permanently preserve this area as agricultural open space, it is likely to be annexed to one of these cities and developed within the next few decades. Unless a definitive agreement is in place to preserve this area as permanent open space, any sewerage facilities constructed to serve this watershed should be built with sufficient capacity to serve this area.
4. The City of Verona is located at the downstream end of this watershed. The city is growing rapidly in this area and plans are underway to increase sewerage capacity to serve this growth. The District intends to work with the city to assure that any new sewerage facilities constructed in this area have sufficient capacity to serve the areas upstream of the city in the future.



MMSD	**Madison Metropolitan Sewerage District**	Figure 1 Lower Badger Mill Creek Drainage Basin	Prepared by: JP Approved by: JS Date: 1/12/2005

A 20-year present worth cost analysis was conducted that included the following major variables related to serving this area:

- Rate of growth in the watershed – a high growth rate double the current growth rate and a low growth rate equal to the current growth rate were used. This growth is assumed to occur outside of the South Point Lift Station service area.
- Provision of sewer service in the Town of Verona – one option was included that would provide service in this area immediately, another that would provide service within the next fifteen years, and two options were included that would maintain this area as permanent open space.
- Future treatment plant locations – all options and growth rates were analyzed for the scenario of a new Sugar River Treatment Plant that would go on-line in 2020 and for the scenario of new relief sewers along the route of the Nine Springs Valley Interceptor and expansion of the Nine Springs Treatment Plant.

  Under the most likely scenario, a Sugar River regional treatment plant will be constructed when the capacity is needed to economically maintain the inter-basin water balance and to relieve the Nine Springs Valley Interceptor and loadings to the Nine Springs treatment plant. Initial flows to this plant will be in the range of 4 to 5 mgd, and it will have a design capacity of 7 to 10 mgd. It will be located south of the City of Verona. Pumping Station No. 17 will be modified to serve as the influent pumping station to this plant. A diversion sewer will be constructed from the Nine Springs Valley Interceptor – Mineral Point Extension to Pumping Station No. 17 to divert the wastewater generated in the Upper Badger Mill Creek watershed from the Nine Springs treatment plant to the new Sugar River treatment plant. Existing flows from the City of Verona and the Lower Badger Mill Creek watershed would also be treated at this new plant.

  In a less likely scenario, all wastewater would continue to be treated at the Nine Springs plant. Then, the existing Nine Springs Valley Interceptor downstream of Pumping Station No. 12 would require relief beginning in about 2020. The Nine Springs Plant would require expansion between 2025 and 2030. To maintain the water balance between the Sugar River and Yahara River basins, additional facilities would need to be constructed to return effluent from the Nine Springs plant to the Sugar River basin.

The area in the City of Madison north of Midtown Road in the Lower Badger Mill Creek drainage basin had 260 residential connections as of May, 2004. This area started to develop in 2000. The average growth rate has been 65 connections per year.

The Shady Point Lift Station serves the current development in this basin and is located in the Hawks Landing development on Midtown Road. It has a capacity of 630 gallons per minute (907,200 gallons per day) and a current average daily flow of 45,000 gallons per day. It has sufficient capacity to serve 1,250 residential connections. At the current growth rate in this basin, it would reach this level in 2020. If the growth rate were to double to 130 new connections per year, it would reach full capacity in 2012.

The Shady Point Lift Station is not located on the floor of the Lower Badger Mill Creek valley. In order to serve 1,250 connections, another small lift station could be built on the valley floor and pump to the Shady Point Lift Station. The location of this small lift station would coincide with the location of the interceptor sewer that would serve this valley. It would be located 1,000 feet west of the Shady Point Lift Station, which is located on the north side of Midtown Road as shown on Figure 1.

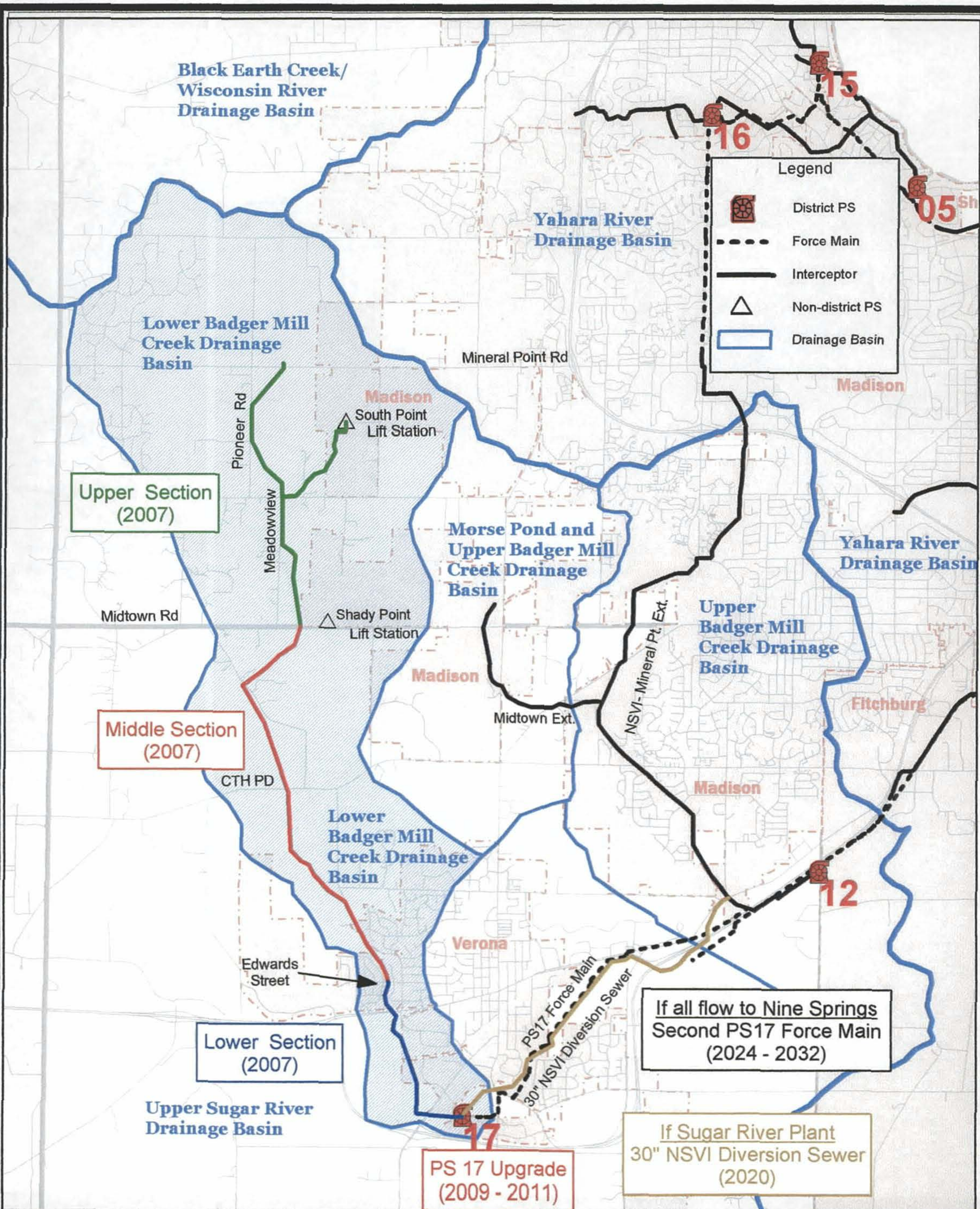
A second lift station in this basin, the South Point Lift Station, is located south of Mineral Point Road at the site of the City of Madison's future public works building. It has a capacity of 500 gallons per minute (720,000 gallons per day), but no current usage. It is currently configured to pump to the City of Madison's Wexford Interceptor that flows to Pumping Station No. 16. The Wexford Interceptor has sufficient capacity to handle an additional 500 gpm of flow, but no more. If the capacity of the this interceptor is reached, there is an option to divert the flows from this lift station to the City of Madison's RIK Interceptor, which also flows to Pumping Station No. 16. This would require extending sewers from the Blackhawk Neighborhood north of Old Sauk Road south to Mineral Point Road, which is part of the long-range plan for that neighborhood. The existing South Point Lift Station has sufficient capacity to serve 1,000 equivalent dwelling units which should be enough to handle the flow from the first phase development in this area as defined in the City of Madison's 2004 Pioneer Neighborhood Development Plan.

Development in the Lower Badger Mill Creek basin north of Midtown Road will continue as the City of Madison grows in this area. The City of Madison has developed neighborhood plans for all lands east of Pioneer Road and Meadowview Road in this basin, and much of this area has been annexed to the city. Construction of additional small lift stations that pump over the eastern drainage divide of this basin to existing sewers that flow to the Nine Springs Valley Interceptor is impractical as a long-range solution to providing sewer service. As this area builds out, a more economical system with better reliability will be needed. This can best be provided by the construction of an interceptor sewer north of Midtown Road. This interceptor should be built by 2007 with sufficient capacity to serve areas in the City of Madison and the Town of Middleton.

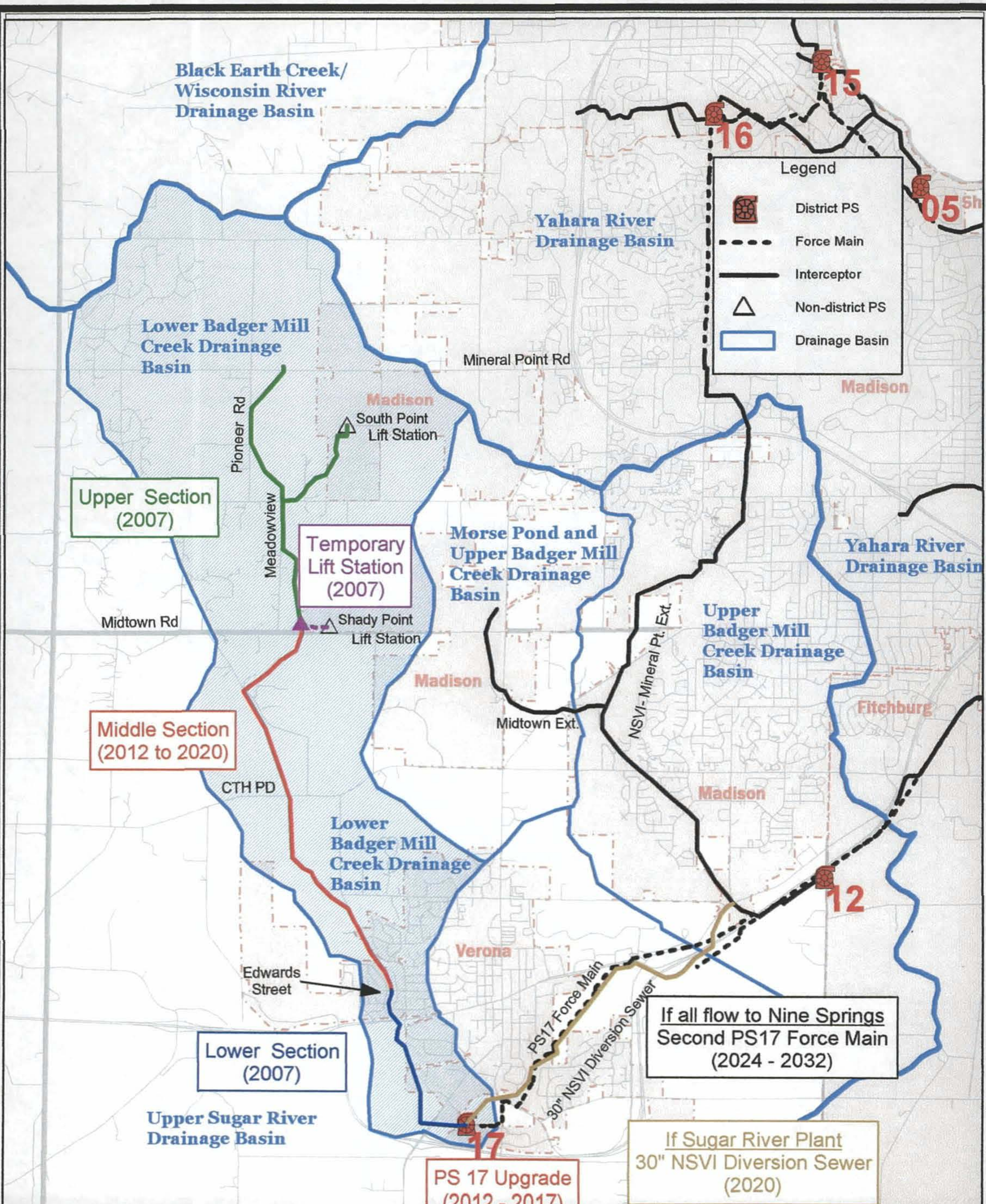
The floor of the Lower Badger Mill Creek valley south of Midtown Road includes a narrow drainage way through a section of large rural residential lots north of Shady Oak Lane and then spreads over a broad area of prime farmland that extends from Shady Oak Lane to the north side of the City of Verona. The narrow drainage way north of Shady Oak Lane dictates the location of an interceptor sewer in this reach of the valley. There is no immediate need for service in this valley south from Midtown Road to the north side of the City of Verona. The Town of Verona and the City of Madison both show this area to be permanent open space in their land use plans. However, no boundary agreements are yet in place that would preserve this area as open space. If no such agreements are reached, development in this area can logically be expected at some point in the future since it is adjacent to City of Madison development to the north and east and to City of Verona development to the south. If an interceptor was built to convey wastewater from the area north of Midtown Road to Pumping Station No. 17 on the south side of the City of Verona, it could also provide service to this area if required in the future.

The immediate question is how the wastewater generated north of Midtown Road should be handled. Four options are described below and shown on Figures 2, 3, 4, and 5. In addition to the location of the various components included in each option, the year, or the range of years, when the component will be constructed is also shown on these figures. All four options assume that the portion of the interceptor north of Midtown Road and the lower portion of the interceptor from Pumping Station No. 17 to Edwards Street in the City of Verona will be built in 2007. It is also assumed that the South Point Lift Station will remain in service until permanent facilities are constructed with sufficient capacity to serve the entire basin north of Midtown Road.

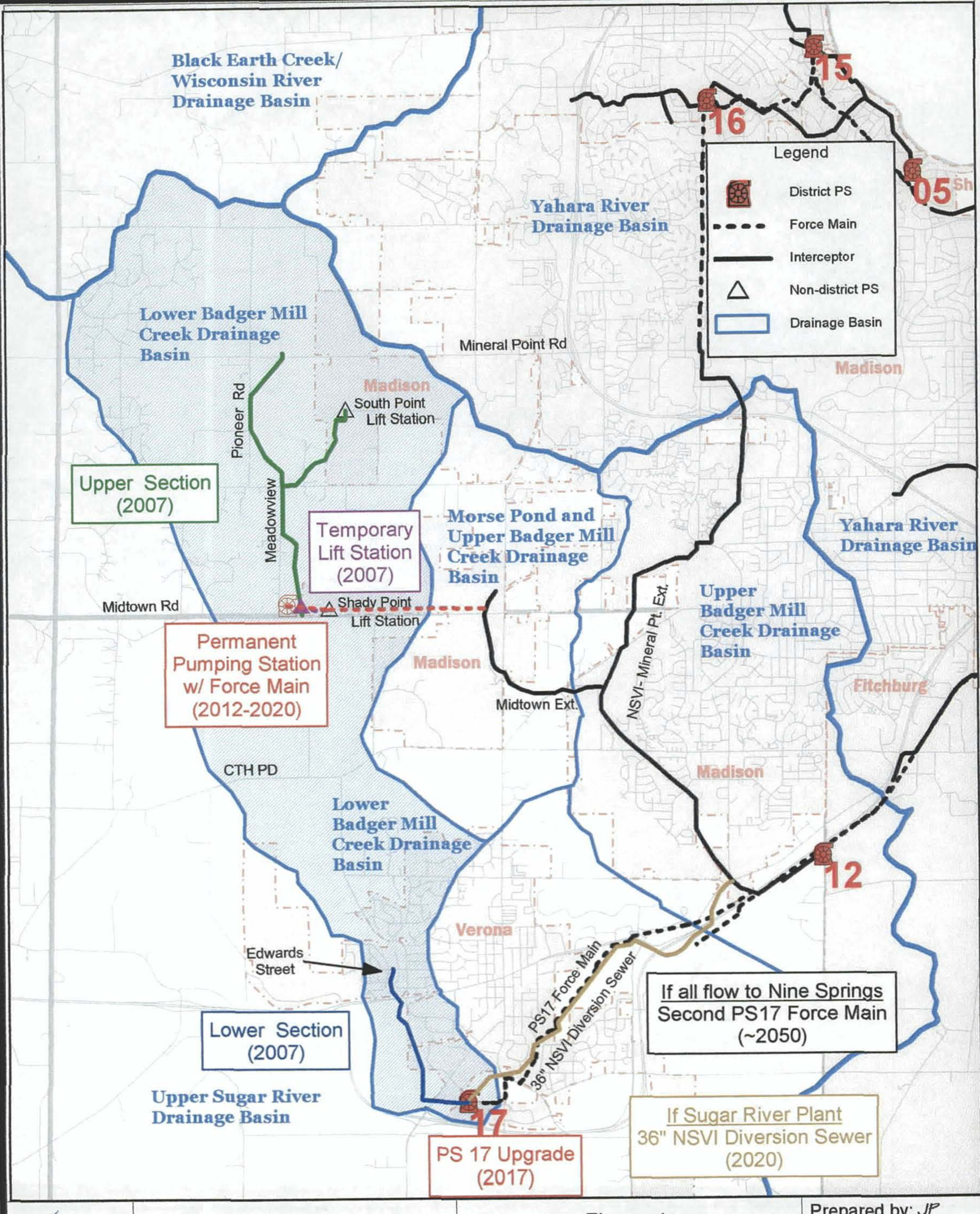
- Option 1 – Build the intermediate portion of the interceptor from Edwards Street to Midtown Road in 2007 (Figure 2).
- Option 2 – Build a temporary lift station at the terminus of the upper portion of the interceptor near Midtown Road and 1,000 feet of force main to pump wastewater to the Shady Point Lift Station in 2007. When the Shady Point Lift Station reaches capacity, build the intermediate portion of the interceptor from Edwards Street to Midtown Road (Figure 3).
- Option 3 – Build a small lift station at the terminus of the upper portion of the interceptor near Midtown Road and 1,000 feet of force main to pump wastewater to the Shady Point Lift Station in 2007. When the Shady Point Lift Station reaches capacity, build a permanent lift station and relief force main that would replace both the Shady Point Lift Station and the small station and would pump to the Midtown Extension (Figure 4).
- Option 4 – Build a permanent pumping station at the terminus of the upper portion of the interceptor near Midtown Road and 1,000 feet of force main to pump wastewater to the Shady Point Lift Station in 2007. When the Shady Point Lift Station reaches capacity, build a relief force main to the Midtown Extension from this station and replace the pumps and motors with larger units (Figure 5).



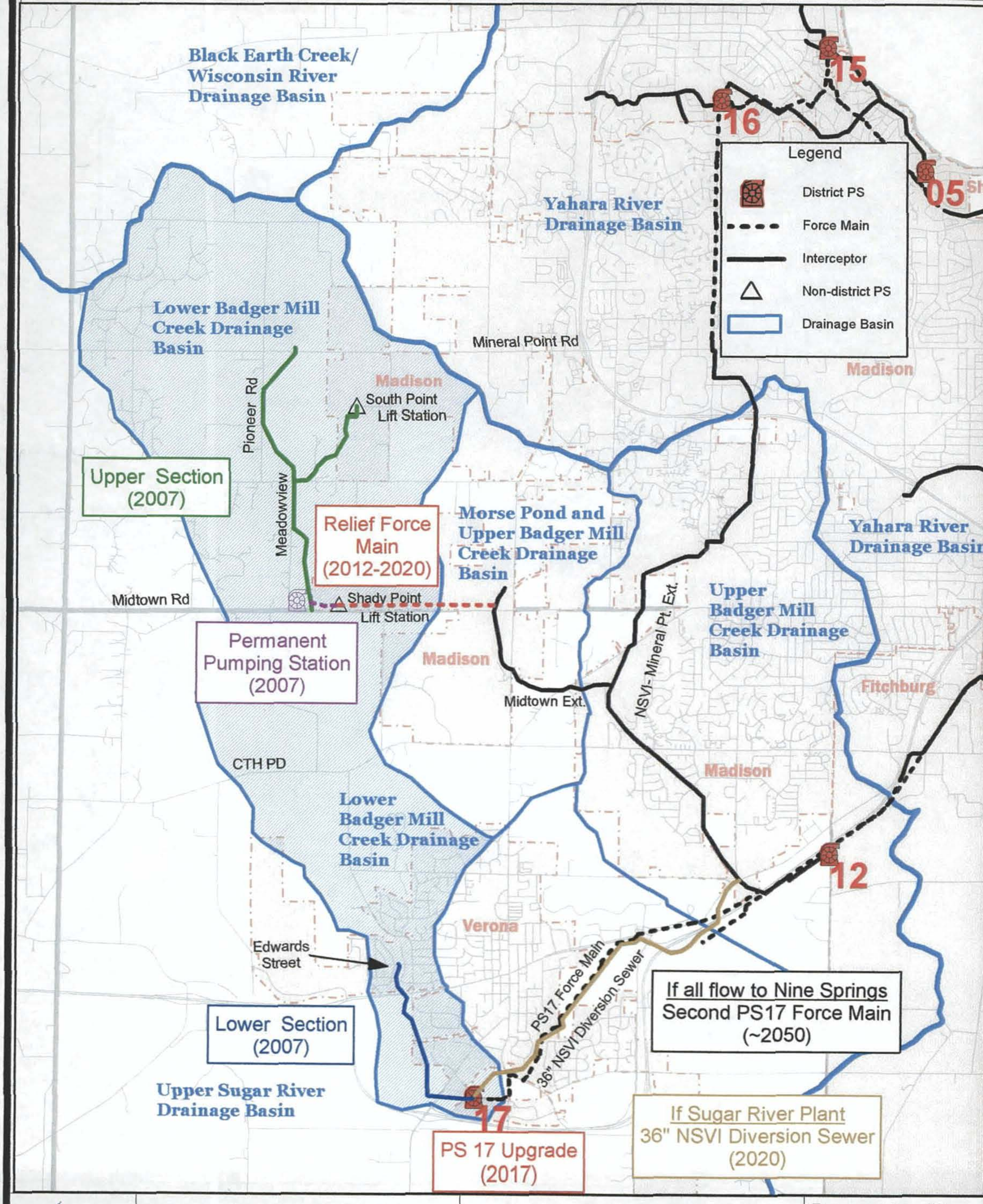
MMSD	**Madison Metropolitan Sewerage District**	Figure 2 Option 1 Facilities Plan	Prepared by: JP Approved by: JS Date: 1/12/2005



Madison Metropolitan Sewerage District	Figure 3 Option 2 Facilities Plan	Prepared by: JP Approved by: JS Date: 1/12/2005



MMSD	**Madison Metropolitan Sewerage District**	Figure 4 Option 3 Facilities Plan	Prepared by: JP Approved by: JS Date: 1/12/2005



MMSD	**Madison Metropolitan Sewerage District**	Figure 5 Option 4 Facilities Plan	Prepared by: JP Approved by: JS Date: 1/12/2005

Impact on Downstream Facilities

Pumping Station No. 17. This pumping station has a capacity of 2,900 gpm with two of its three pumps operating in parallel. This capacity is sufficient for an average daily flow of 1.05 mgd. Once this flow is reached, new 250 horse-power motors and variable-speed drives can be installed on the existing pumps to increase the peak capacity to 4,300 gpm with two of the three pumps operating in parallel. This capacity is sufficient for an average daily flow of 1.68 mgd. Once the average daily flow reaches this level, a second force main would need to be constructed to handle higher flows.

Under Option 1 all flows in the watershed would flow by gravity to this pumping station beginning in 2007. Depending on when the City of Madison built sewers to relieve the South Point Lift Station, flows from that service area would be handled by Pumping Station No 17 as early as 2007 also. This would necessitate increasing the capacity of Pumping Station No. 17 by upgrading the motors and drives in 2009 under the high growth scenario and in 2011 under the low growth scenario.

Under Option 2, only flows from the Verona Urban Service Area would be handled by this pumping station until the intermediate portion of the interceptor were constructed. Under the high growth scenario, this would happen in 2012, and would require upgrading the motors and drives at Pumping Station No. 17 at that time. Under the low growth scenario, the motors and drives would be upgraded in 2017 and the intermediate portion of the interceptor would be built in 2020.

For both Options 1 and 2 the second force main to the Nine Springs Valley Interceptor would be required in 2024 under the high growth scenario and in 2032 under the low growth scenario. This force main would not be necessary if a Sugar River treatment plant were constructed.

Costs associated with modifying Pumping Station No. 17 for use as the main pumping station to a new Sugar River treatment plant would be the same under all four service options, so they have not been included in the present worth cost analysis. Under Options 3 and 4, Pumping Station No. 17 would handle only flows from the Verona Urban Service Area if all flows continue to be pumped to the Nine Springs plant. In that case, the motors and drives at this pumping station would be upgraded in 2017, and the second force main would not be required until about 2050.

Nine Springs Valley Interceptor – Mineral Point Extension (NSVI-MP Ext). The portion of this interceptor from its junction with the Midtown Extension to its junction with the Pumping Station No. 17 force main will carry different flows under the various options. Replacement of this portion of the NSVI-MP Ext will not until after 2050 under options 1 and 2, and will be needed no sooner than 2030 under options 3 and 4. Costs that far in the future have no impact on the 20-year present worth of an option and were not included in this analysis.

Nine Springs Valley Interceptor Diversion Sewer to Pumping Station No. 17. When the Sugar River treatment plant is built, all wastewater generated in the Lower Badger Mill Creek basin will be treated at that plant. The wastewater generated in the Upper Badger Mill Creek basin, which is served by the NSVI-MP Ext, will also be treated at that plant. This will require a new interceptor that would be built to divert a portion of the flow from the NSVI-MP Ext to Pumping Station No. 17. It will parallel the Pumping Station No. 17 force main as shown on Figures 2 - 5. This interceptor will be built in 2020. Under Options 1 and 2, it would be a 30-inch sewer. Under Options 3 and 4 it would be a 36-inch sewer.

Present Worth Cost Analysis

The following table shows the results of the 20-year present worth cost analysis. Details are included in the appendix.

	Treatment at Nine Springs		New Sugar River Plant	
Option	High Growth	Low Growth	High Growth	Low Growth
1	$2,900,000	$2,741,000	$3,511,000	$3,397,000
2	$2,783,000	$2,067,000	$3,390,000	$2,723,000
3	$3,855,000	$2,737,000	$4,583,000	$3,570,000
4	$4,214,000	$4,100,000	$5,066,000	$4,933,000

In the table above, costs can only be compared within each column. Although the present worth analysis presents data for two different treatment plant scenarios, the costs for a similar option and growth rate cannot be compared across the treatment plant scenarios since the full costs associated with the future treatment plant scenarios are not included. Rather, the costs of the options under each growth rate column can be compared since each of these options includes the costs of all the facilities that may vary in size or time of construction for that particular growth rate and that particular treatment plant scenario.

Option 2 provides the lowest present worth cost under all four scenarios. Two sensitivity analyses were conducted between Options 1 and 2 to further study the cost difference. In the first, the growth rate was increased until the present worths of the two options were equal. This analysis showed that the lift station built in 2007 at Midtown Road would have to have at least a four-year service life to be cost-effective. This would require about 140 connections to be added in this area each year between 2004 and 2011. Option 1 would be equally cost-effective in that case. The high growth rate analyzed in this study assumes the addition of 130 new homes a year. A growth rate higher than 140 connections per year is not out of the question.

The second sensitivity analysis considered added cost over time for constructing the intermediate portion of the interceptor between Verona and Midtown Road. The longer the time until this portion of the interceptor is built, the more congested the area along the route of this sewer will become. Added development may mean more road crossings, poorer access for construction, and similar situations that would add to the initial cost of the facility. Under the high growth rate conditions analyzed, this congestion would need to increase the costs at a rate of three-fourths of a percent per year beyond the rate of construction inflation to result in Option 1 being more cost-effective than Option 2. This amounts to an increase above the rate of construction inflation of $23,000 a year. This would not be an unreasonable expectation of increased cost. Under the low growth rate conditions analyzed, this congestion would need to increase the costs at a rate of five and three-fourths of a percent per year beyond the rate of construction inflation to result in

Option 1 being more cost-effective than Option 2. This amounts to an increase above the rate of construction inflation greater than $175,000 a year, and seems unlikely.

There are no conditions under which Option 3 or 4 would be the most cost-effective option. If other considerations favored the construction of a permanent lift station at Midtown Road, Option 3 is always more cost-effective than Option 4.

Non-Economic Issues

Options 1 and 2 would ultimately provide service to the entire valley and would result in the most reliable service in the likely scenario of a future regional treatment plant in the Sugar River basin. Options 3 and 4 would require that additional sewers be built to serve the intermediate portion of the valley.

Options 3 and 4 are consistent with the Town of Verona's long-range plans for preserving farmland in this portion of the town and the City of Madison's 1990 Peripheral Development Plan that calls for the area between Midtown Road and the City of Verona to remain as permanent open space between the cities of Madison and Verona. Although the presence of an interceptor does not guarantee development of the adjacent areas, it does increase the pressure to develop these lands. Option 2 would reduce development pressure in this area for a short period of time, and Option 1 would result in the greatest amount of pressure to develop this area.

It is envisioned that Pumping Station No. 17 located on the south side of the City of Verona would serve as the main pumping station to a treatment plant in the Sugar River basin. The interceptor system under options 1 and 2 would result in the most efficient and reliable service to the Lower Badger Mill Creek basin since the wastewater generated there would only be pumped one time, at Pumping Station No. 17, to reach the treatment plant. In addition to double-pumping of the wastewater generated in the Lower Badger Mill Creek basin under options 3 and 4, the capacity of the diversion interceptor from the NSVI-MP Ext to Pumping Station No. 17 would need to be greater than under options 1 or 2.

If all wastewater continues to be pumped to the Nine Springs plant, there are only minor service advantages to options 1 and 2 since there would be one less pumping station to operate and maintain. All wastewater generated in the Lower Badger Mill Creek basin would be pumped three times to reach the plant.

When the South Point Lift Station reaches capacity and needs to be relieved, downstream facilities that relieve the Shady Point Lift Station under Option 2, 3, or 4 must be in place. It is possible that relief of the South Point Lift Station will be the milestone that triggers the need to relieve the Shady Point Lift Station, rather than growth in the remainder of the watershed north of Midtown Road. As this condition nears, it may be possible to prolong the life of the downstream temporary facilities by building an overflow structure at the South Point Lift Station that would allow flows in excess of this lift station's capacity to be bypassed to the relief sewer rather than simply abandoning this lift station and diverting all of its service area flows to the relief sewer at that time.

Recommendations

1. The temporary lift station at Midtown Road and force main to the Shady Oak Lift Station should be constructed in 2007. This will preserve both the future option of building a full gravity interceptor system to serve this valley and the future option of building a permanent lift station at Midtown Road once the Shady Oak Lift Station reaches its capacity. Additional information is expected to be available at that time to aid in the selection of one of these options. The comprehensive plans of the City of Madison, the City of Verona, and the Town of Verona will be adopted and may include a more definitive plan for the area between Midtown Road and the City of Verona, and a final decision concerning a future Sugar River Treatment Plant may have been made.
2. To preserve capacity at the Shady Point Lift Station, the South Point Lift Station should remain in service as long as possible.
3. The District and the City of Madison should enter an agreement that would allow the District to purchase a portion of the capacity of the interceptor north of Midtown Road in the future if it becomes necessary to provide service in the Town of Middleton.
4. To maintain the flexibility to implement Option 2 in the future, the following activities are recommended:
    a. The District should work with the City of Madison and the City of Verona as they design the upper and lower reaches of this interceptor to assure that an overall interceptor design is formulated that would allow for the later construction of the intermediate portion of this interceptor without compromising the capacity of this sewer. Along those reaches of this interceptor where the topography dictates its location, easements should be secured at the time of design to lessen the future work load and the potential for delays that sometimes accompany the procurement of easements.
    b. The District and the City of Verona should enter an agreement that defines the area in this watershed that is expected to be served by the City of Verona and how the costs for the portion of the interceptor built by the City of Verona should be split between the City of Verona and the District.
5. The District should communicate with the Town of Middleton and the Town of Verona concerning the long-range plan for providing service in this watershed and the impact it may have on their jurisdictions.
6. The District should provide this information to the Dane County CAPD for their review and for use in updating the Dane County Water Quality Plan.

Appendix

Capital Cost Estimates,
Flow and Power Usage Estimates,
and
Present Worth Cost Analyses Details

Capital Costs

Lower Badger Mill Creek Interceptor (LBMC Int)

This is the portion of the interceptor from Edwards Street in the City of Verona to Midtown Road.

Description	Unit Cost	Units	Number of Units	Cost
30" Interceptor from Edwards Street to Midtown Road	$ 130.00	Lineal Feet	19,300	$ 2,500,000
Special construction in Nine Mound Road ravine	$ 500.00	Lineal Feet	1,000	$ 500,000
Special construction in Shady Oak Lane ravine	$ 500.00	Lineal Feet	200	$ 100,000
Total Cost (2004)				**$ 3,100,000**

Pumping Station No. 17 Motors and Drives Replacement

This project involves replacing the original 100 HP motors and variable-speed drives with 250 HP motors and new variable-speed drives and associated electrical work.

Description	Unit Cost	Units	Number of Units	Cost
250 HP Motor and Variable-Speed Drive Units	$ 100,000	Each	3	$ 300,000
Total Cost (2004)				**$ 300,000**

Second PS 17 Force Main

This force main would run from the PS 17 dry well to the Nine Springs Valley Interceptor and be parallel to the original force main.

Description	Unit Cost	Units	Number of Units	Cost
16" Force Main from PS 17 to NSVI	$ 90.00	Lineal Feet	13,500	$ 1,200,000
Total Cost (2004)				**$ 1,200,000**

Small Lift Station at Midtown Road (Small LS)

This would be a submersible pumping station design, similar to the Shady Point Lift Station.

Description	Unit Cost	Units	Number of Units	Cost
8-foot diameter Dry Well and Wet Well	$ 30,000	Each	2	$ 60,000
Submersible Pumps and Piping	$ 40,000	Each	2	$ 80,000
Site Work	$ 10,000	Each	1	$ 10,000
Electrical Work	$ 25,000	Each	1	$ 25,000
Total Cost (2004)				**$ 175,000**

Force Main from Samll Lift Station at Midtown Road to the Shady Point Lift Station

Description	Unit Cost	Units	Number of Units	Cost
6" PVC Force Main	$ 50.00	Lineal Feet	1,000	$ 50,000
Total Cost (2004)				**$ 50,000**

Permanent Midtown Pumping Station

Description	Unit Cost	Units	Number of Units	Cost
General Construction - Structure	$ 1,000,000	Each	1	$ 1,000,000
Pumps and Variable-Speed Drives	$ 100,000	Each	3	$ 300,000
Other Mechanical Equipment and Piping	$ 350,000	Each	1	$ 350,000
Site Work	$ 50,000	Each	1	$ 50,000
Electrical Work	$ 500,000	Each	1	$ 500,000
Total Cost (2004)				**$ 2,200,000**

Force Main from Permanent Midtown Pumping Station to the Shady Point Lift Station

This force main would be constructed at a constant grade such that it could serve as a gravity sewer from the Shady Point Lift Station in the future.

Description	Unit Cost	Units	Number of Units	Cost
18" PVC Force Main	$ 100.00	Lineal Feet	1,000	$ 100,000
Total Cost (2004)				**$ 100,000**

Force Main from Permanent Midtown Pumping Station to Midtown Extension

This force main would be built parallel to and operate in parallel with the existing 10-inch force main from the Shady Point Lift Station to the Midtown Extension.

Description	Unit Cost	Units	Number of Units	Cost
18" PVC Force Main (includes pavement restoration)	$ 150.00	Lineal Feet	5,300	$ 800,000
Total Cost (2004)				**$ 800,000**

Pumping Station Power Requirements

High Growth Rate

					Daily KWH							PS 17 Alone		PS 17 w/LBMC	
	Average Day Flow					Midtown PS		PS 17 Alone		PS 17 w/LBMC		PS 17 Avg	PS 17 Avg	PS 17 Avg	PS 17 Avg
Year	Small LS	PS 17 Alone	PS 17 w/LBMC	Midtown PS	Small LS	One FM	Two FMs	PS 17 Current	PS 17 250 HP	PS 17 Current	PS 17 250 HP	Pumping Rate	Input Power	Pumping Rate	Input Power
	(gpd)	(gpd)	(gpd)	(gpd)	(KWH)	(KWH)	(KWH)	(KWH)	(KWH)	(KWH)	(KWH)	(gpm)	(KW)	(gpm)	(KW)
2004	45,000	789,735	834,735	45,000	34	134	120	488		510		765	23.00	765	23.00
2005	69,130	809,598	878,729	69,130	53	156	135	498		532		765	23.00	765	23.00
2006	93,261	829,461	922,722	93,261	71	179	150	508		542		765	23.00	801	23.43
2007	117,391	849,324	966,715	117,391	89	201	165	518		549		765	23.00	839	23.82
2008	141,522	869,187	1,010,708	141,522	107	224	180	528		557		765	23.00	877	24.22
2009	165,652	889,049	1,054,702	165,652	126	246	196	536			588	772	23.10	2,000	56.40
2010	189,783	908,912	1,098,695	189,783	144	269	211	539			608	789	23.30	2,000	56.40
2011	213,913	928,775	1,142,688	213,913	162	291	226	543			629	806	23.50	2,000	56.40
2012	238,043	948,638	1,186,681	238,043	181	314	241	546			650	823	23.66	2,000	56.40
2013	262,174	968,501	1,230,675	262,174	199	336	256	550			670	841	23.85	2,000	56.40
2014	286,304	988,364	1,274,668	286,304	217	359	271	553			691	858	24.00	2,000	56.40
2015	310,435	1,008,226	1,318,661	310,435	236	381	286	557			712	875	24.20	2,000	56.40
2016	334,565	1,028,089	1,362,655	334,565	254	404	301	560			732	892	24.38	2,000	56.40
2017	358,696	1,047,952	1,406,648	358,696	272	426	316		585		753	2,000	56.40	2,000	56.40
2018	382,826	1,067,815	1,450,641	382,826	291	449	331		594		774	2,000	56.40	2,000	56.40
2019	406,957	1,087,678	1,494,634	406,957	309	471	346		603		794	2,000	56.40	2,000	56.40
2020	431,087	1,107,541	1,538,628	431,087	327	494	361		613		815	2,000	56.40	2,000	56.40
2021	455,217	1,127,403	1,582,621	455,217	346	516	376		622		836	2,000	56.40	2,000	56.40
2022	479,348	1,147,266	1,626,614	479,348	364	539	392		631		857	2,000	56.40	2,000	56.40
2023	503,478	1,167,129	1,670,607	503,478	382	561	407		641		877	2,000	56.40	2,000	56.40
2024	527,609	1,186,992	1,714,601	527,609	401	584	422		650		761	2,000	56.40	2,000	46.80
2025	551,739	1,206,855	1,758,594	551,739	419	606	437		659		778	2,000	56.40	2,000	46.80
2026	575,870	1,226,718	1,802,587	575,870	437	629	452		669		795	2,000	56.40	2,000	46.80
2027	600,000	1,246,581	1,846,581	600,000	456	651	467		678		812	2,000	56.40	2,000	46.80

Note: For PS 17 average day flows greater than 1,680,000, a second 16" force main is assumed (2024 and later).

Low Growth Rate

					Daily KWH							PS 17 Alone		PS 17 w/LBMC	
	Average Day Flow					Midtown PS		PS 17 Alone		PS 17 w/LBMC		PS 17 Avg	PS 17 Avg	PS 17 Avg	PS 17 Avg
Year	Small LS	PS 17 Alone	PS 17 w/LBMC	Midtown PS	Small LS	One FM	Two FMs	PS 17 Current	PS 17 250 HP	PS 17 Current	PS 17 250 HP	Pumping Rate	Input Power	Pumping Rate	Input Power
	(gpd)	(gpd)	(gpd)	(gpd)	(KWH)	(KWH)	(KWH)	(KWH)	(KWH)	(KWH)	(KWH)	(gpm)	(KW)	(gpm)	(KW)
2004	45,000	789,735	834,735	45,000	34	134	120	488		510		765	23.00	765	23.00
2005	56,087	809,598	865,685	56,087	43	144	127	498		526		765	23.00	765	23.00
2006	67,174	829,461	896,635	67,174	51	155	134	508		537		765	23.00	778	23.19
2007	78,261	849,324	927,585	78,261	59	165	141	518		543		765	23.00	805	23.47
2008	89,348	869,187	958,534	89,348	68	175	148	528		548		765	23.00	832	23.75
2009	100,435	889,049	989,484	100,435	76	186	155	536		553		772	23.10	859	24.03
2010	111,522	908,912	1,020,434	111,522	85	196	162	539		559		789	23.30	886	24.31
2011	122,609	928,775	1,051,384	122,609	93	206	169	543			586	806	23.50	2,000	56.40
2012	133,696	948,638	1,082,334	133,696	102	217	176	546			601	823	23.66	2,000	56.40
2013	144,783	968,501	1,113,283	144,783	110	227	182	550			615	841	23.85	2,000	56.40
2014	155,870	988,364	1,144,233	155,870	118	237	189	553			630	858	24.00	2,000	56.40
2015	166,957	1,008,226	1,175,183	166,957	127	248	196	557			644	875	24.20	2,000	56.40
2016	178,043	1,028,089	1,206,133	178,043	135	258	203	560			659	892	24.38	2,000	56.40
2017	189,130	1,047,952	1,237,083	189,130	144	268	210		585		673	2,000	56.40	2,000	56.40
2018	200,217	1,067,815	1,268,032	200,217	152	279	217		594		688	2,000	56.40	2,000	56.40
2019	211,304	1,087,678	1,298,982	211,304	160	289	224		603		703	2,000	56.40	2,000	56.40
2020	222,391	1,107,541	1,329,932	222,391	169	299	231		613		717	2,000	56.40	2,000	56.40
2021	233,478	1,127,403	1,360,882	233,478	177	310	238		622		732	2,000	56.40	2,000	56.40
2022	244,565	1,147,266	1,391,832	244,565	186	320	245		631		746	2,000	56.40	2,000	56.40
2023	255,652	1,167,129	1,422,781	255,652	194	330	252		641		761	2,000	56.40	2,000	56.40
2024	266,739	1,186,992	1,453,731	266,739	203	341	259		650		775	2,000	56.40	2,000	56.40
2025	277,826	1,206,855	1,484,681	277,826	211	351	266		659		790	2,000	56.40	2,000	56.40
2026	288,913	1,226,718	1,515,631	288,913	219	361	273		669		804	2,000	56.40	2,000	56.40
2027	300,000	1,246,581	1,546,581	300,000	228	372	279		678		819	2,000	56.40	2,000	56.40

Options 1 thru 4 with High Growth

Option 1 with High Growth Rate

Project	Year	Construction Cost		O&M Costs				Salvage Value		Total 2007 Present Worth
		Cost in Year Constructed	2007 Present Worth	2007	Intermediate Year	2027	2007 Present Worth	Year-20	2007 Present Worth	
LBMC Int	2007	3,407,000	3,407,000	4,300	5,100	9,500	86,000	4,691,000	2,141,000	1,352,000
PS 17	2007			49,000	52,000		99,000			99,000
PS 17 - 250 HP Motors	2009	351,000	325,000		56,000	139,000	945,000	248,000	113,000	1,157,000
Second PS 17 FM	2024	2,253,000	1,157,000		134,000	164,000	220,000	2,377,000	1,085,000	292,000
Totals			**4,889,000**				**1,350,000**		**3,339,000**	**2,900,000**

Option 2 with High Growth Rate

Project	Year	Construction Cost		O&M Costs				Salvage Value		Total 2007 Present Worth
		Cost in Year Constructed	2007 Present Worth	2007	Intermediate Year	2027	2007 Present Worth	Year-20	2007 Present Worth	
Small LS	2007	192,000	192,000	6,800	12,700	0	42,000	0	0	234,000
FM to Shady Point LS	2007	55,000	55,000	0	0	0	0	0	0	55,000
PS 17	2007			48,000	59,000		237,000			237,000
LBMC Int	2012	3,988,000	3,278,000	0	5,300	9,500	65,000	5,117,000	2,335,000	1,008,000
PS 17 - 250 HP Motors	2012	386,000	317,000		68,000	139,000	781,000	310,000	141,000	957,000
Second PS 17 FM	2024	2,253,000	1,157,000		134,000	164,000	220,000	2,377,000	1,085,000	292,000
Totals			**4,999,000**				**1,345,000**		**3,561,000**	**2,783,000**

Option 3 with High Growth Rate

Project	Year	Construction Cost		O&M Costs				Salvage Value		Total 2007 Present Worth
		Cost in Year Constructed	2007 Present Worth	2007	Intermediate Year	2027	2007 Present Worth	Year-20	2007 Present Worth	
Small LS	2007	192,000	192,000	6,800	12,700	0	42,000	0	0	234,000
FM to Shady Point LS	2007	110,000	110,000	0	0	0	0	151,000	69,000	41,000
PS 17	2007			48,000	59,000		237,000			237,000
Permanent Midtown PS	2012	2,830,000	2,326,000	0	69,500	159,900	976,000	2,270,000	1,036,000	2,266,000
Force Main to Midtown Ext	2012	1,029,000	846,000	0	600	1,100	7,000	1,320,000	602,000	251,000
PS 17 - 250 HP Motors	2017	452,000	305,000		82,000	449,000	709,000	413,000	188,000	826,000
Second PS 17 FM	2050									
Totals			**3,779,000**				**1,971,000**		**1,895,000**	**3,855,000**

Option 4 with High Growth Rate

Project	Year	Construction Cost		O&M Costs				Salvage Value		Total 2007 Present Worth
		Cost in Year Constructed	2007 Present Worth	2007	Intermediate Year	2027	2007 Present Worth	Year-20	2007 Present Worth	
Permanent Midtown PS	2007	2,198,000	2,198,000	52,000	66,100		261,000	1,376,000	628,000	1,831,000
FM to Shady Point LS	2007	110,000	110,000	0	0	0	0	151,000	69,000	41,000
PS 17	2007			48,000	59,000		237,000			237,000
Upgrade Permanent Midtown PS	2012	386,000	317,000	0	69,500	159,900	976,000	310,000	141,000	1,152,000
Force Main to Midtown Ext	2012	1,029,000	846,000	0	600	1,100	7,000	1,320,000	602,000	251,000
PS 17 - 250 HP Motors	2017	452,000	294,000		95,000	152,000	602,000	426,000	194,000	702,000
Second PS 17 FM	2050									
Totals			**3,765,000**				**2,083,000**		**1,634,000**	**4,214,000**

Assumptions: Base interest rate is 4.0%
Construction cost escalation rate is 3.2%
Electric energy escalation rate is 6.0%
Annual O&M costs for interceptors are $0.20 per foot
Annual O&M costs for small lift stations are 2.0% of initial cost + energy costs for pumping
Annual O&M costs for PS 17 are based on historical data + energy costs for pumping
O&M costs increase at the base interest rate

Future Nine Springs Treatment Plant Expansion
Options 1 thru 4 with Low Growth

Option 1 with Low Growth Rate

Project	Year	Construction Cost: Cost in Year Constructed	Construction Cost: 2007 Present Worth	O&M Costs: 2007	O&M Costs: Intermediate Year	O&M Costs: 2027	O&M Costs: 2007 Present Worth	Salvage Value: Year-20	Salvage Value: 2007 Present Worth	Total 2007 Present Worth
LBMC Int	2007	3,407,000	3,407,000	4,300	5,100	9,500	86,000	4,691,000	2,141,000	1,352,000
PS 17	2007			49,000	57,000		199,000			199,000
PS 17 - 250 HP Motors	2011	374,000	320,000		61,000	154,000	1,002,000	289,000	132,000	1,190,000
Second PS 17 FM	2032									
Totals			**3,727,000**				**1,287,000**		**2,273,000**	**2,741,000**

Option 2 with Low Growth Rate

Project	Year	Construction Cost: Cost in Year Constructed	Construction Cost: 2007 Present Worth	O&M Costs: 2007	O&M Costs: Intermediate Year	O&M Costs: 2027	O&M Costs: 2007 Present Worth	Salvage Value: Year-20	Salvage Value: 2007 Present Worth	Total 2007 Present Worth
Small LS	2007	192,000	192,000	5,800	19,000	0	109,000	0	0	301,000
FM to Shady Point LS	2007	55,000	55,000	0	0	0	0	0	0	55,000
PS 17	2007			48,000	76,000		487,000			487,000
LBMC Int	2020	5,131,000	3,082,000	0	7,200	9,500	30,000	5,800,000	2,647,000	465,000
PS 17 - 250 HP Motors	2017	452,000	305,000		82,000	154,000	642,000	413,000	188,000	759,000
Second PS 17 FM	2032									
Totals			**3,634,000**				**1,268,000**		**2,835,000**	**2,067,000**

Option 3 with Low Growth Rate

Project	Year	Construction Cost: Cost in Year Constructed	Construction Cost: 2007 Present Worth	O&M Costs: 2007	O&M Costs: Intermediate Year	O&M Costs: 2027	O&M Costs: 2007 Present Worth	Salvage Value: Year-20	Salvage Value: 2007 Present Worth	Total 2007 Present Worth
Small LS	2007	192,000	192,000	5,800	19,000	0	109,000	0	0	301,000
FM to Shady Point LS	2007	110,000	110,000	0	0	0	0	0	0	110,000
PS 17	2007			48,000	76,000		487,000			487,000
Permanent Midtown PS	2020	3,642,000	2,187,000	0	92,500	130,900	404,000	3,481,000	1,589,000	1,002,000
Force Main to Midtown Ext	2020	1,324,000	795,000	0	900	1,100	4,000	1,497,000	683,000	116,000
PS 17 - 250 HP Motors	2017	452,000	305,000		82,000	138,000	604,000	413,000	188,000	721,000
Second PS 17 FM	2050									
Totals			**3,589,000**				**1,608,000**		**2,460,000**	**2,737,000**

Option 4 with Low Growth Rate

Project	Year	Construction Cost: Cost in Year Constructed	Construction Cost: 2007 Present Worth	O&M Costs: 2007	O&M Costs: Intermediate Year	O&M Costs: 2027	O&M Costs: 2007 Present Worth	Salvage Value: Year-20	Salvage Value: 2007 Present Worth	Total 2007 Present Worth
Permanent Midtown PS	2007	2,198,000	2,198,000	50,600	93,600		680,000	1,376,000	628,000	2,250,000
FM to Shady Point LS	2007	110,000	110,000	0	0	0	0	151,000	69,000	41,000
PS 17	2007			48,000	76,000		487,000			487,000
Upgrade Permanent Midtown PS	2020	497,000	298,000	0	92,500	130,900	404,000	475,000	217,000	485,000
Force Main to Midtown Ext	2020	1,324,000	795,000	0	900	1,100	4,000	1,497,000	683,000	116,000
PS 17 - 250 HP Motors	2017	452,000	305,000		82,000	138,000	604,000	413,000	188,000	721,000
Second PS 17 FM	2050									
Totals			**3,706,000**				**2,179,000**		**1,785,000**	**4,100,000**

Assumptions:
- Base interest rate is 4.0%
- Construction cost escalation rate is 3.2%
- Electric energy escalation rate is 6.0%
- Annual O&M costs for interceptors are $0.20 per foot
- Annual O&M costs for small lift stations are 2.0% of initial cost + energy costs for pumping
- Annual O&M costs for PS 17 are based on historical data + energy costs for pumping
- O&M costs increase at the base interest rate

Options 1 thru 4 with High Growth

Option 1 with High Growth Rate

Project	Year	Construction Cost		O&M Costs				Salvage Value		Total 2007 Present Worth
		Cost in Year Constructed	2007 Present Worth	2007	Intermediate Year	2027	2007 Present Worth	Year-20	2007 Present Worth	
LBMC Int	2007	3,407,000	3,407,000	4,300	5,100	9,500	86,000	4,691,000	2,141,000	1,352,000
PS 17	2007			49,000	52,000		99,000			99,000
PS 17 - 250 HP Motors	2009	351,000	325,000		56,000	170,000	1,192,000	248,000	113,000	1,404,000
30" NSVI Diversion to PS 17	2020	6,800,000	4,084,000	0	6,100	8,000	21,000	7,557,000	3,449,000	656,000
Totals			**7,816,000**				**1,398,000**		**5,703,000**	**3,511,000**

Option 2 with High Growth Rate

Project	Year	Construction Cost		O&M Costs				Salvage Value		Total 2007 Present Worth
		Cost in Year Constructed	2007 Present Worth	2007	Intermediate Year	2027	2007 Present Worth	Year-20	2007 Present Worth	
Small LS	2007	192,000	192,000	6,800	12,700	0	42,000	0	0	234,000
FM to Shady Point LS	2007	55,000	55,000	0	0	0	0	0	0	55,000
PS 17	2007			48,000	59,000		237,000			237,000
LBMC Int	2012	3,988,000	3,278,000	0	5,300	9,500	65,000	5,117,000	2,335,000	1,008,000
PS 17 - 250 HP Motors	2012	386,000	317,000		68,000	170,000	1,024,000	310,000	141,000	1,200,000
30" NSVI Diversion to PS 17	2020	6,800,000	4,084,000	0	6,100	8,000	21,000	7,557,000	3,449,000	656,000
Totals			**7,926,000**				**1,389,000**		**5,925,000**	**3,390,000**

Option 3 with High Growth Rate

Project	Year	Construction Cost		O&M Costs				Salvage Value		Total 2007 Present Worth
		Cost in Year Constructed	2007 Present Worth	2007	Intermediate Year	2027	2007 Present Worth	Year-20	2007 Present Worth	
Small LS	2007	192,000	192,000	6,800	12,700	0	42,000	0	0	234,000
FM to Shady Point LS	2007	110,000	110,000	0	0	0	0	151,000	69,000	41,000
PS 17	2007			48,000	59,000		237,000			237,000
Permanent Midtown PS	2012	2,830,000	2,326,000	0	69,500	159,900	976,000	2,270,000	1,036,000	2,266,000
Force Main to Midtown Ext	2012	1,029,000	846,000	0	600	1,100	7,000	1,320,000	602,000	251,000
PS 17 - 250 HP Motors	2017	452,000	305,000		82,000	138,000	604,000	413,000	188,000	721,000
36" NSVI Diversion to PS 17	2020	8,700,000	5,225,000	0	6,100	8,000	21,000	9,669,000	4,413,000	833,000
Totals			**9,004,000**				**1,887,000**		**6,308,000**	**4,583,000**

Option 4 with High Growth Rate

Project	Year	Construction Cost		O&M Costs				Salvage Value		Total 2007 Present Worth
		Cost in Year Constructed	2007 Present Worth	2007	Intermediate Year	2027	2007 Present Worth	Year-20	2007 Present Worth	
Permanent Midtown PS	2007	2,198,000	2,198,000	52,000	66,100		261,000	1,376,000	628,000	1,831,000
FM to Shady Point LS	2007	110,000	110,000	0	0	0	0	151,000	69,000	41,000
PS 17	2007			48,000	59,000		237,000			237,000
Upgrade Permanent Midtown PS	2012	386,000	317,000	0	69,500	159,900	976,000	310,000	141,000	1,152,000
Force Main to Midtown Ext	2012	1,029,000	846,000	0	600	1,100	7,000	1,320,000	602,000	251,000
PS 17 - 250 HP Motors	2017	452,000	305,000		82,000	138,000	604,000	413,000	188,000	721,000
36" NSVI Diversion to PS 17	2020	8,700,000	5,225,000	0	6,100	8,000	21,000	9,669,000	4,413,000	833,000
Totals			**9,001,000**				**2,106,000**		**6,041,000**	**5,066,000**

Assumptions:
- Base interest rate is 4.0%
- Construction cost escalation rate is 3.2%
- Electric energy escalation rate is 6.0%
- Annual O&M costs for interceptors are $0.20 per foot
- Annual O&M costs for small lift stations are 2.0% of initial cost + energy costs for pumping
- Annual O&M costs for PS 17 are based on historical data + energy costs for pumping
- O&M costs increase at the base interest rate

Future Sugar River Treatment Plant
Options 1 thru 4 with Low Growth

Option 1 with Low Growth Rate

		Construction Cost		O&M Costs				Salvage Value		Total 2007 Present Worth
Project	Year	Cost in Year Constructed	2007 Present Worth	2007	Intermediate Year	2027	2007 Present Worth	Year-20	2007 Present Worth	
LBMC Int	2007	3,407,000	3,407,000	4,300	5,100	9,500	86,000	4,691,000	2,141,000	1,352,000
PS 17	2007			49,000	57,000		199,000			199,000
PS 17 - 250 HP Motors	2011	374,000	320,000		61,000	154,000	1,002,000	289,000	132,000	1,190,000
30" NSVI Diversion to PS 17	2020	6,800,000	4,084,000	0	6,100	8,000	21,000	7,557,000	3,449,000	656,000
Totals			**7,811,000**				**1,308,000**		**5,722,000**	**3,397,000**

Option 2 with Low Growth Rate

		Construction Cost		O&M Costs				Salvage Value		Total 2007 Present Worth
Project	Year	Cost in Year Constructed	2007 Present Worth	2007	Intermediate Year	2027	2007 Present Worth	Year-20	2007 Present Worth	
Small LS	2007	192,000	192,000	5,800	19,000	0	109,000	0	0	301,000
FM to Shady Point LS	2007	55,000	55,000	0	0	0	0	0	0	55,000
PS 17	2007			48,000	76,000		487,000			487,000
LBMC Int	2020	5,131,000	3,082,000	0	7,200	9,500	30,000	5,800,000	2,647,000	465,000
PS 17 - 250 HP Motors	2017	452,000	305,000		82,000	154,000	642,000	413,000	188,000	759,000
30" NSVI Diversion to PS 17	2020	6,800,000	4,084,000	0	6,100	8,000	21,000	7,557,000	3,449,000	656,000
Totals			**7,718,000**				**1,289,000**		**6,284,000**	**2,723,000**

Option 3 with Low Growth Rate

		Construction Cost		O&M Costs				Salvage Value		Total 2007 Present Worth
Project	Year	Cost in Year Constructed	2007 Present Worth	2007	Intermediate Year	2027	2007 Present Worth	Year-20	2007 Present Worth	
Small LS	2007	192,000	192,000	5,800	19,000	0	109,000	0	0	301,000
FM to Shady Point LS	2007	110,000	110,000	0	0	0	0	0	0	110,000
PS 17	2007			48,000	76,000		487,000			487,000
Permanent Midtown PS	2020	3,642,000	2,187,000	0	92,500	130,900	404,000	3,481,000	1,589,000	1,002,000
Force Main to Midtown Ext	2020	1,324,000	795,000	0	900	1,100	4,000	1,497,000	683,000	116,000
PS 17 - 250 HP Motors	2017	452,000	305,000		82,000	138,000	604,000	413,000	188,000	721,000
36" NSVI Diversion to PS 17	2020	8,700,000	5,225,000	0	6,100	8,000	21,000	9,669,000	4,413,000	833,000
Totals			**8,814,000**				**1,629,000**		**6,873,000**	**3,570,000**

Option 4 with Low Growth Rate

		Construction Cost		O&M Costs				Salvage Value		Total 2007 Present Worth
Project	Year	Cost in Year Constructed	2007 Present Worth	2007	Intermediate Year	2027	2007 Present Worth	Year-20	2007 Present Worth	
Permanent Midtown PS	2007	2,198,000	2,198,000	50,600	93,600		680,000	1,376,000	628,000	2,250,000
FM to Shady Point LS	2007	110,000	110,000	0	0	0	0	151,000	69,000	41,000
PS 17	2007			48,000	76,000		487,000			487,000
Upgrade Permanent Midtown F	2020	497,000	298,000	0	92,500	130,900	404,000	475,000	217,000	485,000
Force Main to Midtown Ext	2020	1,324,000	795,000	0	900	1,100	4,000	1,497,000	683,000	116,000
PS 17 - 250 HP Motors	2017	452,000	305,000		82,000	138,000	604,000	413,000	188,000	721,000
36" NSVI Diversion to PS 17	2020	8,700,000	5,225,000	0	6,100	8,000	21,000	9,669,000	4,413,000	833,000
Totals			**8,931,000**				**2,200,000**		**6,198,000**	**4,933,000**

Assumptions: Base interest rate is 4.0%
Construction cost escalation rate is 3.2%
Electric energy escalation rate is 6.0%
Annual O&M costs for interceptors are $0.20 per foot
Annual O&M costs for small lift stations are 2.0% of initial cost + energy costs for pumping
Annual O&M costs for PS 17 are based on historical data + energy costs for pumping
O&M costs increase at the base interest rate

Future Sugar River Treatment Plant
Determination of Year when Option 1 equals Option 2

Option 1

Project	Year	Construction Cost		O&M Costs				Salvage Value		Total 2007 Present Worth
		Cost in Year Constructed	2007 Present Worth	2007	Intermediate Year	2027	2007 Present Worth	Year-20	2007 Present Worth	
LBMC Int	2007	3,407,000	3,407,000	4,300	5,100	9,500	86,000	4,691,000	2,141,000	1,352,000
Total										**1,352,000**

Year Shady Point Lift Station reaches capacity: **2011**

Option 2 with High Growth Rate

Project	Year	Construction Cost		O&M Costs				Salvage Value		Total 2007 Present Worth
		Cost in Year Constructed	2007 Present Worth	2007	Intermediate Year	2027	2007 Present Worth	Year-20	2007 Present Worth	
Small LS	2007	192,000	192,000	6,800	11,500	0	34,000	0	0	226,000
FM to Shady Point LS	2007	55,000	55,000	0	0	0	0	0	0	55,000
LBMC Int	2011	3,873,000	3,302,000	0	5,100	9,500	68,000	5,038,000	2,299,000	1,071,000
Total										**1,352,000**

Assumptions: Base interest rate is 4.0%
Construction cost escalation rate is 3.2%
Electric energy escalation rate is 6.0%
Annual O&M costs for interceptors are $0.20 per foot
Annual O&M costs for small lift stations are 2.0% of initial cost + energy costs for pumping
Annual O&M costs for PS 17 are based on historical data + energy costs for pumping
O&M costs increase at the base interest rate

Future Sugar River Treatment Plan
Option 1 versus Option 2 with Factor to Reflect Added Costs for Interceptor Construction Due to Devlopment Along the Route Over Time Equal to 0.75% for High Growth 5.74% for Low Growth

Option 1 with High Growth Rate

		Construction Cost		O&M Costs				Salvage Value		Total 2007 Present Worth
Project	Year	Cost in Year Constructed	2007 Present Worth	2007	Intermediate Year	2027	2007 Present Worth	Year-20	2007 Present Worth	
LBMC Int	2007	3,407,000	3,407,000	4,300	5,300	9,500	86,000	4,691,000	2,141,000	1,352,000
Totals			**3,407,000**				**86,000**		**2,141,000**	**1,352,000**

Option 2 with High Growth Rate

		Construction Cost		O&M Costs				Salvage Value		Total 2007 Present Worth
Project	Year	Cost in Year Constructed	2007 Present Worth	2007	Intermediate Year	2027	2007 Present Worth	Year-20	2007 Present Worth	
Small LS	2007	192,000	192,000	6,800	12,700	0	42,000	0	0	234,000
FM to Shady Point LS	2007	55,000	55,000	0	0	0	0	0	0	55,000
LBMC Int	2012	4,226,000	3,473,000	0	5,300	9,100	65,000	5,423,000	2,475,000	1,063,000
Totals			**3,720,000**				**107,000**		**2,475,000**	**1,352,000**

Option 1 with Low Growth Rate

		Construction Cost		O&M Costs				Salvage Value		Total 2007 Present Worth
Project	Year	Cost in Year Constructed	2007 Present Worth	2007	Intermediate Year	2027	2007 Present Worth	Year-20	2007 Present Worth	
LBMC Int	2007	3,407,000	3,407,000	4,300	7,000	8,800	86,000	4,691,000	2,141,000	1,352,000
Totals			**3,407,000**				**86,000**		**2,141,000**	**1,352,000**

Option 2 with Low Growth Rate

		Construction Cost		O&M Costs				Salvage Value		Total 2007 Present Worth
Project	Year	Cost in Year Constructed	2007 Present Worth	2007	Intermediate Year	2027	2007 Present Worth	Year-20	2007 Present Worth	
Small LS	2007	192,000	192,000	6,800	12,700	0	42,000	0	0	234,000
FM to Shady Point LS	2007	55,000	55,000	0	0	0	0	0	0	55,000
LBMC Int	2020	12,200,000	7,327,000	0	7,200	8,800	30,000	13,790,000	6,294,000	1,063,000
Totals			**7,574,000**				**72,000**		**6,294,000**	**1,352,000**

Assumptions: Base interest rate is 4.0%
Construction cost escalation rate is 3.2%
Electric energy escalation rate is 6.0%
Annual O&M costs for interceptors are $0.20 per foot
Annual O&M costs for small lift stations are 2.0% of initial cost + energy costs for pumping
Annual O&M costs for PS 17 are based on historical data + energy costs for pumping
O&M costs increase at the base interest rate

verona

CITY OF VERONA
Public Works
Parks & Recreation
410 Investment Court
Verona, Wisconsin 53593-8749

Fax: (608) 845-5761
Telephone: (608) 845-6695

April 7, 2006

RECEIVED
APR 11 2006
Madison Metropolitan
Sewerage District

Madison Metropolitan Sewerage District
Attn: Jon Schellpfeffer
1610 Moorland Rd.
Madison, WI 53713

Dear Mr. Schellpfeffer:

Enclosed is a copy of the Memorandum of Understanding: Lower Badger Mill Creek Interceptor, Phase 1 Pumping Station 17 to Edward Street.

If you have any questions or comments, you can contact me at 848-6801.

Sincerely,

RONALD R. RIEDER
Director, Public Works

RRR/pr
Enc.

Memorandum of Understanding:
Lower Badger Mill Creek Interceptor – Phase 1
Pumping Station 17 to Edwards Street

This memorandum of understanding describes a cooperative joint effort by the Madison Metropolitan Sewerage District (MMSD) and the City of Verona (the City) for the purpose of constructing the first section of the Lower Badger Mill Creek Interceptor from MMSD Pumping Station 17 to Edward Street in the City of Verona.

The City of Verona needs to replace its Westside Interceptor at this time to accommodate growth in the City. MMSD needs to construct an interceptor sewer in the Lower Badger Mill Creek valley to service existing and proposed development in the cities of Madison and Verona and the towns of Middleton and Verona. The route of the existing City Westside Interceptor and the proposed MMSD Lower Badger Mill Creek Interceptor are identical from MMSD Pumping Station 17 to Edward Street.

To realize the economies of scale provided by constructing a single sanitary sewer, rather than two separate sewers, the City and MMSD agreed to work together to ensure the Lower Badger Mill Creek Interceptor sewer has been designed with sufficient capacity to transport sewage from existing and future development in the City, from currently developed and undeveloped areas in MMSD, and from currently developed and undeveloped areas beyond the current MMSD boundaries in the Lower Badger Mill Creek valley.

Since the more immediate need for additional capacity is the result of growth in the City of Verona, the City contracted with Earth Tech for the design of this proposed sewer and has paid all of the design costs. The estimated construction cost of this sewer and the related work of reconnecting local City sewers to the new interceptor sewer is $2,392,660. Based on the design flows from the City of Verona that would be tributary to the Lower Badger Mill Creek Interceptor at Edward Street or downstream of Edward Street, the design flows for the balance of the interceptor, and the estimated cost of the various project components, the equitable split of design and construction costs for the Lower Badger Mill Creek Interceptor from Pumping Station 17 to Edward Street is thirty percent (30%) to the City and seventy percent (70%) to MMSD.

The design flow at Pumping Station 17 from the City is 3.019 cfs. The balance of the design flow at Pumping Station 17 is 5.925 cfs. The design flow at Manhole 17-102 from the City is 2.174 cfs, and the balance of the design flow at this manhole is 5.925 cfs. The design service area for the City of Verona to this sewer includes those lands in the Lower Badger Mill Creek valley and the Sugar River valley that are currently served by the City's Westside Interceptor and future service areas in the Lower Badger Mill Creek valley in sections 9, 10, 21, 27, and 28 of the Town of Verona (T6N, R8E) and in the Sugar River valley east of the Sugar River in sections 7, 8, 17, 21, and 28 of the Town of Verona (T6N, R8E).

This memorandum of understanding details the responsibilities of the City and MMSD, the allocation of design and construction costs, and the schedule of payments for the design and construction of this interceptor sewer. It also defines the interceptor connection charge rates to be applied to newly served areas tributary to this interceptor.

a) The City of Verona will complete the design and construction of the project.

b) MMSD will review and approve the plans for the project prior to construction.

c) The City will obtain all necessary easements necessary for the construction of this project. All permanent easements shall be granted to both the City and MMSD.

d) After completion of all design work, the City will provide an accounting of all design costs and invoice MMSD for their seventy percent (70%) share of the design costs. MMSD will pay for these design services within 30 days of receipt of the invoice.

e) During construction of the project, the City will provide the contractor's monthly payment request to MMSD for review and approval.

f) During construction of the project, the City will invoice MMSD monthly for their seventy percent (70%) share of the construction costs and related engineering costs, and MMSD will pay the City for these costs within 30 days of receipt of the invoice.

g) The cost of change orders to the contract shall be split between the City and MMSD based on the location of the change. Changes related to construction of the interceptor sewer shall be split seventy percent (70%) to MMSD and thirty percent (30%) to the City. Changes related to the City's sewers, including connections to the interceptor sewer, shall be paid in total by the City.

h) The City will make all payments to the contractor.

i) The City will furnish MMSD one full size set of as-built plans for the project and a CD with electronic files of the as-built plans in both AutoCad 2002 and Microstation V8.

j) Upon completion of construction and acceptance of the project by the City and MMSD, MMSD will assume ownership of the interceptor sewer and will provide all operation and maintenance activities for it, including repair and replacement as necessary to maintain service.

k) Newly served areas in the City of Verona that are tributary to this interceptor at or downstream of Edward Street and which were included in the City's design service area described above will be assessed an MMSD interceptor connection charge at the time sanitary sewerage service is provided based on the PS 17 Service Area rate.

l) In accordance with MMSD policy, MMSD will establish a new interceptor connection charge rate for the Lower Badger Mill Creek Interceptor, and all newly served areas, except those described in the previous paragraph, that are tributary to this interceptor will be assessed an MMSD interceptor connection charge at the time sanitary sewerage service is provided based on this new rate.

m) MMSD shall be responsible for injuries, claims and losses arising from or caused by the acts or omissions of its officers, employees, agencies, boards, commissions and representatives. The City shall be responsible for injuries, claims and losses arising from or caused by the acts or omissions of its officers, employees, agencies, boards, commissions and representatives. The obligations of the parties under this paragraph shall survive the expiration or termination of this agreement.

Signed:

For MMSD:	For City of Verona:
Jon W. Schellpfeffer	John Volker
Chief Engineer and Director	Mayor
Date: 3-1-06	Date: 3/27/06

AGREEMENT

THIS AGREEMENT, entered into on this 28 day of January, 2008, by and between the City of Madison, a Municipal Corporation in Dane County, Wisconsin, hereinafter referred to as the "City of Madison", and the Madison Metropolitan Sewerage District, a Municipal Corporation in Dane County, Wisconsin, hereinafter referred to as the "MMSD".

WITNESSETH:

WHEREAS, the MMSD provides regional sewerage service in central Dane County, Wisconsin, including sanitary sewer interceptor service, and

WHEREAS, the MMSD policy *Construction and Use of District Interceptors* states in part:

> "The District shall construct and operate an interceptor only when such interceptor will serve at least two municipalities, or when, in the judgment of the Commission, to a reasonable engineering probability, an interceptor will serve two municipalities in the reasonably foreseeable future", and

WHEREAS, the City of Madison intends to construct a sanitary sewer interceptor in the Lower Badger Mill Creek drainage basin north of Midtown Road to serve existing and future City of Madison Sewer Utility customers, and

WHEREAS, there are no current plans to provide sanitary sewerage service to the areas in the drainage basin north of Midtown Road outside of the current City of Madison boundary, and

WHEREAS, it would be wise and cost-effective to construct this interceptor with sufficient capacity to serve all areas in the drainage basin north of Midtown Road, including those areas beyond the current City of Madison boundary, and

WHEREAS, it is unlikely that this interceptor will serve more than City of Madison Sewer Utility customers in the foreseeable future, and

WHEREAS, areas in the Town of Middleton, currently served or planned to be served by private sewerage systems, may, at some time in the future, require municipal sanitary sewerage service, and

WHEREAS, such future municipal sanitary sewerage service in the Town of Middleton might be provided through the formation of a town sanitary district or town utility district, and

WHEREAS, such a town sanitary district or town utility district may be annexed to the MMSD,

NOW THEREFORE, the City of Madison and the Madison Metropolitan Sewerage District agree as follows:

1. The MMSD shall have the right to purchase this interceptor from the City of Madison at any time upon written request by MMSD, the purchase price being based upon the then present value of outstanding interceptor impact fees or special assessments.

2. After purchase, the MMSD shall have the right to recuperate the cost of the interceptor by imposing future interceptor connection charges for all lands, whether located in the City of Madison, Town of Middleton, or any other municipality, that are not served by the interceptor at the time of purchase.

3. After purchase, the MMSD will be responsible for all future operating, maintenance, and replacement costs of this interceptor.

IN WITNESS WHEREOF, the City of Madison and MMSD have executed this Agreement effective as of the date when all parties hereto have affixed their respective and duly authorized signatures.

CITY OF MADISON, DANE COUNTY, WISCONSIN

1-31-08

David J. Cieslewicz, Mayor Date

Maribeth Witzel-Behl 1-25-2008

~~Ray Fisher~~, City Clerk Date

Countersigned:

Randy Whitehead 1-28-08

for Dean Brasser, Comptroller Date

Approved as to Form:

28 January 2008

Michael P. May, City Attorney Date

MADISON METROPOLITAN SEWERAGE DISTRICT, DANE COUNTY, WISCONSIN

Edward V Schten 1-17-08

Edward V. Schten, President Date

P Mac Berthouex 1-17-08

P. Mac Berthouex, Secretary Date

Appendix A7

EPA Request for Information, April 2010

MADISON METROPOLITAN SEWERAGE DISTRICT

1610 Moorland Road
Madison, WI 53713-3398
Telephone (608) 222-1201
Fax (608) 222-2703

Jon W. Schellpfeffer
Chief Engineer & Director

Protecting Public Health and the Environment

December 3, 2009

Water Enforcement and Compliance Assurance Branch
U.S. Environmental Protection Agency, Region 5
77 West Jackson Blvd. (WC-15J)
Chicago, IL 60604

Attention: Duane Heaton

Subject: **Wet Weather/Sanitary Sewer System Information Request**
Docket No. V-W-10-308-01

Dear Mr. Heaton:

Enclosed please find the response of the Madison Metropolitan Sewerage District to the subject information request. If you require anything further, please contact me.

Sincerely,

Jon W. Schellpfeffer
Chief Engineer and Director

Encl. as stated

REQUEST FOR INFORMATION

1. Provide the name and address of the location(s) where records relative the operation and maintenance of the wastewater collection system are maintained.

Madison Metropolitan Sewerage District
1610 Moorland Road
Madison, WI 53713

2. Provide the name of the primary contact person responsible for wastewater and collection system maintenance. Also include telephone, fax and email contact information.

Paul Nehm
Director of Operations & Maintenance
Phone: 608-222-1201 (x252)
Fax: 608-222-2703
pauln@madsewer.org

3. Provide missing collection system data as indicated below:

A. Service Area 179.37 (in square miles)

B. Population Served 335,700 *(per Dane County Capital Area Regional Planning Commission's "MMSD Collection System Evaluation", January 2009. See attached.)*

C. System Inventory

Miles of gravity sewer	Miles of force main	Number of maintenance access structures	Number of pump stations	Number of siphons	Number of air, vacuum, or air/vacuum relief valves
95	44(1)	1,594	17	11	52

(1) The District operates and maintains 29 miles of force mains conveying raw wastewater and 15 miles of force mains conveying effluent from Nine Springs WWTP.

D.. Number of Service Connections(1)

Residential	89,422	Commercial	12,626
Industrial	293	Public Authorities	764
Other Users	44	Total	103,149

(1). *Data taken from public water supply annual reports as submitted to Wisconsin Public Service Commission (2008). See attached summary report.*

E. Indicate or describe a property owner's responsibility for maintenance and repair of lateral sewer lines (check one):

1. At main line connection only _____
2. From main line to property line or easement/cleanout _____
3. Beyond property line/cleanout _____
4. Other __X__ Explain.
The property owner is responsible for maintenance and repair of the sewer lateral from the District's sewer main to the building it serves. In general lateral connections are made to local sewers that are owned and maintained by the District's satellite communities. The District only permits direct connection to its interceptor sewers in special circumstances.

F. Is the collection system combined (storm and sanitary)?
Yes_____ No __X__
If yes, what percent of the collection system is a combined system? _____

G. Does the collection system have constructed relief points?
Yes__X__ No_____ If yes, describe the locations of the relief points in the system, indicate if they are in the separate sanitary portion of the system or in a combined area and identify the water body or location where these relief points discharge. *(Please see attached an edited version of Table 5 from the District's "Interceptor Maintenance Program Guidelines" for a listing of relief points).*

H. Average Annual Precipitation __34__ (in inches)

(Please see attached map from National Weather Service)

I. Provide the flow ratings/characteristics at the wastewater treatment plant:

Design Average Daily Flow (MGD)	Design Peak Dry Weather Flow (MGD)	Design Peak Wet Weather Flow (MGD)
50	N/A	140

Provide current actual flows or ratings experienced: *(See attached sheets)*

	Average Daily Flow (MGD)	Average Daily Water Consumption (MGD)
Residential	N/A	14.6
Commercial	N/A	14.6
Industrial	N/A	3.0
Other	N/A	4.4
Total[(1)]	47.3	36.6

(1). Totals are 2008 values. Year 2008 selected as basis to allow for comparison with water consumption data from Public Service Commission. Rainstorm events in 2008 significantly impacted lake levels and I/I and likely contributed to a higher than normal discrepancy between water consumption and wastewater flows.

	Per Capita Wastewater Flow (gpcd)	Per Capita Water Consumption Flow (MGD)
Maximum Month	141	N/A
Maximum Week	177	N/A
Maximum Day	226	N/A

J. Provide infrastructure age distribution estimates for the collection system

Age(1)	Gravity Sewer, miles	Force Mains, miles ~~or feet~~	Number of Pump Stations
0 - 25 years	21.28	20.88	2
26 - 50 years	60.81	14.71	11
51 - 75 years	6.91	8.16	4
> 76 years	6.08	0	0

(1). Note: This does not take into account rehabilitation and revitalization projects undertaken by the District.

K. Provide pipe size distribution estimates for the collection system

Diameter in inches	Gravity Sewer, miles	Force Mains, miles ~~or feet~~
8 inches or less	0.19	0.01
9 - 18 inches	18.29	3.58
19 - 36 inches	54.48	24.24
> 36 inches	22.11	15.92

4. Does the collection system receive flow from satellite communities?
Yes _X_ No_____ If yes, complete the following chart and answer questions A-C below:

Satellite Name (If additional room is needed, continue on last page.)	% of flow contribution
(See attached pages for complete list of community flows in 2008)	

A. Is flow measured where satellite flow enters the Madison Metropolitan Sewerage District sewer system?
Yes _X_ No_____

B. Does the Madison Metropolitan Sewerage District have the authority to surcharge satellites for excessive flows (e.g., for excessive I/I?)
Yes _X_ No_____

C. Has the Madison Metropolitan Sewerage District exercised its authority to surcharge satellites for excessive flows (e.g., for excessive I/I?)
Yes_____ No _X_

5. Do satellite communities enter into written agreements for wastewater services (contracts, charters, court orders, etc.)?

Yes_____ No__X__ If yes, answer A-C below: *(Please see attached correspondence from MMSD legal counsel to Wisconsin Department of Natural Resources for further details regarding this issue).*

A. Does the agreement extend the requirements of the sewer use ordinance (SUO) to the satellites?
Yes_____ No_____

B. Do the agreements have a date of termination and allow for renewal under different terms?
Yes_____ No_____

C. Does Madison Metropolitan Sewerage District maintain the legal authority to control the maximum flow introduced into the collection system from satellite communities?
Yes_____ No_____

6. Does the SUO clearly document standards, inspections, and approval for new connections?

Yes_X_ No_____ *(per Article IV of MMSD Sewer Use Ordinance)*

7. Does the SUO require satellite communities to adopt the same inspection and sampling schedules as required by the pretreatment ordinance?

Yes_____ No __X__

Inspection and sampling schedules are not specifically outlined in the District's Sewer Use Ordinance. Instead they are provided for in the District's "Pretreatment Program Procedures" (June 1991) and in the individual discharge permits that are issued for users requiring pretreatment. There is no specific requirement for satellite communities to adopt the same standards that the District imposes regarding inspection of facilities and sampling of flows. The Wisconsin Department of Natural Resources has reviewed and approved the District's "Pretreatment Program Procedures".

8. Does the SUO contain procedures for the following: inspection standards, pretreatment requirements, building/sewer permit issues, inflow prohibition?

Yes _X_ No_____

9. Does the SUO contain procedures and enforcement authority to control for the following:

A. Fats, oils, and grease (FOG)? Yes_X_ No_____
B. Inflow and infiltration? Yes_X_ No_____
C. Building structures over the sewer lines? Yes_____ No_X_
D. Storm water connections to sanitary lines? Yes_X_ No_____
E. Defects in service laterals located on private property? Yes_____ No _X_
F. Sump pumps, air conditioner discharge? Yes_X_ No_____

10. Describe the processes or procedures that are used to determine whether the capacity of existing gravity sewer system, pump stations and force mains are adequate for new connections. Address the following in the description:

A. Is metering of flow performed prior to allowing new connections?
B. Is a hydraulic model of the system used to predict the effects of new connections?
C. Is there any certification as to the adequacy of the sewer system to carry additional flow from new connections required?

District staff reviews each new or rehabilitated connection to the collection system that is proposed. A separate review is also conducted by the Wisconsin Department of Natural Resources and by the regional planning agency to assess conformance with adopted water quality plans. The applicant is required to state on the application form how much flow is anticipated from the new development and the adequacy of the downstream collection system to convey the additional flow.

In general the District uses flow evaluations of its collection system as a guide in determining capacity for new connections. The Dane County Area Regional Planning Commission recently completed a report entitled "MMSD Collection System Evaluation (2008)" for the District. This document assesses the existing capacity of each interceptor segment in the District's collection system and projects peak flows through the year 2060. The District is also updating its Collection System Facilities Plan at this time. This facility plan addresses both capacity and condition concerns for the District's collection system.

The District routinely performs flow monitoring for billing of its satellite communities, but this data is generally not needed or used for analysis of new connections. The District recently acquired a new hydraulic model of its collection system. The model has been used for analysis of new connections in special cases.

11. Describe the number of wastewater treatment plant effluent limitation exceedances, partial treatment bypass, or treatment upsets due to wet weather flow experienced in the last five (5) years. Provide the dates of those incidents.

There were no effluent limitation exceedances, partial treatment bypasses, or treatment upsets due to wet weather flow during the last five years. The only wet weather-related treatment plant incidents involve the routing of treated water to Nine Springs Creek, which is not one of the two permitted discharge points. On May 24, 2006, July 27, 2006, and June 13, 2008, the flow rate of water through the treatment plant was greater than *the capacity of the effluent pumps. The excess flows were discharged to Nine Springs Creek instead of being pumped to Badfish Creek or Badger Mill Creek. Volumes of these exceedances have been estimated for each of the events as follows:*

- *May 24, 2006: 39,000 gallons*
- *July 27, 2006: 381,000 gallons*
- *June 13, 2008: 14,600,000 gallons*

12. Describe any atypical local conditions that may increase the complexity or difficulty during design, construction, operation, and maintenance of the collection system. Provide a brief explanation of the local conditions and measures utilized to compensate for the condition such as:

A. Weather (ex. precipitation, temperature);
B. Terrain/Geology/Soils;
C. Groundwater or surface water body influence.

MMSD's collection system is unique due to the presence of four large lakes, hilly topography, and a densely populated isthmus in the City of Madison. Although a direct correlation between lake level and wastewater flows has not been clearly documented, flows to the treatment plant do tend to rise and fall with the levels of Lake Mendota and Lake Monona and the intensity of precipitation. As a result of the aforementioned local conditions, the District has an usually high number of pumping stations (17) in its collection system. The District's satellite communities also have a significant number of pumping stations, 44 of which the District maintains for its customers. The District's maintenance staff ensures that all of these stations are visited and inspected no less than once per week.

13. Has the system experienced corrosion problems in the last five (5) years? Describe the extent that corrosion in the collection system is a maintenance problem. Is there a corrosion control program in place? If so, what has been the preferred treatment or prevention program selected or implemented?

Deterioration of pipes due to corrosion has been a problem primarily for concrete interceptor sewers in MMSD's collection system. Most problems are observed above the water line in intercepting sewers due to attack of the pipe wall by sulfuric acid, a byproduct of hydrogen sulfide. Force mains constructed of concrete typically do not show any deterioration due to their fully submerged condition and generally appear in excellent condition when inspected.

Interceptor sewers, in general, are televised no less than once every ten years and any corrosion problems are noted during this time. The District's standard practice when detecting corrosion problems is to replace the corroded sections in their entirety or rehabilitate the concrete sewer with a new lining. Sliplining has also been used as a rehabilitation tool. In the past ten years the following concrete interceptors have been replaced or rehabilitated due to concerns with corrosion:

- *Northeast Interceptor: PS10 to Lien Road (2009-10) – 9,200 ft of 48", 54" & 63"*
- *Cottage Grove Extension to Far East Interceptor (2009-2010) – 5,500 ft of 18"*
- *Northeast Interceptor at Dane County Regional Airport (2006-08) – 7,300 ft of 48"*
- *West Interceptor Extension (2007) – 3,200 feet of 36" & 42"*
- *Northeast Interceptor: Buckeye Road to Femrite Drive (2005) – 7,200 feet of 36", 42", 48" & 54"*
- *Nine Springs Valley Interceptor (2001) – 1,100 feet of 30"*

As standard practice the District no longer uses concrete pipe for interceptor sewers. Fiberglass, PVC, and epoxy-lined ductile iron are commonly chosen materials for new sewers.

14. Is there an odor control program in place? Describe the extent of any odor control issues and identify the locations where odor complaints originate.

Odor complaints have been received in the vicinity of the following District pumping stations;

Pump Station 2 - 833 West Washington Avenue, Madison
Pump Station 7 - 6300 Metropolitan Lane, Monona
Pump Station 10 – 110 Regas Road, Madison
Pump Station 13 – 3634 Amelia Earhart Drive, Madison
Pump Station 16 – 1301 Gammon Road, Middleton

At Pump Stations 2 and 16 odors have been reported by neighboring residents. A restaurant next door to Pump Station 7, a US Post Office next to Pump Station 10, and a restaurant next to Pump Station 13 have reported odors from these stations. At Pump Stations 2 and 10 the ventilation has been modified to reduce or eliminate the odors in surrounding homes and businesses. At Pump Stations 7, 10, 13, and 16 a chemical addition system has been installed in the ventilation systems.

15. Is there a grease control program in place? Describe the program and the extent that grease blockages in the collection system are a problem, and identify the locations where chronic grease blockages occur. What tools are used to address grease problems, e.g., grease trap ordinances and inspections, physical removal of grease, or chemical additions to dislodge or dissolve grease?

Blockages in the District's collection system due to grease are rare. See the 2005 incident reported in Question 17. The District has eleven siphons. Nine of them are cleaned twice per year to avoid problems that could be caused by grease or other debris. The wet wells at many of the pumping stations are cleaned several times each year to remove grease and debris. The District's Sewer Use Ordinance contains limitations on grease of petroleum origin and grease of animal or vegetable origin.

16. Is there a root control program in place? Describe the program and the extent that root blockages in the collection system are a problem and identify the locations where chronic root blockages occur.
Roots in the collection system have not been a problem. If roots are found during the television of an interceptor they are cut out. Roots had been prevalent in a section of the Southwest Interceptor near 5 Boston Court in the City of Madison. For a number of years the roots were removed annually as a preventive measure. That section of interceptor has been lined and roots are no longer a problem.

17. Provide a description of Sanitary Sewer Overflows (SSOs), discharges, releases or bypasses (e.g., whenever the sewage left its piping system) that have occurred in the collection system within the last five (5) years. Include the following information for each incident:

- Date
- Location
- Estimated volume of the SSO, discharges, release or bypass
- Cause of the SSO, discharge, release, or bypass
- Disposition of the SSO, discharge, release, or bypass (did the release reach a waterway, flow to storm sewer, paved areas, basements, etc.)
- Actions taken to mitigate the SSO, discharges, release, or bypass

The following sanitary sewer overflows have occurred in the last five years per the District's Annual Reports to the Wisconsin Department of Natural Resources:

<u>October 7, 2005</u> – *The District's Rimrock Interceptor was plugged by grease and other debris. Approximately 40,500 gallons of wastewater was discharged to the surrounding wetland area. For a period of time a grease dissolving enzyme was added to a small pumping station. A review was made to determine if there were any unusual sources of grease in the service area of the interceptor. No sources were found and the problem has not reoccurred.*

<u>January 20, 2006</u> – *Debris became stuck in the air release valve in the manhole at station 111+81 on the Pump Station 2 force main. About 100 gallons of water exited the manhole and soaked into the surrounding ground. The valve was taken apart to determine if anything other than debris had caused the failure. Nothing was found.*

<u>July 14, 2006</u> – *Debris became stuck in the air release valve on the District's Cross Town force main near the intersection of Wilson Street and Few Street in Madison. Much of the water released was recovered from a pit type of catch basin. About 300-400 gallons of water was not captured and flowed to a storm sewer. Checking of the air release valves is on a maintenance schedule. This schedule was reviewed and the frequency of checking specific valves was increased.*

<u>August 6, 2007</u> - *A contractor who was clearing trees on a District project damaged the District's West Interceptor in Lakeview Park in Middleton near Allen Boulevard. This allowed water to flow out of the interceptor and into a stormwater drainage way. The*

water eventually flowed to Lake Mendota. When the leak was discovered, several vactor trucks were used to remove water upstream of the leak. Portable pumps were also used to pump water around the damaged area. It was estimated that about 200,000 gallons was released from the system. The pipe was repaired.

<u>August 24, 2007</u> - While checking the air release valve in the manhole at station 111+81 on the Pump Station 2 force main our crew found that a small amount of water was running out of the air release valve. This was during a wet weather event and stormwater had run into the manhole and filled it. The rate that water was flowing out of the valve was similar to what would flow from a garden hose. Because of the amount of stormwater in the area, it was not possible to estimate how much wastewater had flowed onto the ground. Debris was removed from the air release valve.

<u>June 8, 2008</u> – Because of several extreme rainfall events during this week several bypasses were made in the collection system to prevent basement back-ups. See the attached June 14, 2008 letter to Larry Benson of the Wisconsin Department of Natural Resources for details. The District took a number of measures in response to this event. Please see Jon Schellpfeffer's September 24, 2008 letter to Larry Benson for details.

<u>June 30, 2008</u> – A late evening automobile accident caused the failure of both electric power feeds to the District's Pump Station 7 at 6300 Metropolitan Lane in Monona. As soon as power was lost at the station, all of the upstream pumping stations that pump wastewater toward Pump Station 7 were shut off. A generator was taken to the pump station, but before power could be restored wastewater was released from manholes along Winnequah Road in Monona. We estimated that 250,000 to 275,000 gallons of wastewater was discharged. Most of this water probably entered Lake Monona. After the event a permanent generator receptacle was installed at the station. In addition, the power utility has provided a third feed to the station. This feed enters the station area from a different direction than the other two feeds.

<u>September 14, 2008</u> – The control system at the District's Pump Station 3 was not operational due to a failed programmable logic controller (PLC). This prevented the station pumps from operating. The station is located at the northwest side of the treatment plant grounds. To minimize any overflow, the pumps at the station were turned on before efforts were made to determine if an overflow had occurred. The area west and north of the pump station is a marsh area with a drainage way running through it. A small overflow was located at a manhole north of the marsh. Based on pumping records an overflow of 110,000 gallons was calculated. Visual observations led us to believe that the overflow was less than this. As soon as the problem at the station was known, a pump was set up to pump water from the drainage way to the pumping station in an attempt to capture water which may have been discharged from the collection system. The station's PLC was replaced and alarms have been added to the treatment plant's SCADA system to warn of a similar problem if it would occur in the future.

<u>January 21, 2009</u> – A contractor who was making soil core borings for a non-District project bored a hole in the District's 36 inch diameter reinforced concrete force main

from Pump Station 6. This occurred on Monona Drive near its intersection with Buckeye Road in Monona. Water leaked out of the pipe as soon as the hole was made. This water entered Lake Monona. The station was shut off, and the District's trucks normally used to haul biosolids were used to haul wastewater from the Pump Station 6 wet well to other areas of the collection system. A temporary plug was placed in the hole to allow the station to be returned to service until repair parts could be obtained. During the final repair the force main was drained back to the station and vactor trucks were used to collect any wastewater that drained from the pipe when it was cut to make the repair. We estimate that about 248,000 gallons of water was discharged during this event. The District has used and continues to use an underground utilities location service to respond to "Digger's Hotline" requests for underground utility locations.

<u>May 8, 2009</u> - A significant amount of debris became stuck in the air release valve in the manhole at station 111+81 on the Pump Station 2 force main. Pumps had to be used to pump water out of the manhole to allow an employee to enter it to close the manual shut-off valve on the line. It was estimated that about 370,000 gallons of water was discharged to the surrounding area. A portion of this water probably entered Lake Monona. After review of a surge analysis on this force main, it was decided to keep the valve to the air release valve closed. The shut off valve is opened manually periodically to check for trapped air. This eliminated the potential for this valve to be stuck open in the future.

<u>September 9, 2009</u> – Wastewater was released from the air release valve manhole on the District's Cross Town force main between Patterson Street and Brearly Street in Madison. A District repair crew had removed the air release valve the previous day for repair. They had closed the isolation valve, but it evidently was not closed completely. It is estimated that the amount of water released was small and most was recovered by a vactor truck. Procedures were implemented to require a blind flange to be bolted over isolation valves whenever an air release valve is removed.

18. Of the SSOs, discharges, releases or bypasses to waterways that are identified in response to the proceeding question, how many were to surface waters that could affect:

A. Primary contact recreation (swimming, bathing, waterskiing, etc.)? 7
(Note: One of these incidents occurred in the winter).

B. Drinking water sources? 0

19. Approximately how many of SSOs, discharges, releases, or bypasses were from:

A. Manholes? 7
B. Pump stations? 2
C. Main and trunk sewers? 1
D. Lateral and branch sewers? 0
E. Structural bypasses? 0
F. Force main break? 1

20. Approximately how many of the SSOs, discharges, releases, or bypasses were caused by the following:

A. Debris buildup? __5__
B. Collapsed pipe? __1__
C. Root intrusion? __0__
D. Capacity limitations? __0__ -
E. Excessive infiltration and inflow? ___1____
F. Fat/Oil/Grease? __1__
G. Vandalism/*utility excavation by others*? ___1_____
H. Power Interruption and/or Lack of Backup Power Source? ___1___
I. Mechanical or Electronic Failure? ___1___
J. Pump failure and/or Lack of Backup (or Duplex) Pumps? ___0___

21. What equipment is available for responding to SSOs, discharges, releases, or bypasses?

A. Two 6" portable pumps and hoses
B. Two 4" portable pumps and hoses
C. Three 2" portable pumps and hoses
D. One 1 ½" portable pump and hoses
E. Fourteen 5000 to 6000 gallon trailers to be pulled by semi tractors
F. Piping sections of various sizes
G. Repair clamps
H. Three portable generators
I. Access to City of Madison vactor trucks
J. Barricades
K. Bobcat loader and trailer
L. Endloader
M. Three dump trucks and access to contractor trucks
N. Hydraulic underground valve turner
O. Forklift
P. Manhole sections
Q. Manhole covers
R. Various power and hand tools
S. Stoplogs
T. Confined space entry equipment
U. Five pick-up trucks with mechanical tools
V. Five vans and trucks with electrical tools
W. Three sewer maintenance vehicles with tools

22. Has Madison Metropolitan Sewerage District developed and adopted written procedures or instructions for the following:

A. SSO, bypass and containment? Yes_____ No __X__
B. Reporting all SSOs to the state regardless of size? Yes____ No _X_
C. Containment or cleanup to mitigate effect of SSOs? Yes_____ No___X___
D. Problem evaluation and solution? Yes_____ No _X_

The District has not formally adopted written procedures for the issues referenced above. However, the District has prepared and updates on a regular basis its "Emergency

Response Manual". This document includes procedures for dealing with a variety of issues involving emergency response, including force main breaks and power interruptions at pumping stations. It also includes contact information for public health officials, municipal customers, regulators, and other emergency contacts.

All employees involved in SSO incidents understand the importance of containment of SSOs and bypasses. This is demonstrated in the incidents listed in question 17. Measures are taken to contain discharges whenever possible.

All SSOs are reported to the District's Area Engineer (Larry Benson) at the Wisconsin Department of Natural Resources. Copies of the reports are stored in the District's filing system and are further reported on the annual Compliance Maintenance Annual Report (CMAR).

23. Are locations where multiple SSOs, discharges, releases, or bypasses have occurred posted with warning signs?

No. The District is not aware of any points in the collection system with multiple and/or recurring SSO discharges that would necessitate the posting of warnings.

24. Are there areas that experience sanitary sewer limitations resulting in basement backups or street flooding?

Yes_____ No __X__ If yes, describe these areas.

25. Provide the following information related to SSOs, discharges, release or bypasses that occurred during dry weather:

A. Number of stoppages annually.
B. Average time to clear a stoppage.
C. Number of stoppages resulting in overflows and/or backups annually.
D. Total quantity of overflows or releases.

The District has had only one reportable dry weather event in the last five years. That was the 2005 plugging of the Rimrock Interceptor (see Question 17 for details). In that incident it took about 3.5 hours to unplug the line. A release of about 40,500 gallons occurred.

26. Provide the following information related to pump and lift station design:

A. Total number of stations in the collection system. __17__
B. Number of pump stations with pump capacity redundancy. __15__

All of the District's pump stations have two or more pumps to incorporate redundancy. It is assumed that this question is referring to the provision of firm capacity. Two pumping stations (PS 4 & PS 7) have firm pumping capacities that are slightly less than benchmark peak flowrates. Firm capacity improvements at both stations are scheduled between 2012 and 2020. Please see attached Table 2.2 from the District's 'Collection

System Facilities Plan (2002)', which is currently being updated, for further details on the firm capacities at all of the District's pumping stations.

C. Number of pump stations with backup power sources. 16

Note: MMSD's Pump Station 3 requires a portable generator as its backup power source. This pumping station is located on the grounds of the treatment plant where the portable generator is stored.

D. Number of pump stations with dry weather capacity limitations. 0

E. Number of pump stations with wet weather capacity limitations. 0

F. Number of pump or lift station failures resulting in overflows/releases or backups in the last five (5) years. 2

G. Total quantity of SSOs, discharges, releases, or bypasses expressed in gallons or million gallons (MG). 0.38 MG *(See Question 17)*

H. For each SSO, discharge, release or bypass, is failure mode and effect diagnosed?
Yes X No____

I. Are future preventive measures initiated based on diagnosis?
Yes X No____

27. Provide the number of miles or feet of force main monitored annually (visual surface inspection of alignment). **See below**

MMSD does not have a formal program for surface inspection of its force mains. Internal inspections of force mains are not possible. They must remain in-service at all times and are impossible to bypass. Further, they are pressurized at all times, prohibiting access. MMSD's Sewer Maintenance crew does perform inspection and maintenance of air release valves on its force mains on a semiannual basis, although the entire length of the force main is generally not inspected.

28. Provide the miles or feet of force main monitored annually (pressure profile, capacity). 6.3 miles/year (44 miles total every 7 years)

MMSD staff is currently preparing an update to its 'Collection System Facilities Plan (2002).' In this document is a tabular listing of all force main segments with corresponding capacities and characteristics, including any pressure limitations. Typically MMSD force mains have plenty of capacity and the amount of flow does not change drastically from year-to-year. Table 2.3 of the District's 'Collection System Facilities Plan' is attached for further reference.

Several MMSD force mains are protected with surge relief valves or surge tanks located at the pumping stations.

29. Provide the number of force main failures in the last five (5) years. 6

September, 2009	*Crosstown FM Air Release Valve*
May, 2009	*PS2FM Air Release Valve*
January, 2009	*PS6FM Break by Soil-Boring Machine*
August, 2007	*PS2FM Air Release Valve*
July, 2006	*Crosstown FM Air Release Valve*
January, 2006	*PS2FM Air Release Valve*

30. Provide a description of the cause(s) of each force main failure in the last five (5) years.

Of the six failures, five have been directly related to failure of air release valves on District force mains. The District's most commonly used air release valve employs the use of a float that rises and falls in the valve in response to fluctuations in system pressure. Hard plastics and grease in the raw wastewater have been found to impede the free movement of the float assembly and/or wedge the float in the "open" position. MMSD maintenance staff has increased the frequency for cleaning of these valves in the collection system. Further, MMSD has isolated some air release valves from service in locations where they are not deemed necessary to control surges and to provide air release or vacuum relief.

With regard to the other force main failure, it was due to a broken pipe caused by the drilling of a soil boring contractor. Please see Question 17 for additional information regarding this failure.

31. Provide the following information related to service to wastewater system users and customers:

A. Average annual user charge rate for residential user. __________
In 2008 the District's service charge for the average residential user was about $10 per month. The additional charge from the satellite community for the average residential user was about $7 per month. This results in an average annual charge of $202. (from Typical Charges for City of Madison Residential Customer.xls and 2008 Service Charge Rates.xls)

B. Is the residential rate based on water consumption or a flat rate? Describe.
The residential service charge is based on a combination of a volume charge based on consumption and a flat rate based on the capacity of the water meter.

C. Number of user complaints annually for the last five (5) years.
From 2004 through 2008 the District logged an average of 10 complaints per year. Odor, vehicle operation, and biosolids reuse program complaints averaged 2 per year each. Noise from the treatment plant averaged one complaint per year. (MMSD complaint database - summarized)

D. Number of complaints that are Madison Metropolitan Sewerage District responsibility annually for the last five (5) years.
About 9 out of 10 complaints received each year were the District's responsibility. (MMSD complaint database)

E. Number of wastewater public health or other warnings issued by Madison Metropolitan Sewerage District annually for the last five (5) years.
The District works with the City of Madison/Dane County Public Health Department to issue warnings. The District does not issue warnings directly. In June, 2008, during a series of heavy rain storms that resulted in flooding and wastewater by-passes, the District and the health department worked closely over about a one week period to inform the public of beaches that had been closed on the Madison lakes due to sewage overflows. Also in June of 2008, the District notified the health department of an overflow caused by the loss of both electric power feeds to one of the District's pump stations that resulted in the overflow of sewage to Lake Monona and the Yahara River. This amounts to about one warning per year, even though all warnings over the last five years were issued in one month; June, 2008. (personal recollection of Jon Schellpfeffer)

F. Number of claims for damages due to backups annually for the last five (5) years.
Four claims for damage have been received and/or acted on in the last five years. One of these was from the City of Madison Parks Department for suspected damage to wetland plants due to the by-pass of wastewater in June, 2008. Two claims were submitted for basement back-ups caused by a pump station failure. The final claim was submitted for a back-up into a commercial building from a District interceptor sewer.

G. Provide the total cost of claims against Madison Metropolitan Sewerage District related to wastewater management settled annually for the last five (5) years. *(Information taken from District's accounting software)*

1. *2009: $444.00 to clean a residential basement when there was a backup at the Bible Camp pump station.**

2. *2008: No claims paid to date due to wetlands damage in City of Madison.*

3. *2007: $112.78 to clean a residential basement when there was a backup at the Bible Camp pump station.**

4. *2005: $906.57 to clean a commercial building when there was a back-up on the north leg of the Southwest Interceptor*

* *Note: Bible Camp pump station is maintained by MMSD and owned by a satellite community.*

Total cost is $1463.35.

H. When was the last wastewater-related rate increase? January 1, 2009

32. Provide the following information related to financial management of wastewater service: *(information based on 2008 Financial Report and 2009 budget and rates)*

A. Total annual revenue received from wastewater user charges:
In 2008 the District received $20,776,193 in service charge revenue.
  1. Percent of revenue used for long-term debt; *34 percent.*
  2. Percent of revenue used for treatment and disposal; *45 percent.*
  3. Percent of revenue used for collection and conveyance. *12 percent.*

B. Other sources of revenue (i.e., property tax, tap-in fees, etc.):
For servicing pump stations owned by certain satellite communities, $393,313; from septage disposal, $262,903; from pretreatment fees, $20,378; from investment income, $663,046; from rent, $59,336; from connection charges, $496,515; and miscellaneous revenues of $60,007, for a total of $1,955,498.
  1. Percent of revenue used for long-term debt; *25 percent.*
  2. Percent of revenue used for treatment and disposal; *34 percent.*
  3. Percent of revenue used for collection and conveyance. *9 percent.*

C. Provide the annual operation and maintenance budget: *(for 2009)*
  1. For the wastewater treatment plant; *$9,565,235.*
  2. For wastewater collection and conveyance system. *$2,097,242.*

D. Provide the annual costs of operation and maintenance for:
  1. The wastewater treatment plant for the most recently completed service year; *For 2008, $9,401,294.*
  2. The wastewater collection and conveyance system for the most recently completed service year. *For 2008, $2,431,166.*

E. Does Madison Metropolitan Sewerage District have a long-range wastewater Capacity Improvement Project (CIP) plan for system expansion, rehabilitation, and replacement?
Yes _X_ No____

33. Have infiltration/inflow (I/I) assessments been done to determine the extent of these components as part of the system's wastewater flow? Has it been proven that it is cost effective to eliminate I/I rather than continue to treat it? Has a sewer system evaluation survey (SSES) been performed on system? If yes, when? Have rehabilitation projects been prioritized for correcting I/I problems? If so, how far has the I/I elimination program progressed?

The District completed an extensive sewer system evaluation survey (SSES) in February of 1978. This evaluation resulted in the discovery and documentation of a significant *amount of I/I in the collection systems of the District and those of its satellite communities. This evaluation also led to the formalization of important inspection and maintenance programs for District sewers.*

The District periodically assesses the amount of inflow and infiltration in its collection system in response to high flow storm events and requests that satellite communities do the same. Internal studies have been performed periodically to assess if it is cost effective to eliminate I/I rather than convey it to the plant for treatment. In general it has been determined that it is more cost effective to eliminate I/I in remote parts of the collection system (due to pumping costs) and cheaper to treat it when it is located closer to the treatment plant.

The following I/I initiatives have been undertaken directly by the District or the City of Madison within the last fifteen years:

- *Memorandum entitled 'High Flow Event of June 17, 1996', prepared by MMSD staff. This internal study and memo analyzed the impact of a significant rainfall event on the District's collection system in June of 1996 and provided recommendations for future follow-up from the District and its satellite communities.*

- *Report on 'Hoard/Kedzie Street' (1997), prepared by Strand Associates, Inc. for City of Madison at request of MMSD. This I/I study investigated chronic problems in a sewer basin tributary to MMSD Pump Station 1.*

- *'Sanitary Sewerage Conveyance System Study, Baldwin Street & Elizabeth Street Area' (1998) for City of Madison at request of MMSD. This I/I study investigated chronic problems in a sewer basin tributary to MMSD Pump Station 1.*

- *Report on 'Stormwater Inflow Monitoring' (1999), prepared by Strand Associates, Inc. for MMSD. This study employed the use of extensive flow monitoring in three different pump station basins in an attempt to quantify the amount of I/I in the District's collection system.*

- *'Collection System Facilities Plan' (2002), prepared by MMSD. This facility plan proposed that I/I studies be undertaken in the following pump station service areas: 9, 10, 12, 13 & 14. Studies have been completed for Pump Stations 9, 12, and 13.*

- *'Truax Area Sewer Study' (2005), prepared by Brown and Caldwell, for City of Madison at request of MMSD. This study employed the use of flow metering, smoke testing, manhole inspections, and hydraulic modeling to investigate chronic I/I problems upstream of MMSD Pump Station 13.*

- *The District submitted a written request to 15 of its satellite communities in August of 2009 requesting that I/I investigations of their collection systems be undertaken (see attached letter).*

- *The District has acquired a dynamic hydraulic model of its collection system. One intended use for this model is to identify areas of the collection system with excessive I/I and evaluate reduction strategies within those areas.*

34. Does the Madison Metropolitan Sewerage District operate an industrial pretreatment program? Yes _X_ No_____

35. Does the Madison Metropolitan Sewerage District or the municipalities within the WWTP service area have a private source inflow and infiltration reduction program?
Yes _____ No _X_ If yes, describe this program.

36. Does the Madison Metropolitan Sewerage District use:
  - A. Internal T.V. inspection for evaluating the condition of the collection system? Yes _X_ No_____
  - B. Smoke testing? Yes_X_ No_____

37. Does the Sanitary Sewer System experience chronic O & M problems that are the result of design issues in the system? If yes, provide brief explanation.

No, the District is not aware of any design deficiencies that result in increased O&M problems.

38. Does the Sanitary Sewer System experience chronic O & M problems that are the result of construction issues in the system? If yes, provide brief explanation.

No.

CERTIFICATION

I certify under penalty of law that this response and all attachments were prepared under my direction or supervision in accordance with a system designed to assure that qualified personnel properly gather and evaluate the information submitted. Based on my inquiry of the person(s) who manage the system, or those person(s) directly responsible for gathering the information, the information submitted is, to the best of my knowledge and belief, true, accurate and complete. I am aware that there are significant penalties for submitting false information, including the possibility of fine and imprisonment for knowing violations.

Jon W. Schellpfeffer
Chief Engineer & Director, MMSD

QUESTION #3B

Table 2-1: Population Trends and Forecasts for the MMSD

	1980	1990	2000	2009	2030	2060
Central USA	218,344	245,390	268,850		339,222	*404,204*
Cottage Grove USA	901	1,131	4,059		9,372	*11,798*
Dane USA			799		1,351	*1,594*
Fox Bluff LSA			240		240	*240*
Kegonsa LSA			2,228		2,252	*2,252*
Morrisonville USA			352		428	*464*
Northern USA	5,393	7,160	9,901		16,883	*23,825*
Verona USA			7,306		15,685	*20,178*
Waubesa LSA			2,027		2,027	*2,027*
Waunakee USA	3,890	5,899	9,000		17,458	*23,367*
Windsor Prairie LSA			509		509	*509*
Westport LSA			377		377	*377*
MMSD	228,528	259,580	305,648	335,700	405,804	*490,835*

Historic and forecasted population figures for three urban service areas that are outside, but nearby, the current MMSD service area are shown in Table 2-2.

Table 2-2: Population Trends and Forecasts for Other USAs

	1980	1990	2000	2030	2060
Oregon USA	3,927	4,528	7,514	13,106	*17,275*
Stoughton USA	8,256	9,265	12,671	18,609	*23,064*
Sun Prairie USA	13,306	15,481	20,533	36,211	*45,188*

Traffic Analysis Zone Data

In addition to population forecasts at the urban service area level, socioeconomic data is available in smaller analysis units called traffic analysis zones (TAZ). The Madison Area Transportation Planning Board (MATPB) developed the most recent TAZ data in 2000 for transportation planning. This data divides Dane County into over 1,000 analysis zones, which range in size from 3.7 acres in the central urban area, to over 6,000 acres in rural areas. The socioeconomic data associated with each zone includes population, number of households, and total employment for the year 2000 as well as forecasts for the year 2030.

TAZ Data Sources

The TAZ allocation of year 2000 population and household data is based on US Census data and Census block boundaries. The MATPB developed the TAZ 2030 population and household data by allocating the DOA/CARPC population forecasts to TAZ regions based on community comprehensive plans and neighborhood development plans. They noted in their *Regional Transportation Plan 2030*, that the allocation of forecasted 2030 growth is far less than a build-out scenario of the planned growth identified in local plans.

QUESTION #3D

Question #3D

COMMUNITY WATER METERS - 2008

Municipality	Classification of Meters[1]						
	Residential	Commercial	Industrial	Public Authority	Wholesale, Interdepartmental or Utility Use	In Stock and Deduct Meters	TOTAL LESS DEDUCT AND IN STOCK METERS
Cottage Grove	2,086	112	10	21	0	59	2,229
Dane	320	31	3	6	0	42	360
DeForest	2,840	255	35	21	0	139	3,151
Fitchburg	5,244	742	39	13	0	0	6,038
Madison	56,033	8,783	53	481	12	663	65,362
Maple Bluff	549	6	0	6	0	41	561
McFarland	2,586	271	0	29	6	128	2,892
Middleton	4,821	864	48	55	10	73	5,798
Monona	2,512	309	0	24	0	114	2,845
Morrisonville	148	5	0	3	0	23	156
Shorewood	591	28	0	7	0	49	626
Verona	3,549	362	87	43	0	106	4,041
Waunakee	3,703	272	1	24	13	30	4,013
Westport	324	33	0	1	0	10	358
Windsor #1	822	88	6	2	1	78	919
TOTAL METERS WITH PUBLIC WATER SUPPLY	**86,128**	**12,161**	**282**	**736**	**42**	**1,555**	**99,349**
CORRECTION FOR PRIVATE WATER SUPPLIES[2]	**1.04**	**1.04**	**1.04**	**1.04**	**1.04**		
TOTAL CONNECTIONS	**89,422**	**12,626**	**293**	**764**	**44**		**103,149**
% OF TOTAL	**86.7%**	**12.2%**	**0.3%**	**0.7%**	**0.0%**	**0.0%**	

(1). Information taken from Municipality Annual Reports provided to Public Service Commission of Wisconsin.

(2). MMSD customer records indicate approximately 3,800 customers are served by private water supplies.

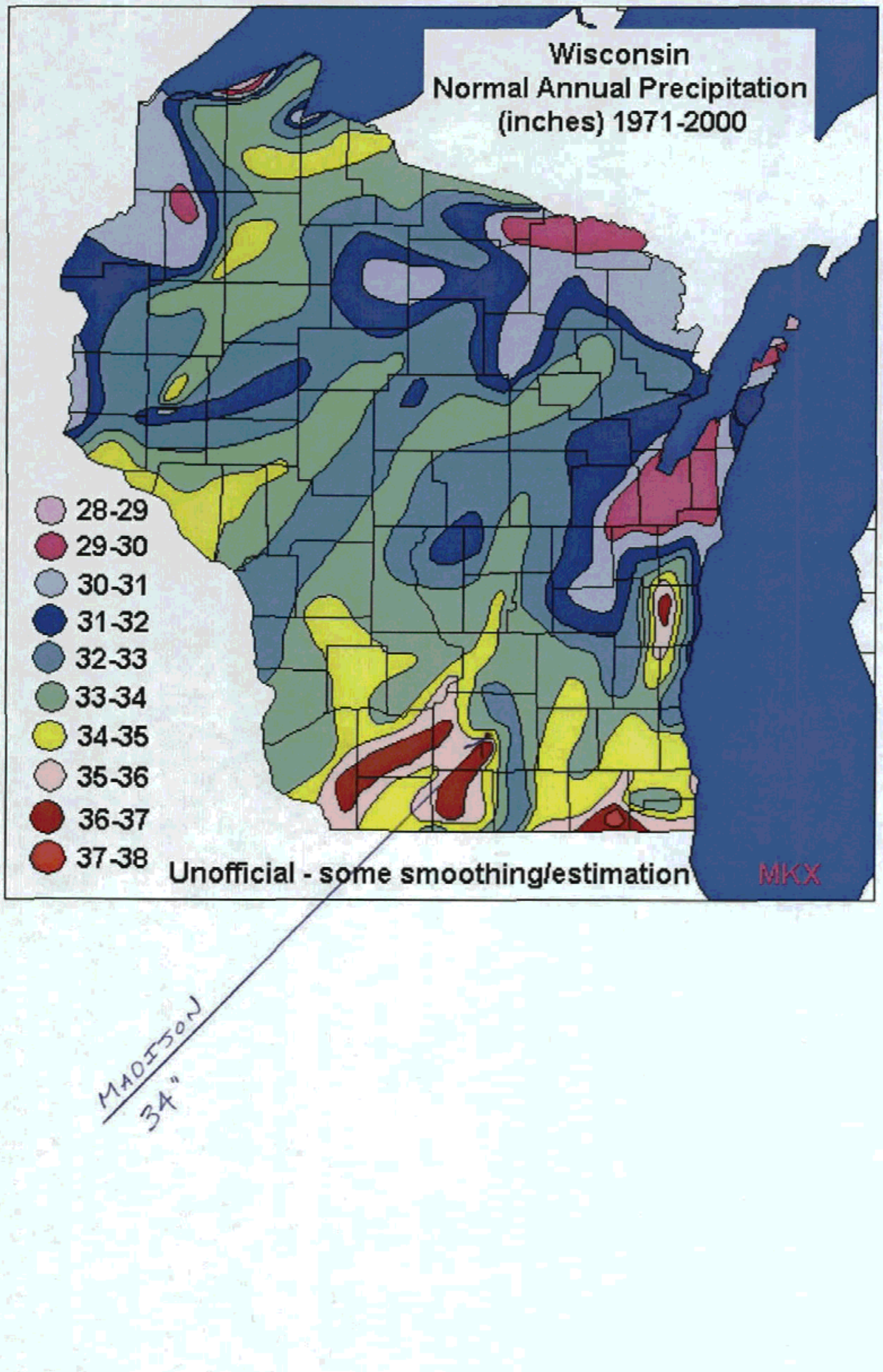
11/25/2009

QUESTION 3G

MMSD RELIEF POINTS				
	Facility	**MH**	**Location & Comments**	**Downstream Watercourse**
1	Bedford Street Stoplogs.	CT-3420	Northshore Drive at end of Bedford Street, adjacent to Monona Bay.	Monona Bay
2	PS5 Stoplog	5-403	Mendota Drive across from PS5	Lake Mendota
3	PS6 Flapgate	6-102	Drainage ditch near PS6	Lake Monona
4	PS7 Stoplog	PS7	Entrance chamber behind PS7	Yahara River
5	PS8 Stoplog at Wingra Creek	8-100	North side of Wingra Creek across from PS8	Wingra Creek
6	NEI Flapgate upstream of PS10	10-114	At Starkweather Creek, south of Sycamore Ave and west of Walsh Rd. To be removed in 2009 during NEI-PS10 to Lien Road Project.	Starkweather Creek
7	PS11 Flapgate	PS11	PS11 near entrance chamber	Upper Mud Lake
8	NEI Truax Ext Flapgate upstream of PS13	13-105	Along drainage ditch, west of Hwy 51 at Dane County Airport access road. Inside airport perimeter fence.	Starkweather Creek

NOTE: All relief points constructed in separate sanitary sewerage system.

QUESTION #34



* Map provided by National Weather Service.

QUESTION #3I

COMMUNITY WATER SALES - 2008

Thousands of Gallons of Water Sold

Municipalities with Public Water Supplies	Unmetered Sales: Residential	Unmetered Sales: Commercial	Unmetered Sales: Industrial	Unmetered Sales: Public Authority	Metered Sales: Residential	Metered Sales: Commercial	Metered Sales: Industrial	Metered Sales: Public Authority	Private Fire Protection	Public Fire Protection	Other	Resale	Inter-departmental	TOTAL VOLUME LESS RESALE (1,000 gal)	TOTAL FLOW (MGD)
Cottage Grove	1	1	0	0	117,765	22,608	14,120	2,059	0	0	0	0	0	156,554	0.4
Dane	0	0	0	0	17,040	2,398	1,879	93	0	0	0	0	0	21,410	0.1
DeForest	0	0	0	0	157,840	53,048	12,696	4,842	0	0	0	0	0	228,426	0.6
Fitchburg	5	0	0	0	324,859	321,689	41,427	1,868	0	0	0	0	0	689,848	1.9
Madison	0	21,325	0	0	3,276,816	3,962,569	753,572	1,485,966	0	0	0	229,288	0	9,729,536	26.7
Maple Bluff	0	0	0	0	51,562	3,052	0	0	0	0	1,148	0	0	55,762	0.2
McFarland	3,355	0	0	0	144,172	43,166	0	7,530	0	0	0	0	0	198,223	0.5
Middleton	0	0	0	0	300,517	361,137	56,792	0	0	0	20,579	0	0	739,025	2.0
Monona	0	720	0	0	173,422	146,853	0	5,181	0	0	0	0	0	326,176	0.9
Morrisonville	0	0	0	0	8,969	401	0	0	0	0	114	0	0	9,484	0.0
Shorewood	0	0	0	0	42,838	6,695	0	2,347	0	0	0	0	0	51,880	0.1
Verona	0	1,858	0	0	206,550	66,686	76,221	22,504	0	0	0	0	0	373,819	1.0
Waunakee	1	38	0	0	245,107	64,780	106,043	8,892	0	0	0	0	212	425,073	1.2
Westport	0	0	0	0	19,237	17,998	0	0	0	0	0	0	0	37,235	0.1
Windsor #1	40	0	0	0	51,014	26,511	6,836	623	0	0	0	0	0	85,024	0.2
TOTALS	3,402	23,942	0	0	5,137,708	5,099,591	1,069,586	1,541,905	0	0	21,841	229,288	212	13,127,475	36.0
% OF TOTAL	0.03%	0.18%	0.00%	0.00%	39.14%	38.85%	8.15%	11.75%	0.00%	0.00%	0.17%	1.75%	0.00%		
PUBLIC WATER SYSTEMS (MGD)	0.01	0.07	0.00	0.00	14.08	13.97	2.93	4.22	0.00	0.00	0.06	0.63	0.00		36.0
CORRECTION FOR PRIVATE WATER SUPPLIERS(1)	1.04	1.04	1.04	1.04	1.04	1.04	1.04	1.04							
TOTAL ESTIMATED WATER CONSUMPTION (MGD)	**0.01**	**0.07**	**0.00**	**0.00**	**14.61**	**14.51**	**3.04**	**4.39**							36.6

(1). Total number of public water customers in MMSD collection system is approximately 99,300 per PSC records (see attached sheet). MMSD customer records indicate approximately 3,800 customers are served by private water supplies.

12/3/2009

QUESTION # 4

PUMPING STATIONS OPERATED AND MAINTAINED BY THE DISTRICT

Owner	Number of Pumping Stations
Madison Metropolitan Sewerage District	17
City of Madison	29
City of Verona	1
Village of Maple Bluff	3
Town of Dunn Sanitary District No. 1	4
Town of Dunn Sanitary District No. 3	3
Town of Madison	3
Dane County Lake Farm Park	1
Total	**61**

Quantity of Wastewater

The District received 17,292,768,000 gallons of wastewater at the Nine Springs Wastewater Treatment Plant in 2008. This was a 10.5% increase from 2007. The average daily quantities received from each municipality and through infiltration into the District's intercepting sewers in 2008 were as follows:

AVERAGE DAILY QUANTITIES OF WASTEWATER *

Municipality	2008(GPD)	% of Total
City of Fitchburg	1,960,000	4.15
City of Madison	31,649,000	66.99
City of Middleton	1,939,000	4.10
City of Monona	1,038,000	2.20
City of Verona	914,000	1.93
Village of Cottage Grove	755,000	1.60
Village of Dane	59,000	0.13
Village of DeForest	1,053,000	2.23
Village of Maple Bluff	264,000	0.56
Village of McFarland	705,000	1.49
Village of Shorewood Hills	705,000	0.43
Village of Waunakee	1,722,000	3.64
Town of Blooming Grove	5,500	0.01
Town of Blooming Grove San. Dist. No. 2	228,000	0.48
Town of Blooming Grove San. Dist. No. 10	17,000	0.04
Town of Burke Util. Dist. No. 2	4,000	0.01
Town of Burke Util. Dist. No. 6	800	<0.01
Town of Burke – Token Creek San. Dist.	123,000	0.26
Town of Dunn San. Dist. No. 1	252,000	0.53
Town of Dunn San. Dist. No. 3	74,000	0.16
Town of Dunn San. Dist. No. 4	38,000	0.08

* Information taken from MMSD's 2008 Annual Report

Municipality	2008(GPD)	% of Total
Town of Dunn Kegonsa San. Dist.	172,000	0.36
Town of Madison	932,000	1.97
Town of Middleton San. Dist. No. 5	16,000	0.03
Town of Pleasant Springs San. Dist. No. 1	61,000	0.13
Town of Verona	600	<0.01
Town of Verona Util. Dist. No. 1	23,000	0.05
Town of Vienna Util. Dist. No. 1	51,000	0.11
Town of Vienna Util. Dist. No. 2	45,000	0.09
Town of Westport Util. Dist. No. 1	165,000	0.35
Town of Westport Util. Dist. No. 2	433,000	0.92
Town of Westport Util. Dist. No. 3	15,000	0.03
Town of Westport Util. Dist. No. 4	14,000	0.03
Town of Westport - Cherokee Golf and Tennis	4,900	0.01
Town of Windsor San. Dist. No. 1	284,000	0.60
Town of Windsor San. Dist. No. 3	400	<0.01
Town of Windsor - Illinois Foundation Seed	100	<0.01
Town of Windsor - Hidden Springs San. Dist.	3,700	0.01
Town of Windsor - Lake Windsor San. Dist.	60,000	0.13
Town of Windsor - Morrisonville San. Dist.	84,000	0.18
Town of Windsor - Oak Springs San. Dist.	40,000	0.09
Total Wastewater	45,410,000	96.11
Infiltration into District Interceptors	1,838,000	3.89
Total Received at the Treatment Plant	47,248,000	100

Wastewater Treatment

The Nine Springs Wastewater Treatment Plant is located in the Town of Blooming Grove at the intersection of South Towne Drive and Moorland Road.

Preliminary treatment includes influent wastewater fine screening and grit removal. Fine screening is accomplished with three rotating band screens with 6 mm openings and a vortex grit system for grit removal. Variable speed drives for the band screens are used to control the influent well level and to maintain a minimum level above the influent flow meters. Grit is removed continuously from the vortex grit chambers. The grit and screenings are disposed of by Waste Management, Inc.

All material removed by the fine screens is conveyed to a screenings processing well. Two to four times a day the grit must be removed from the well with the operators present to oversee the pumping operation. The grit and accompanying rags are pumped to a separate settling basin (termed a "Snail") which had previously been used by the District in a primary sludge degritting process. The material settled in the snail was conveyed to small two yard dumpsters and required removal and contract hauling to the landfill three to five times per week. In November 2008, an auger was installed to transport grit removed by the Snail into a much larger grit dumpster. This eliminated the extra labor and expense required to move and haul temporary two-yard dumpsters.

QUESTION #5

WISCONSIN
DEPT. OF NATURAL RESOURCES

State of Wisconsin \ DEPARTMENT OF NATURAL RESOURCES

Carroll D. Besadny
Secretary

Box 7921
Madison, Wisconsin 53707

March 17, 1992

IN REPLY REFER TO: 8700 CWF

Mr. James Nemke
Madison Metropolitan Sewerage District
1610 Moorland Road
Madison, Wi 53713

SUBJECT: Intermunicipal Agreements

Dear Mr. Nemke:

The Department has reviewed your January 10, 1992 correspondence discussing the necessity of MMSD having intermunicipal agreements with its constituent municipalities to meet the requirements of Wis. Admin. Code NR 162.08(4)(a)2. The Department agrees with your interpretation of Wis. Stats.,s. 66.20 that it is not necessary for MMSD to have intermunicipal agreements with municipalities that have been annexed into the District, thus becoming a part of the District. The statutory requirement of annexation of territory into a metropolitan sewerage district, therefore eliminates the need for intermunicipal agreements to meet the requirements of the Clean Water Fund loan program.

If you have any questions, please call Diane Alme of my staff at 608-266-5889.

Sincerely,

Ruthe Badger

Ruthe Badger, Chief
Project Management Section
Bureau of Environmental Loans

DKA:RB
CC: Milton Donald/EA 6
Southern District
Sheila Henneger/EL
Bureau of Finance

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Wisconsin Dells, WI 53965
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Facsimile (608) 254-8582

MADISON METROPOLITAN SEWERAGE DISTRICT
RECEIVED
JAN 13 1992

January 10, 1992

Ms. Diane Alme
Wisconsin Department of Natural Resources
Bureau of Environmental Loans
P.O. Box 7921
Madison, WI 53707

RE: Intermunicipal Agreements - Financial Assistance
Our File No. 36352

Dear Ms. Alme:

Pursuant to our telephone conversation of January 8, 1992, the purpose of this letter is to discuss the necessity and desirability of Madison Metropolitan Sewerage District having intermunicipal agreements with its constituent municipalities for treatment of the wastewater that is discharged to the District from their territories which have been annexed to the District.

As I indicated by telephone, the position of MMSD is that such contracts would not be appropriate with respect to the handling of normal wastewater generated within the established boundaries of the District because all of such receipt and treatment of sewage is a matter of <u>intradistrict</u> governance and is not to be determined by <u>intermunicipal</u> agreement. All of the rights and obligations of the District relative to the territory that comprises the District are set forth in the comprehensive statutory framework of <u>Wis</u>. <u>Stats</u>. 66.20 <u>et. seq.</u> It is MMSD's view that none of those statutory rights, duties and obligations of MMSD with respect to its own territories should be subject to creation, diminution or modification by contract, and indeed any attempt to do so is subject to serious legal challenge.

Pursuant to Sec. 66.22, the creation of the district must simultaneously determine the territory to comprise the District, and must be accomplished by a finding that the formation of the District will be conducive to fiscal and physical management of a "unified system of sanitary sewage collection and treatment." Similarly, Sec. 66.26 permits the addition of territory to the District on motion of either a municipality or the District itself only again on a finding that it be conducive to a "unified" system. There is no provision for detachment of territory from the district, and indeed, once territory has become part of a unified system of collection and treatment, there can be no right of detachment.

The essence of the statutory framework is that once territory is added to the District, the District exercises direct jurisdictional control over all wastewater aspects, and governs the system directly by ordinance and rule. With respect to matters within the purview of the unified system, none of the constituent municipalities retain any rights of control under statute except as permitted by the District or as elsewhere set forth by statute. The District's right of control over the treatment process and the wastewater and the involvement of other municipalities is completely determined by statute, and is in no way dependent upon the subsequent decisions of other municipalities to grant contractual authority. For the District to seek contractual authority and arrangements from other municipalities is for it to undermine the direct statutory grants of authority that exist. The need for intermunicipal agreements relates solely to situations of extraterritorial matters.

One of the purposes of intermunicipal agreements in these cases is to provide assurance that the financial assistance will be meaningful in promoting sewage collections and treatment, and the resulting constructed facilities will be useful, appropriate and effective. Thus, <u>Wis</u>. <u>Stats</u>. Sec. 144.241 (8)(d) provides that an unsewered municipality that is not constructing a treatment work and will be disposing of wastewater to another municipality is not eligible unless it enters into an agreement with the other municipality to treat the wastewater.

This is also the purpose of NR 162.08(4)(a)(2), which requires an applicant to submit:

> "An executed intermunicipal agreement, if wastewater <u>generated by the applicant will be discharged to or through wastewater facilities of another municipality</u>. The department may waive the requirement of an executed intermunicipal agreement if an order under 144.07(1), Stats., has been issued." (Emphasis added)

Thus, the rule has applicability only to the situation where wastewater generated by the applicant will be discharged for treatment to another municipality. The concern in that case is that the applicant does not own or control the treatment facility. The purposes of the grant could be thwarted by subsequent refusal of the other municipality to accept sewage for treatment, so a contract is needed to demonstrate the applicant's access to treatment facilities. In our case, the applicant is MMSD. It does not generate any wastewater. It receives wastewater and treats it at its owned facilities. It is not dependent on any other municipality for control of treatment facilities.

It is further instructive that the rule permits waiver where the DNR has ordered a municipality to receive wastewater from another. In that case, the statutory order supersedes any need for contractual agreement. In our case, the very nature of the District renders contractual assurances superfluous, as the District owns and controls all treatment facilities and exercises direct jurisdiction over all territory within its boundaries.

Moreover, for a number of policy reasons, the execution of agreements between MMSD and other municipalities for treatment of wastewater generated within the District is dangerous and inappropriate.

1. Such an agreement undermines the District's statutory authority to own and operate the underlying treatment system.

2. Such an agreement undermines the operation of a unified treatment system over which the respective municipalities have ceased to exercise individual control.

3. Such an agreement opens the door for different and discriminatory provisions in the various contract documents, which would furnish a basis for legal claims of discrimination.

4. Such agreement would detract from the underlying financial security by substituting uncertain contractual obligations for the clear and certain statutory rights that presently apply.

AXLEY BRYNELSON

Attorneys Since 1885

We would be pleased to further discuss the various points covered in this letter. If we can provide further information or assistance, please advise.

Very truly yours,

AXLEY BRYNELSON

Griffin G. Dorschel

GGD:msk

cc: ✓Mr. John Schellpfeffer

QUESTION #17 – JUNE 8, 2008 STORM EVENT

MADISON METROPOLITAN SEWERAGE DISTRICT

1610 Moorland Road
Madison, WI 53713-3398
Telephone (608) 222-1201
Fax (608) 222-2703

Jon W. Schellpfeffer
Chief Engineer & Director

Protecting Public Health and the Environment

June 14, 2008

Mr. Larry Benson
DNR South Central Region
3911 S. Fish Hatchery Road
Madison, WI 53711

Dear Larry:

I am writing to report on the bypass and diversion events that occurred during the record rainfall events of the week of June 8, 2008. The wet weather resulted in the largest amount of water being treated at our treatment plant in a single day. Most of the pumping stations owned by the District and lift stations owned by others but maintained by the District were operating at their peak capacity. All of the wastewater that entered the treatment plant received complete advanced secondary treatment. We are still checking our records, but we think that all of the water also received ultraviolet disinfection. If any of the effluent was not disinfected, it was stored in our effluent storage lagoon.

The first significant rainfall event began on the morning of Sunday, June 8. At 9:45 am that day the total flow rate to the treatment plant was 48 MGD. Three hours later the flow rate had increased to 122 MGD. Our records show that we saw this rapid increase in flow throughout our service area. By 6:00 pm the flow rate into the plant had decreased to 70 MGD. However, the storm that started about that time and continued through the evening resulted in a peak flow rate to the plant of 145 MGD at 11:45 pm that night. During this time dual pumping was occurring at our pumping stations 1, 2, 5, 7, 8, 11, 12, and 17. Dual pumping also occurred at many of the pumping stations that we maintain for other communities. The flow rate into the plant remained above 80 MGD until about 11:45 pm on Tuesday, June 10. The rainfall events of June 12 and 13 resulted in the flow rate to the plant increasing to above 100 MGD by 8:00 pm on June 12 and remaining above 100 MGD until 5:00 pm on June 13.

metrogro

Woodley Pump Station Bypass

The first bypass in the collection system occurred at about 9:00 pm on June 8 at the City of Madison's Woodley Pumping Station. This station is located at 2712 Waunona Way on the south shore of Lake Monona. Earlier that afternoon we received a report that a significant basement backup had occurred near the station as a result of the morning storm. Our crew and a City of Madison Crew investigated at that time and could not find any problems with the station. Due to the afternoon storm, this station went on high well alarm at 6:50 pm even though both pumps in the station were pumping. Both the District and the City of Madison began receiving numerous calls indicating that basements were backing up in this area. Our crew found that the pumps at the station were operating, but the local sewers downstream of where the station discharges were full. It appeared that pumping from the station was being inhibited by the surcharged downstream sewers. To prevent further basement flooding, it was decided to pump from the station wet well to Lake Monona. At this time water was already running out of a few manholes in this area. The pumping lasted about 1.75 hours. We are estimating that a little over 50,000 gallons of water was bypassed in this event.

Pump Station 14 Bypass

The District's Pump Station 14 is located at 5000 School Road near Cherokee Marsh. This station began being adversely affected by the rainfall events on the morning of June 8. The wet well level at this station was 6.6 ft at 9:00 am. By 11:30 am the level had risen to 12.1 ft. At 1:15 pm the level had risen to a point that further increases could not be read by the level sensor. Visual observations from crews in the field indicated that the wet well level was rising to the point that basements in the area around the station would soon be backing up. The decision was made to pump water from the wet well to the surrounding surface waters to prevent the basement backups. A six inch portable pump began pumping out of the well at 12:30 am on June 9. This operation continued until 6:30 pm on June 9. During that time we estimate that about 1,080,000 gallons were bypassed.

Pump Station 13 Bypass

Pump Station 13 is located east of the airport at 3634 Amelia Earhart Drive. This station receives the flow being discharged by Pump Station 14, several City of Madison pump stations, and local gravity sewers. The wet well level at Pump Station 13 was 8.2 ft at 9:45 am on June 8. It rose quickly to 11.4 ft at 11:30 am. By 11:50 pm it was at 20.6 ft. To avoid basement backups in this area and to reduce the amount of water that would need to be handled downstream at Pump Station 10, which was already flooded, the decision was made to pump from the wet well to the area surrounding the station. A six inch portable pump pumped from this station from 1:00 am on June 9 to 5:00 am that morning. We estimate that 245,000 gallons were bypassed during this operation.

James Street Bypass

The City of Madison's James Street Pump Station is located along Starkweather Creek at 3135 James Street. This station has two ejectors rather than pumps. The ejectors were not able to keep up with the flow of water coming to the station. Many basements in the area were experiencing sewer backups. One of our mechanics connected a small portable pump to the discharge line from the station to increase the amount of water being pumped. However, this was not enough added capacity to prevent the backups. From 6:00 am to 11:30 am on June 9 a three inch portable was used intermittently to pump water from the collection system to Starkweather Creek. We estimate that about 48,000 gallons were bypassed.

Winnequah Road Bypass

On the morning of June 9 it was discovered that the covers on several manholes along Winnequah Drive had been lifted off of the manholes by the surcharged water. Water had flowed out of the manholes onto lawns along Lake Monona's Squaw Bay. Since the overflows were not reported to us and had stopped by the time we were aware that the covers were off of the manholes, it is difficult to estimate the volumes that were discharged. Since this area is just upstream of the District's Pump Station 7, I compared the ground level of the manholes to the water level in the wet well at Pump Station 7. It appears the level of water in the sewer was probably high enough to lift the covers only from about 10:00 pm to 11:00 pm on June 8. I have estimated no more than 4000 gallons were bypassed.

Pump Station 14 discharge line bypass

The forcemain from Pump Station 14 discharges to the District's 48 inch Northeast Interceptor. On the morning of June 10 it was discovered that the interceptor was surcharged at the point of the forcemain discharge. This caused some of the water being pumped into the interceptor to flow onto the surrounding ground. This is at the south side of the Cherokee Golf Course along Golf Course Road. Later in the afternoon the bypass stopped. This is consistent with our records showing that the water level in the interceptor was beginning to drop. If the bypass was due to high levels in the interceptor, the bypass could have started around 9:00 pm on June 8. We estimate that about 17,200 gallons were bypassed.

Carroll Street bypass:

During the week the forcemain at the City of Madison's Carroll Street Pumping Station at 621 North Carroll Street broke. I do not believe this was related to the storm events. While the forcemain was being repaired, wastewater flowed into Lake Mendota. I understand that reporting of this incident is being handled by the City of Madison.

Effluent Storage Lagoon Diversion

During times when the District's effluent pumps can not discharge effluent faster than it is being produced, the excess water flows into the District's effluent storage lagoons. Treated effluent began to flow into the lagoons at 11:20 am on June 8. All of the treated water was captured in the lagoons until 7:00 am on June 13. At that time the lagoons were full and water began to flow over the weirs at the diversion structure. This water was a combination of the rain that had fallen on the lagoons and the treated effluent that had been placed in it. Daily samples were collected in accordance with our WPDES permit. The results will be submitted to you. Two six-inch pumps have been installed to pump the contents of the lagoon back to the treatment plant. As of Saturday, June 14, the pumps have been turned on. However, it is expected that throughout the day there will be times when treated water will be placed in the lagoons as flows to the plant rise.

The DNR form 3400-184 Sanitary Sewer Overflow or Bypass Notification Summary Report has been completed for each bypass and is included with this letter. If you have any questions about the information that I have supplied, please contact me.

Sincerely,

Paul H Nehm

Paul H. Nehm
Director of Operations and Maintenance

Sanitary Sewer Overflow or Bypass Notification Summary Report

Form 3400-184 (4/02)

Notice: Under s.283.55 (1)(dm), Wis. Stats., and in accordance with reporting requirements in your WPDES permit, all permittees shall provide the following notices if an **unscheduled** sanitary sewer overflow or bypass occurs:

- Within **24 hours** of the occurrence, notify the DNR regional wastewater staff by telephone (FAX, email or voice mail, if staff are unavailable).
- Within **5 days** of the occurrence, provide a written report describing the overflow or bypass, including all information requested on this form. The permittee is required to submit this form or other equivalent written notification to the DNR Regional Office.

Failure to notify the Department as specified may result in fines up to $10,000 for each day of violation [s. 283.91(2), Wis. Stats.].

Personally identifiable information will be used for program administration and will also be made available to requesters as required under Wisconsin Open Records law [ss. 19.31 - 19.39, Wis. Stats.].

Instructions: Use this form to report all unscheduled sanitary sewer overflow or bypass occurrences. Attach additional information as necessary to explain or document the overflow or bypass. For the purpose of this report, an overflow or bypass is defined as the discharge of untreated sewage from the sanitary sewer collection system to a surface water and/or ground due to circumstances such as those identified by the check boxes in the overflow or bypass details section of this form.

Use one form per occurrence. A single occurrence may be more than one day if the circumstance causing the overflow or bypass results in a discharge duration more than 24-hours. If there is a stop and restart of the overflow or bypass within 24-hours, but it's caused by the same circumstances, report it as one occurrence. If the discharges are separated by more than 24 hours, they should be reported as separate occurrences.

Notification Information

Permittee (Municipality or Facility Name)	Overflow or Bypass Reported To DNR	
Madison Metropolitan Sewerage District	Date: June 9, 2008	Time: ≈ 8:00 [X] am [ ] pm
Person Representing Permittee Who Contacted DNR	DNR Office and Person Contacted	
Paul Nehm	Larry Benson Southern District	

Overflow or Bypass Details

Date(s) and Duration of Overflow or Bypass Occurrence (complete a separate form for **each occurrence**)

Start Date	Time (to nearest 15 minutes)	End Date	Time (to nearest 15 minutes)
June 8, 2008	9:00 [ ] am [X] pm	June 8, 2008	10:45 [ ] am [X] pm

Duration of the overflow or bypass (hours and minutes)	Estimated Volume of Wastewater Discharged (gallons)
1 hr 45 min	50,000

Location of the Overflow or Bypass (complete a separate form for **each discharge location**)

City of Madison Woodley Pump Station at 2712 Waunona Way in Madison

Circumstances Causing the Overflow or Bypass (check all that apply)

- [X] Rain
- [ ] Rain and Snow Melt
- [ ] Snow Melt
- [ ] Power Outage
- [ ] Plugged Sewer
- [ ] Broken Sewer
- [ ] Equipment Failure
- [X] Widespread Flooding
- [ ] Other (explain below)

Provide a narrative description to further explain why the overflow or bypass occurred. For example, describe what equipment failed, what caused the power outage, or what plugged the sewer. Flooding should only be indicated as a cause if there is significant flooding that is caused by high river, stream, or lake water levels, not just localized high water in the street.

Significant widespread flooding cause numerous basement backups in the station service area. The pumps at the station could not keep up because the downstream sewers were surcharged.

Wet Weather Data (if applicable)

Document the weather conditions if it contributed to the cause of the overflow or bypass. An overflow or bypass may be caused by a series of short rain storms or in combination with a snow melt. The wet weather data should include the cumulative amount of precipitation that caused the overflow or bypass.

Date(s) and Duration of Rainfall			
Start Date	Time (to nearest 15 minutes)	End Date	Time (to nearest 15 minutes)
June 7, 2008	☐ am ☐ pm	June 8, 2008	☐ am ☐ pm

Amount of Rainfall (nearest rain gauge to 0.1 inch accuracy)
6.34" on June 7 and 8

Amount of Snow Melt (estimated inches melted)

Contributing Soil Conditions (saturated, frozen, soil type)
Saturated soils

Where Did the Discharge from the Overflow or Bypass Go? (check all that apply)

Provide the name of the local receiving water that the wastewater enters, which could be a nearby stream, river, lake, or wetland. If discharge does not enter directly into a surface water, but indirectly by way of a ditch or storm sewer, trace the path of the ditch or storm sewer to find the receiving water.

☐ Runs on ground and absorbs into the soil.

☐ Ditch. Name of surface water it drains to:

☐ Storm sewer. Name of surface water it drains to:

☒ Surface water direct discharge: Water was discharged to Lake Monona at the station

☐ Other, describe:

Actions to Correct This Occurrence and Prevent Future Overflows or Bypasses

Describe what actions were taken to minimize the volume of wastewater discharged from the overflow or bypass reported on this form. Also describe what actions are planned to prevent or minimize future overflows or bypasses. The WPDES permit prohibits overflows or bypasses, unless certain specified conditions are met. If the permittee fails to operate and maintain the sewage collection system to prevent overflows and bypasses, they will be subject to enforcement action.

The station was operating at capacity.
The City of Madison will investigate
sources of inflow and infiltration.

Report Completed By

Authorized Representative Name (Print)	Title
Paul H. Nehm	Director of Operations and Maintenance

Authorized Representative Signature	Date
Paul H Nehm	June 14, 2008

State of Wisconsin
Department of Natural Resources

Sanitary Sewer Overflow or Bypass Notification Summary Report

Form 3400-184 (4/02)

Notice: Under s.283.55 (1)(dm), Wis. Stats., and in accordance with reporting requirements in your WPDES permit, all permittees shall provide the following notices if an **unscheduled** sanitary sewer overflow or bypass occurs:

- Within **24 hours** of the occurrence, notify the DNR regional wastewater staff by telephone (FAX, email or voice mail, if staff are unavailable).
- Within **5 days** of the occurrence, provide a written report describing the overflow or bypass, including all information requested on this form. The permittee is required to submit this form or other equivalent written notification to the DNR Regional Office.

Failure to notify the Department as specified may result in fines up to $10,000 for each day of violation [s. 283.91(2), Wis. Stats.].

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Use one form per occurrence. A single occurrence may be more than one day if the circumstance causing the overflow or bypass results in a discharge duration more than 24-hours. If there is a stop and restart of the overflow or bypass within 24-hours, but it's caused by the same circumstances, report it as one occurrence. If the discharges are separated by more than 24 hours, they should be reported as separate occurrences.

Notification Information

Permittee (Municipality or Facility Name)	Overflow or Bypass Reported To DNR	
Madison Metropolitan Sewerage District	Date: June 9, 2008	Time: about 8:00 am ☒ am ☐ pm
Person Representing Permittee Who Contacted DNR: Paul Nehm	DNR Office and Person Contacted: Larry Benson Southern District	

Overflow or Bypass Details

Date(s) and Duration of Overflow or Bypass Occurrence (complete a separate form for **each occurrence**)

Start Date	Time (to nearest 15 minutes)	End Date	Time (to nearest 15 minutes)
June 9, 2008	12:30 ☒ am ☐ pm	June 9, 2008	6:30 ☐ am ☒ pm

Duration of the overflow or bypass (hours and minutes)	Estimated Volume of Wastewater Discharged (gallons)
18 hours	1,080,000

Location of the Overflow or Bypass (complete a separate form for **each discharge location**)

The District's Pump Station 14 at 5000 School Road in Madison

Circumstances Causing the Overflow or Bypass (check all that apply)

☒ Rain	☐ Power Outage	☐ Equipment Failure
☐ Rain and Snow Melt	☐ Plugged Sewer	☒ Widespread Flooding
☐ Snow Melt	☐ Broken Sewer	☐ Other (explain below)

Provide a narrative description to further explain why the overflow or bypass occurred. For example, describe what equipment failed, what caused the power outage, or what plugged the sewer. Flooding should only be indicated as a cause if there is significant flooding that is caused by high river, stream, or lake water levels, not just localized high water in the street.

Significant widespread flooding caused the well level at the station to reach a level that would cause basement backups.

Wet Weather Data (If applicable)

Document the weather conditions if it contributed to the cause of the overflow or bypass. An overflow or bypass may be caused by a series of short rain storms or in combination with a snow melt. The wet weather data should include the cumulative amount of precipitation that caused the overflow or bypass.

Date(s) and Duration of Rainfall			
Start Date	Time (to nearest 15 minutes)	End Date	Time (to nearest 15 minutes)
June 7, 2008	☐ am ☐ pm	June 8, 2008	☐ am ☐ pm

Amount of Rainfall (nearest rain gauge to 0.1 inch accuracy): 6.34" on June 7 and 8

Amount of Snow Melt (estimated inches melted):

Contributing Soil Conditions (saturated, frozen, soil type): Saturated soils

Where Did the Discharge from the Overflow or Bypass Go? (check all that apply)

Provide the name of the local receiving water that the wastewater enters, which could be a nearby stream, river, lake, or wetland. If discharge does not enter directly into a surface water, but indirectly by way of a ditch or storm sewer, trace the path of the ditch or storm sewer to find the receiving water.

☐ Runs on ground and absorbs into the soil.

☐ Ditch. Name of surface water it drains to:

☐ Storm sewer. Name of surface water it drains to:

☒ Surface water direct discharge: Discharge was to Cherokee Marsh

☐ Other, describe:

Actions to Correct This Occurrence and Prevent Future Overflows or Bypasses

Describe what actions were taken to minimize the volume of wastewater discharged from the overflow or bypass reported on this form. Also describe what actions are planned to prevent or minimize future overflows or bypasses. The WPDES permit prohibits overflows or bypasses, unless certain specified conditions are met. If the permittee fails to operate and maintain the sewage collection system to prevent overflows and bypasses, they will be subject to enforcement action.

The pumping station was pumping at capacity. Construction is underway to increase the firm capacity of the station. Sources of inflow and infiltration will be investigated.

Report Completed By

Authorized Representative Name (Print)	Title
Paul H. Nehm	Director of Operations and Maintenance
Authorized Representative Signature	**Date**
Paul H. Nehm	June 14, 2008

State of Wisconsin
Department of Natural Resources

Sanitary Sewer Overflow or Bypass Notification Summary Report

Form 3400-184 (4/02)

Notice: Under s.283.55 (1)(dm), Wis. Stats., and in accordance with reporting requirements in your WPDES permit, all permittees shall provide the following notices if an **unscheduled** sanitary sewer overflow or bypass occurs:

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- Within **5 days** of the occurrence, provide a written report describing the overflow or bypass, including all information requested on this form. The permittee is required to submit this form or other equivalent written notification to the DNR Regional Office.

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Use one form per occurrence. A single occurrence may be more than one day if the circumstance causing the overflow or bypass results in a discharge duration more than 24-hours. If there is a stop and restart of the overflow or bypass within 24-hours, but it's caused by the same circumstances, report it as one occurrence. If the discharges are separated by more than 24 hours, they should be reported as separate occurrences.

Notification Information

Permittee (Municipality or Facility Name): Madison Metropolitan Sewerage District

Overflow or Bypass Reported To DNR

Date: June 9, 2008
Time: ≈ 8:00 [X] am [ ] pm

Person Representing Permittee Who Contacted DNR: Paul Nehm

DNR Office and Person Contacted: Larry Benson Southern District

Overflow or Bypass Details

Date(s) and Duration of Overflow or Bypass Occurrence (complete a separate form for **each occurrence**)

Start Date	Time (to nearest 15 minutes)	End Date	Time (to nearest 15 minutes)
June 9, 2008	1:00 [X] am [ ] pm	June 9, 2008	5:00 [X] am [ ] pm

Duration of the overflow or bypass (hours and minutes): 4 hours

Estimated Volume of Wastewater Discharged (gallons): 245,000

Location of the Overflow or Bypass (complete a separate form for **each discharge location**)

The Districts' Pump Station 13 at 3634 Amelia Earhart Drive in Madison

Circumstances Causing the Overflow or Bypass (check all that apply)

- [X] Rain
- [ ] Rain and Snow Melt
- [ ] Snow Melt
- [ ] Power Outage
- [ ] Plugged Sewer
- [ ] Broken Sewer
- [ ] Equipment Failure
- [X] Widespread Flooding
- [ ] Other (explain below)

Provide a narrative description to further explain why the overflow or bypass occurred. For example, describe what equipment failed, what caused the power outage, or what plugged the sewer. Flooding should only be indicated as a cause if there is significant flooding that is caused by high river, stream, or lake water levels, not just localized high water in the street.

Significant wide spread flooding caused the well level at the Station to reach a level that would cause basement backups

Wet Weather Data (If applicable)

Document the weather conditions if it contributed to the cause of the overflow or bypass. An overflow or bypass may be caused by a series of short rain storms or in combination with a snow melt. The wet weather data should include the cumulative amount of precipitation that caused the overflow or bypass.

Date(s) and Duration of Rainfall			
Start Date	Time (to nearest 15 minutes)	End Date	Time (to nearest 15 minutes)
June 7, 2008	☐ am ☐ pm	June 8, 2008	☐ am ☐ pm

Amount of Rainfall (nearest rain gauge to 0.1 inch accuracy): 6.34" on June 7 and 8

Amount of Snow Melt (estimated inches melted):

Contributing Soil Conditions (saturated, frozen, soil type): Saturated soils

Where Did the Discharge from the Overflow or Bypass Go? (check all that apply)

Provide the name of the local receiving water that the wastewater enters, which could be a nearby stream, river, lake, or wetland. If discharge does not enter directly into a surface water, but indirectly by way of a ditch or storm sewer, trace the path of the ditch or storm sewer to find the receiving water.

☐ Runs on ground and absorbs into the soil.

☐ Ditch. Name of surface water it drains to:

☐ Storm sewer. Name of surface water it drains to:

☒ Surface water direct discharge: Surface discharge eventually flowed to Starkweather Creek

☐ Other, describe:

Actions to Correct This Occurrence and Prevent Future Overflows or Bypasses

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The pumping station was operating at capacity. Construction is underway to increase the firm capacity of the station. Sources of inflow and infiltration will be investigated.

Report Completed By

Authorized Representative Name (Print)	Title
Paul H. Nehm	Director of Operations and Maintenance

Authorized Representative Signature	Date
Paul H Nehm	June 14, 2008

State of Wisconsin
Department of Natural Resources

Sanitary Sewer Overflow or Bypass Notification Summary Report

Form 3400-184 (4/02)

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Notification Information

Permittee (Municipality or Facility Name)	Overflow or Bypass Reported To DNR	
Madison Metropolitan Sewerage District	Date: June 9, 2008	Time: ~ 8:00 [X] am [ ] pm
Person Representing Permittee Who Contacted DNR: Paul Nelson	DNR Office and Person Contacted: Larry Benson Southern District	

Overflow or Bypass Details

Date(s) and Duration of Overflow or Bypass Occurrence (complete a separate form for each occurrence)

Start Date	Time (to nearest 15 minutes)	End Date	Time (to nearest 15 minutes)
June 9, 2008	6:00 [X] am [ ] pm	June 9, 2008	11:30 [X] am [ ] pm

Duration of the overflow or bypass (hours and minutes)	Estimated Volume of Wastewater Discharged (gallons)
Intermittent over 5 hours 30 minutes	48,000

Location of the Overflow or Bypass (complete a separate form for each discharge location)

City of Madison James St Pump Station at 3135 James Street in Madison

Circumstances Causing the Overflow or Bypass (check all that apply)

- [X] Rain
- [ ] Rain and Snow Melt
- [ ] Snow Melt
- [ ] Power Outage
- [ ] Plugged Sewer
- [ ] Broken Sewer
- [ ] Equipment Failure
- [X] Widespread Flooding
- [ ] Other (explain below)

Provide a narrative description to further explain why the overflow or bypass occurred. For example, describe what equipment failed, what caused the power outage, or what plugged the sewer. Flooding should only be indicated as a cause if there is significant flooding that is caused by high river, stream, or lake water levels, not just localized high water in the street.

Significant widespread flooding caused numerous basement backups in the station service area. The ejectors at the station could not keep up with the high amount of water flowing into the system

Wet Weather Data (If applicable)

Document the weather conditions if it contributed to the cause of the overflow or bypass. An overflow or bypass may be caused by a series of short rain storms or in combination with a snow melt. The wet weather data should include the cumulative amount of precipitation that caused the overflow or bypass.

Date(s) and Duration of Rainfall			
Start Date	Time (to nearest 15 minutes)	End Date	Time (to nearest 15 minutes)
June 7, 2008	☐ am ☐ pm	June 8, 2008	☐ am ☐ pm

Amount of Rainfall (nearest rain gauge to 0.1 inch accuracy): 6.34" on June 7 and 8

Amount of Snow Melt (estimated inches melted):

Contributing Soil Conditions (saturated, frozen, soil type): saturated soil

Where Did the Discharge from the Overflow or Bypass Go? (check all that apply)

Provide the name of the local receiving water that the wastewater enters, which could be a nearby stream, river, lake, or wetland. If discharge does not enter directly into a surface water, but indirectly by way of a ditch or storm sewer, trace the path of the ditch or storm sewer to find the receiving water.

☐ Runs on ground and absorbs into the soil.

☐ Ditch. Name of surface water it drains to:

☐ Storm sewer. Name of surface water it drains to:

☒ Surface water direct discharge: Discharge was to Starkweather Creek at the station

☐ Other, describe:

Actions to Correct This Occurrence and Prevent Future Overflows or Bypasses

Describe what actions were taken to minimize the volume of wastewater discharged from the overflow or bypass reported on this form. Also describe what actions are planned to prevent or minimize future overflows or bypasses. The WPDES permit prohibits overflows or bypasses, unless certain specified conditions are met. If the permittee fails to operate and maintain the sewage collection system to prevent overflows and bypasses, they will be subject to enforcement action.

The station was operating at capacity. We will discuss potential modifications of the station with the city of Madison. Sources of inflow and infiltration will be investigated.

Report Completed By

Authorized Representative Name (Print): Paul H. Nehm

Title: Director of Operations and Maintenance

Authorized Representative Signature: Paul H Nehm

Date: June 14, 2008

State of Wisconsin
Department of Natural Resources

Sanitary Sewer Overflow or Bypass Notification Summary Report

Form 3400-184 (4/02)

Notice: Under s.283.55 (1)(dm), Wis. Stats., and in accordance with reporting requirements in your WPDES permit, all permittees shall provide the following notices if an **unscheduled** sanitary sewer overflow or bypass occurs:

- Within **24 hours** of the occurrence, notify the DNR regional wastewater staff by telephone (FAX, email or voice mail, if staff are unavailable).
- Within **5 days** of the occurrence, provide a written report describing the overflow or bypass, including all information requested on this form. The permittee is required to submit this form or other equivalent written notification to the DNR Regional Office.

Failure to notify the Department as specified may result in fines up to $10,000 for each day of violation [s. 283.91(2), Wis. Stats.].

Personally identifiable information will be used for program administration and will also be made available to requesters as required under Wisconsin Open Records law [ss. 19.31 - 19.39, Wis. Stats.].

Instructions: Use this form to report all **unscheduled sanitary sewer overflow or bypass occurrences**. Attach additional information as necessary to explain or document the overflow or bypass. For the purpose of this report, an overflow or bypass is defined as the discharge of untreated sewage from the sanitary sewer collection system to a surface water and/or ground due to circumstances such as those identified by the check boxes in the overflow or bypass details section of this form.

Use one form per occurrence. A single occurrence may be more than one day if the circumstance causing the overflow or bypass results in a discharge duration more than 24-hours. If there is a stop and restart of the overflow or bypass within 24-hours, but it's caused by the same circumstances, report it as one occurrence. If the discharges are separated by more than 24 hours, they should be reported as separate occurrences.

Notification Information

Permittee (Municipality or Facility Name)	Overflow or Bypass Reported To DNR	
Madison Metropolitan Sewerage District	Date: June 10, 2005	Time: ? ☐ am ☐ pm
Person Representing Permittee Who Contacted DNR	**DNR Office and Person Contacted**	
Paul Nehm	Larry Benson Southern Region	

Overflow or Bypass Details

Date(s) and Duration of Overflow or Bypass Occurrence (complete a separate form for each occurrence)

Start Date	Time (to nearest 15 minutes)	End Date	Time (to nearest 15 minutes)
Estimated June 8	≈ 10:00 ☐ am ☒ pm	Estimated June 8	≈ 11:00 ☐ am ☒ pm

Duration of the overflow or bypass (hours and minutes)	Estimated Volume of Wastewater Discharged (gallons)
Estimated one hour	4000

Location of the Overflow or Bypass (complete a separate form for each discharge location)

Squaw Bay on Lake Monona along Winnequah Road

Circumstances Causing the Overflow or Bypass (check all that apply)

☒ Rain	☐ Power Outage	☐ Equipment Failure
☐ Rain and Snow Melt	☐ Plugged Sewer	☒ Widespread Flooding
☐ Snow Melt	☐ Broken Sewer	☐ Other (explain below)

Provide a narrative description to further explain why the overflow or bypass occurred. For example, describe what equipment failed, what caused the power outage, or what plugged the sewer. Flooding should only be indicated as a cause if there is significant flooding that is caused by high river, stream, or lake water levels, not just localized high water in the street.

Significant widespread flooding caused the well level at Pump Station 7 to rise high enough to surcharge the sewer along Winnequah Road and lift several manhole covers.

Wet Weather Data (If applicable)

Document the weather conditions if it contributed to the cause of the overflow or bypass. An overflow or bypass may be caused by a series of short rain storms or in combination with a snow melt.. The wet weather data should include the cumulative amount of precipitation that caused the overflow or bypass.

Date(s) and Duration of Rainfall			
Start Date	Time (to nearest 15 minutes)	End Date	Time (to nearest 15 minutes)
June 7, 2008	☐ am ☐ pm	June 8, 2008	☐ am ☐ pm

Amount of Rainfall (nearest rain gauge to 0.1 inch accuracy)	Amount of Snow Melt (estimated inches melted)
6.34" on June 7 and 8	

Contributing Soil Conditions (saturated, frozen, soil type)

Saturated soils

Where Did the Discharge from the Overflow or Bypass Go? (check all that apply)

Provide the name of the local receiving water that the wastewater enters, which could be a nearby stream, river, lake, or wetland. If discharge does not enter directly into a surface water, but indirectly by way of a ditch or storm sewer, trace the path of the ditch or storm sewer to find the receiving water.

☐ Runs on ground and absorbs into the soil.

☐ Ditch. Name of surface water it drains to: ________

☐ Storm sewer. Name of surface water it drains to: ________

☐ Surface water direct discharge: ________

☒ Other, describe: Water leaving the manhole flowed across lawns to Lake Monona

Actions to Correct This Occurrence and Prevent Future Overflows or Bypasses

Describe what actions were taken to minimize the volume of wastewater discharged from the overflow or bypass reported on this form. Also describe what actions are planned to prevent or minimize future overflows or bypasses. The WPDES permit prohibits overflows or bypasses, unless certain specified conditions are met. If the permittee fails to operate and maintain the sewage collection system to prevent overflows and bypasses, they will be subject to enforcement action.

The downstream pumping station was operating at capacity. Sources of inflow and infiltration will be investigated

Report Completed By

Authorized Representative Name (Print)	Title
Paul H. Nehm	Director of Operations and Maintenance

Authorized Representative Signature	Date
Paul H. Nehm	June 14, 2008

State of Wisconsin
Department of Natural Resources

Sanitary Sewer Overflow or Bypass Notification Summary Report

Form 3400-184 (4/02)

Notice: Under s.283.55 (1)(dm), Wis. Stats., and in accordance with reporting requirements in your WPDES permit, all permittees shall provide the following notices if an **unscheduled** sanitary sewer overflow or bypass occurs:

- Within **24 hours** of the occurrence, notify the DNR regional wastewater staff by telephone (FAX, email or voice mail, if staff are unavailable).
- Within **5 days** of the occurrence, provide a written report describing the overflow or bypass, including all information requested on this form. The permittee is required to submit this form or other equivalent written notification to the DNR Regional Office.

Failure to notify the Department as specified may result in fines up to $10,000 for each day of violation [s. 283.91(2), Wis. Stats.].

Personally identifiable information will be used for program administration and will also be made available to requesters as required under Wisconsin Open Records law [ss. 19.31 - 19.39, Wis. Stats.].

Instructions: Use this form to report all <u>unscheduled sanitary sewer overflow or bypass occurrences</u>. Attach additional information as necessary to explain or document the overflow or bypass. For the purpose of this report, an overflow or bypass is defined as the discharge of untreated sewage from the sanitary sewer collection system to a surface water and/or ground due to circumstances such as those identified by the check boxes in the overflow or bypass details section of this form.

<u>Use one form per occurrence.</u> A single occurrence may be more than one day if the circumstance causing the overflow or bypass results in a discharge duration more than 24-hours. If there is a stop and restart of the overflow or bypass within 24-hours, but it's caused by the same circumstances, report it as one occurrence. If the discharges are separated by more than 24 hours, they should be reported as separate occurrences.

Notification Information

Permittee (Municipality or Facility Name): Madison Metropolitan Sewerage District

Overflow or Bypass Reported To DNR

Date: June 13 — Time: ? ☐ am ☐ pm

Person Representing Permittee Who Contacted DNR: Paul Nehm

DNR Office and Person Contacted: Larry Benson Southern Region

Overflow or Bypass Details

Date(s) and Duration of Overflow or Bypass Occurrence (complete a separate form for **each occurrence**)

Start Date	Time (to nearest 15 minutes)	End Date	Time (to nearest 15 minutes)
Estimated June 9, 2008	9:00 ☐ am ☒ pm	June 10, 2008	≈ 2:00 ☐ am ☒ pm

Duration of the overflow or bypass (hours and minutes): Estimated 17 hours

Estimated Volume of Wastewater Discharged (gallons): 17,200

Location of the Overflow or Bypass (complete a separate form for **each discharge location**): South of Cherokee Golf Course on Golf Course Road

Circumstances Causing the Overflow or Bypass (check all that apply)

☒ Rain	☐ Power Outage	☐ Equipment Failure
☐ Rain and Snow Melt	☐ Plugged Sewer	☒ Widespread Flooding
☐ Snow Melt	☐ Broken Sewer	☐ Other (explain below)

Provide a narrative description to further explain why the overflow or bypass occurred. For example, describe what equipment failed, what caused the power outage, or what plugged the sewer. Flooding should only be indicated as a cause if there is significant flooding that is caused by high river, stream, or lake water levels, not just localized high water in the street.

Surcharging of downstream sewers due to widespread flooding resulted in some of the water being discharged by Pump Station 14 from entering the sewer system

Wet Weather Data (if applicable)

Document the weather conditions if it contributed to the cause of the overflow or bypass. An overflow or bypass may be caused by a series of short rain storms or in combination with a snow melt. The wet weather data should include the cumulative amount of precipitation that caused the overflow or bypass.

Date(s) and Duration of Rainfall			
Start Date	Time (to nearest 15 minutes)	End Date	Time (to nearest 15 minutes)
June 7, 2008	☐ am ☐ pm	June 8, 2008	☐ am ☐ pm
Amount of Rainfall (nearest rain gauge to 0.1 inch accuracy)		Amount of Snow Melt (estimated inches melted)	
6.34 " on June 7 and 8			

Contributing Soil Conditions (saturated, frozen, soil type)

Saturated Soils

Where Did the Discharge from the Overflow or Bypass Go? (check all that apply)

Provide the name of the local receiving water that the wastewater enters, which could be a nearby stream, river, lake, or wetland. If discharge does not enter directly into a surface water, but indirectly by way of a ditch or storm sewer, trace the path of the ditch or storm sewer to find the receiving water.

☐ Runs on ground and absorbs into the soil.

☐ Ditch. Name of surface water it drains to: ______

☐ Storm sewer. Name of surface water it drains to: ______

☐ Surface water direct discharge: ______

☒ Other, describe: Ran onto ground and drained to Cherokee Marsh

Actions to Correct This Occurrence and Prevent Future Overflows or Bypasses

Describe what actions were taken to minimize the volume of wastewater discharged from the overflow or bypass reported on this form. Also describe what actions are planned to prevent or minimize future overflows or bypasses. The WPDES permit prohibits overflows or bypasses, unless certain specified conditions are met. If the permittee fails to operate and maintain the sewage collection system to prevent overflows and bypasses, they will be subject to enforcement action.

The elevation of the top of the manhole will be raised. Sources of inflow and infiltration will be investigated.

Report Completed By

Authorized Representative Name (Print)	Title
Paul H. Nehm	Director of Operations and Maintenance
Authorized Representative Signature	Date
Paul H. Nehm	June 14, 2008

QUESTION #17 - JUNE 8, 2008 STORM EVENT

MADISON METROPOLITAN SEWERAGE DISTRICT

1610 Moorland Road
Madison, WI 53713-3398
Telephone (608) 222-1201
Fax (608) 222-2703

Jon W. Schellpfeffer
Chief Engineer & Director

Protecting Public Health and the Environment

September 24, 2008

Mr. Larry Benson
DNR South Central Region
3911 S. Fish Hatchery Road
Madison, WI 53711

Subject: **Response to June Flooding Events and Subsequent Sewage Overflows**

Dear Mr. Benson:

I am writing to tell you some of the things we are doing to help to reduce the possibility of needing to bypass flows in the northeast portion of the District's collection system during extreme rainfall events. The District is also taking the following actions to lessen the likelihood of future events overwhelming the sewerage system throughout our service area:

1. The District is reviewing its design standards for sizing interceptor sewers and pumping stations. The District currently provides an allowance for high flows in these facilities that varies from peak flow capacities 4.0 times greater than the average daily flows for facilities with average day design flows of one million gallons per day to peak flow capacities 2.5 times greater than the average daily flows for facilities with average day design flows of 20 million gallons per day. The review will include data from the storm events of the past fifteen years. If higher peaking factors are judged to be necessary, the schedule for construction of replacement interceptor sewers and pumping stations will need to be accelerated, and new and replaced facilities will be larger. This will lessen the likelihood of back-ups and overflows and will result in higher costs for service.
2. The District is reviewing its design standards for materials used in constructing interceptor sewers, including manholes, to assure that rain waters are less likely to leak into these facilities during heavy rains and floods.
3. The District is reviewing flow data and inspecting its existing interceptor sewers to identify and repair defects that allowed excessive rain water leakage into the District's system. Although no major defects have been found, several manholes were identified that needed repair upstream of Pumping Station 14.
4. The District is reviewing flow data from its customer communities collected during the recent high flow events. This review will identify likely areas in community sewer systems that experienced excessive leakage during the recent

metrogro

high flow events. The District will work with these communities to address these areas.

5. The District will make greater efforts to educate the public in the area of water conservation and how to prevent rain water from leaking into basements. Water conservation and reduced inflow will have positive impacts in both dry and wet weather.

The District is in the process of developing a 50 year Master Plan. Included in this plan will be an evaluation of satellite versus central treatment, future growth, and the condition and capacity of existing facilities. The District is also developing an Asset Management strategy for the collection system and the treatment plant.

We are currently in the process of addressing issues in the northeast portion of our collection system.

- Upgrades are being made at Pump Station 13 and Pump Station 14 to increase the firm capacity of each of these stations. The work is expected to be completed in November.
- We are also relining a portion of our Northeast interceptor between Pump Station 13 and Pump Station 14 at the Dane County Regional Airport. This is approximately 5,300 feet of 48 inch diameter pipe that was installed in the 1960's and crosses under two airport runways. Four of the five sections of this project have been completed. The last section is scheduled for installation this week.
- We have recently sent out requests for proposals for design of facilities to address the condition and capacity of our Northeast Interceptor upstream of Pump Station 10. This section of interceptor has less capacity than the interceptors upstream of it. This project will result in the installation of about 9,500 feet of parallel relief sewer or replacement of the existing pipe with a larger diameter pipe.
- The ability to convey additional volumes of water from the northeast portion of our system will require additional capacity downstream. We have purchased land on which to construct Pump Station 18. This station will relieve Pump Station 7 and provide additional pumping capacity directly to the treatment plant. Design is expected to begin in 2010, and the new pump station and force main should be operational by the end of 2013.

We will be continuing our current program of cleaning and televising a portion of our interceptor system each year. This maintenance program helps us to identify those portions of the system which need repairs or capacity improvements.

Please contact me if you have any questions on this information.

Sincerely,

Jon W. Schellpfeffer

Jon W. Schellpfeffer
Chief Engineer and Director

QUESTION # 26 (B)

Table 2.2
Pumping Station Capacities and Projected Flows

Madison Metropolitan Sewerage District

Pumping Station No.	Diversion Status	Station Maximum Pumping Capacity (mgd)	Station Firm Pumping Capacity (mgd)	Average Flows (mgd)			Benchmark Peak Flows (mgd) per Madison Design Curve[4]			Ratio Firm Capacity / Benchmark		Ratio Max. Capacity / Benchmark	
				2000	2007[1]	2030[3]	2000	2007	2030	2007	2030	2000	2030
1	Normal Scenario: PS15 pumps tp PS8.	38.3	35.3	6.87	5.37	5.54	20.27	16.47	16.91	2.14	2.09	2.33	2.27
2		41.0	41.0	4.48	10.59	10.74	21.34	29.18	29.52	1.41	1.39	1.41	1.39
3		1.5	1.5	0.30	0.34	0.35	1.20	1.36	1.40	1.11	1.08	1.11	1.08
4		4.2	4.2	0.91	1.16	1.03	3.69	4.53	4.10	0.93	1.02	0.93	1.02
5		3.6	3.6	0.70	0.60	0.63	2.80	2.40	2.52	1.50	1.43	1.50	1.43
6		24.2	24.2	7.75	2.60	1.74	15.23	8.94	6.38	2.71	3.80	2.71	3.80
7		45.0	39.0	20.15	16.32	23.94	42.95	41.99	59.85	0.93	0.65	1.07	0.75
8		34.1	34.0	8.77	7.38	9.31	24.89	21.53	26.18	1.58	1.30	1.58	1.30
9		4.5	4.5	0.81	0.87	1.28	3.24	3.48	4.92	1.29	0.91	1.29	0.91
10		42.2	42.2	8.79	8.11	13.26	24.94	23.31	35.26	1.81	1.20	1.81	1.20
11		31.2	25.5	7.50	8.61	15.03	21.82	24.51	39.18	1.04	0.65	1.27	0.80
12		23.5	16.6	4.32	5.42	10.48	13.71	16.60	28.92	1.00	0.57	1.42	0.81
13		20.2	20.0	5.60	6.53	9.14	17.06	19.42	25.77	1.03	0.78	1.04	0.78
14		15.6	15.0	3.34	4.07	5.26	11.04	13.04	16.19	1.15	0.93	1.20	0.96
15		8.8	5.8	1.30	1.27	1.83	4.99	4.89	6.65	1.19	0.87	1.80	1.32
16		18.7	18.7	1.37	1.85	3.05	5.48	6.71	10.23	2.78	1.83	2.78	1.83
17		4.6	4.6	0.67	0.82	3.41	2.68	3.28	11.24	1.40	0.41	1.40	0.41

Pumping Station No.	Diversion Status	Station Maximum Pumping Capacity (mgd)	Station Firm Pumping Capacity (mgd)	Average Flows (mgd)			Benchmark Peak Flows (mgd) per Madison Design Curve(4)			Ratio Firm Capacity / Benchmark		Ratio Max. Capacity / Benchmark	
				2000	2007(1)	2030(3)	2000	2007	2030	2007	2030	2000	2030
8	Alternate Scenario: PS15 pumps to PS16	34.1	34.0	7.47	6.11	7.48	21.75	18.36	21.77	1.85	1.56	1.86	1.57
11		31.2	25.5	8.80	9.88	16.86	24.96	27.52	43.16	0.93	0.59	1.13	0.72
12		23.5	16.6	5.62	6.69	12.31	17.11	19.82	33.12	0.84	0.50	1.19	0.71
15		9.4	4.3	1.30	1.27	1.83	4.99	4.89	6.65	0.88	0.65	1.92	1.41
16		18.7	18.7	2.67	3.12	4.88	9.14	10.43	15.20	1.79	1.23	1.79	1.23

Notes:

1). Year 2007 actual average flows are based on MMSD metered data for PS1, 2, 3, 5, 6, 7, 8, 10, 11, 16 and 17. Pump run-time records are used at all other stations.

2). Year 2007 was selected as the baseline year for recent average annual flows due to the unusually wet weather experienced in 2008. Year 2007 is believed to be a more representative year for purposes of analysis and comparison.

3). Projected Year 2030 average flows are per CARPC's January 2009 report. These flows are generated from population forecasts utilizing traffic analysis zones and application of an uncertainity factor (UF).

4). Benchmark peak flow requirements are computed per Madison Design Curve. Peaking factor of 4.0 applied for all average flowrates less than 1 MGD. Peaking factor of 2.5 applied for all average flowrates greater than 20 MGD. All other peaking factors equal to 4/(ADF)^0.158).

5). Year 2007 flows from PS 1 were apportioned to downstream pumping stations as follows: (a). 5.20 MGD to PS 2; and (b). 0.17 MGD to PS 6. Benchmark peak flows were based on these average flowrates.

6). All flows from PS 15 in Year 2007 were directed to PS 8. No flow was diverted to PS 16.

7). PS15 pump capacities pumping to PS16, as shown, are different than those pumping to PS8.

QUESTION # 28

Table 2.3
Forcemain Capacities and Characteristics
Madison Metropolitan Sewerage District

Pumping Station Forcemain No.	Forcemain Characteristics: Segment Length (feet)	Dia. (inches)	Mat'l	Year Installed	Comments	Nominal FM Capacity (mgd) based on 8 fps velocity	2030 Benchmark Peak Flows (mgd): If PS15 pumps to PS8	If PS15 pumps to PS16
1 (to PS 6)	2,638	30	RCCP	1948		25.4	0.00	
	1,340	24	DI	2000	Segment from PS1 to E. Washington Ave.	16.2		
1 (to PS 2)	998	20	PVC	1995	Segment under Monona Terrace	11.3	16.91	
	14,205	30	DI	2002	Balance of FM from E. Wash. Ave. to PS2	25.4		
2	17,064	36	DI	2001	From PS2 to near old meter vault @ NSWTP	36.5	29.52	
	364	36	DI	2005	Installed during the 10th Addition			
3	5	8	CI	1959	Original forcemain remaining	1.8	1.40	
	21	8	DI	2000	Installed dring PS2FM replacement			
4	100	16	CI	1959	Original forcemain remaining	7.2	4.10	
	60	16	DI	2000	Installed dring PS2FM replacement			
	28	16	DI	1996	Segment from new PS5 to 1959 junction	7.2		
5	504	16	RCCP	1959	Segment to PS15 FM junction	7.2	2.52	
	1,746	24	RCCP	1959	Segment from PS5/15 junction to Whitney	16.2		
6	7,208	36	RCCP	1948		36.5	6.38	
	6,996	2 x 36	RCCP	1948, 63	Dual forcemains from PS7 to plant grounds	65 (based on 8 fps) 55-60 (based on transients) see note 3 below		
7	1,332	48	RCCP	1963	Through plant grounds to 10th Add connection		59.85	
	323	48	DI	2005	Installed during the 10th Addition			
	13,174	42	RCCP	1964	78' of 42" abandoned during 10th Addition	49.7		
8	194	36	RCCP	1964	Located outside of PS#8	36.5	26.18	21.77
	334	42	DI	2005	Installed during the 10th Addition	49.7		
9	4,812	20	DI	1987		11.3	4.92	

Pumping Station Forcemain No.	Forcemain Characteristics					Nominal FM Capacity (mgd) based on 8 fps velocity	2030 Benchmark Peak Flows (mgd)	
	Segment Length (feet)	Dia. (inches)	Mat'l	Year Installed	Comments		If PS15 pumps to PS8	If PS15 pumps to PS16
10	11,112	36	RCCP	1964		36.5	35.26	
11	3,945	36	RCCP	1965	230' of 36" abandoned during 10th Addition	36.5	39.18	43.16
	91	36	DI	2005	Installed during the 10th Addition			
	0	30	RCCP	1964	All 30" was abandoned during 10th Addition			
12	4,795	36	RCCP	1968		36.5	28.92	33.12
13	2,588	36	RCCP	1969		36.5	25.77	
14	4,354	30	RCCP	1971		25.4	16.19	
15 (to PS8)	2,467	24	DI	1974	Segment from PS15 to Thorstrand air release	16.2	6.65	
	4,811	20	DI	1974	Segment from Thorstrand to PS5 FM junction	11.3		
	1,746	24	RCCP	1959	Segment from PS5 FM juction to Whitney Way	16.2		
15 (to PS16)	1,378	24	DI	1974	Segment from PS15 to junction near Univ. Ave.	16.2	6.65	
	4,893	30	RCCP	1982	Segment from FM junction to near PS16	25.4		
16	7,214	36	DI	1979	Segment from PS16 to Gammon high point	36.5	10.23	15.20
	2,965	30	DI	1980	Segment from high point to near Min. Pt. Rd.	25.4		
17	13,357	16	DI	1995	Segment from PS17 to Hwy. 18/151 high pt.	7.2	11.24	
	3,071	20	DI	1995	Forced gravity segment from high pt. to NSVI	11.3		

Notes:
1 Benchmark flows per Table 2.2
2 Nominal FM Capacities shown are based on 8 feet/sec velocity in principal FM segments
3 Limiting capacity for the PS7 FM is 55-60 MGD due to maximum allowable transient pressures in 36"-1948 FM.

QUESTION # 33

MADISON METROPOLITAN SEWERAGE DISTRICT

1610 Moorland Road
Madison, WI 53713-3398
Telephone (608) 222-1201
Fax (608) 222-2703

Jon W. Schellpfeffer
Chief Engineer & Director

Protecting Public Health and the Environment

August 11, 2009

Terri Winans
Waunona Sanitary District No. 2
3325 Thurber Avenue
Madison, WI 53714

Subject: **Request for Efforts to Reduce Infiltration and Inflow**

Dear Ms. Winans:

The District has measured higher than normal volumes of wastewater discharges from Waunona Sanitary District No. 2 over the past eighteen months. This coincides with a period of above normal precipitation and resulting high groundwater levels. The combination of higher than normal wastewater volumes and above normal precipitation and ground water levels indicate potentially excessive amounts of infiltration and inflow (I/I) in your sanitary sewer system. Higher wastewater volumes lead to the need for the Madison Metropolitan Sewerage District to increase the capacity of the conveyance and treatment systems that the District owns and operates. Higher wastewater volumes also increase the District's operating costs. In turn, this increases the sewer service bill we send you each quarter.

If excessive levels of I/I can be eliminated in local sewer systems, the need to expand capacity can be delayed and operating costs for pumping can be reduced. It is generally less expensive to eliminate sources of I/I than to provide capacity and operate a system to handle this clear water. The first step is to identify sources of excessive I/I. Once sources have been identified, remedial efforts can be defined to reduce or eliminate them. Improvements to manholes in flood-prone areas can often be made with limited expense. Improvements include raising the manhole above the flood-zone, replacing leaky adjusting rings, installing gasketed manhole covers, and installing chimney seals.

The Madison Metropolitan Sewerage District requests that Waunona Sanitary District No. 2 include money in your 2010 budget for an I/I study of your sanitary sewer system. Efforts should concentrate in areas with higher groundwater levels, such as near streams, wetlands, and lakes, and on roadways subject to flooding. Inspection of homes for illegally connected sump pumps and roof drains should be conducted as part of this effort. Funding of remedial efforts required in the public portion of your sanitary sewer system, depending on the results of the study, should be considered in the following years. Illegally connected sump pumps and roof drains should be disconnected from the sanitary sewer system as soon as practical, but no more than one year after notification.

metrogro

The Madison Metropolitan Sewerage District will continue to monitor available capacity in our facilities and will upgrade these facilities as necessary to provide capacity for expected growth throughout the District. Your efforts to eliminate excessive I/I will assure that future District expenditures will not be made sooner than necessary, will be limited to the level necessary to provide capacity for expected growth, and not over-sized to handle excessive I/I. Your quarterly charges from the District will also be reduced as this clear water is eliminated from the system.

If you have any questions concerning this request, please contact me by telephone at 222-1201 ext. 266 or by email at jons@madsewer.org. We appreciate your cooperation in assuring continued provision of cost-effective sewer service.

Sincerely,

Jon W. Schellpfeffer
Chief Engineer and Director

Appendix A8

Analysis of West Intercepting System, July 2010

Appendix A8 - Analysis of West Intercepting System
Madison Metropolitan Sewerage District
July, 2010

Outline

This analysis is organized into the following sections:

- Introduction
- Background and History
- CARPC's *Collection System Evaluation*
    - West Interceptor Relief
    - Old West Interceptor
    - Midvale Relief
    - Spring Street Relief
    - Randall Relief
    - Campus Relief
- Conclusions & Recommendations
- Appendices

Introduction

A design memorandum for capacity improvements in the West Intercepting System at the UW campus was included in Appendix V of the 2002 *Collection System Facilities Plan.* The capacity analysis performed in that memo found that the District's interceptors through the west end of campus did not have adequate capacity to serve existing or future peak flows from upstream areas and the UW campus. The memo recommended the installation of a relief interceptor through the west campus area from the intersection of Randall Avenue and Dayton Street to the intersection of Campus Drive and Walnut Street. The District constructed this relief interceptor in four phases, with the first project being completed in 1999 and the final project being completed in 2004.

Even with the Campus Relief project completed, however, the design memo noted that capacity improvements would likely be needed in the West Intercepting System between Walnut Street and Whitney Way in the long term. This appendix will update the 2002 capacity analysis for the West Intercepting System west of Walnut Street based on CARPC's 2009 *Collection System Evaluation.*

Background and History

The West Intercepting System is a complex network of parallel sewers that provides service to the near west side of the City of Madison, the City of Middleton, the Village of Shorewood, and the Town of Westport. In general the system is comprised of two parallel sewer networks that extend westward from Pumping Stations 2 and 8. A more complete description of these improvements and the interconnections in these systems can be found at the end of this document in an internal memo by Gerald Sachs dated July 16, 2008 (Appendix A8-1).

The sewers comprising this system range in age from six to 94 years and in size from 18” to 48”. A summary of the main components of the system are shown in Table A8-1:

Table A8-1: West Intercepting System Characteristics

Interceptor Name	Limits		Size (in)	Primary Years of Construction
	From	To		
West Interceptor Relief	Randall Ave & Dayton St	Old Middleton Rd & Whitney Way	21-36	1959
Old West Interceptor	PS 2	PS 15	12-24	1916 & 1931
Midvale Relief	Shorewood Blvd	Midvale Blvd	21	1971
Spring Street Relief	PS 2	Spring St & Randall Ave	24	1940
Randall Relief	PS 8	Randall Ave & Dayton St	33-48	1964
Campus Relief	Randall Ave & Dayton St	Campus Dr & Walnut St	27-48	1999-2004

CARPC’s *Collection System Evaluation (2009)*

Much of the West Intercepting System has adequate capacity at this time and in the long term future. The Spring Street Relief, Randall Relief, and Campus Relief are not expected to have capacity needs through the year 2060 according to CARPC’s evaluation. CARPC has identified several sections of the West Intercepting System located west of Walnut Street that are in need of capacity relief in the near term. Each major component of the system with near term capacity needs is discussed in turn.

West Interceptor Relief

CARPC’s capacity evaluation suggests an urgent need to provide additional capacity in the West Interceptor Relief Sewer (Table A8-2). Their evaluation estimates that approximately 4,300 feet of 24” and 27” sewer between Whitney Way (MH02-545) and Shorewood Boulevard (MH02-036) has already reached capacity, along with another 4,300 feet of 36” sewer between Shorewood Boulevard and Walnut Street. Many other segments of this interceptor are estimated to reach capacity between 2010 and 2020.

Table A8-2: West Interceptor Relief

Flow Type	From	To	Length (ft)	Pipe Dia. (in)	Nominal Capacity (mgd)	*Peak Flows (mgd)* / Percent Nominal Capacity												Capacity Reached
						2000		2010 UF		2020 UF		2030 TAZ		2030 UF		2060 UF		
GR	MH02-547	MH02-546	497	24	12.57	*7.42*	59%	*7.75*	62%	*8.15*	65%	*7.47*	*59%*	*8.54*	68%	*9.52*	76%	> 2060
GR	MH02-546	MH02-545	192	27	8.95	*7.42*	83%	*7.75*	87%	*8.15*	91%	*7.47*	*83%*	*8.54*	95%	*9.52*	106%	> 2060
GR	MH02-545	MH02-538	3,121	27	8.95	*9.79*	109%	*10.22*	114%	*10.72*	120%	*10.20*	*114%*	*11.21*	125%	*12.15*	136%	**2000**
GR	MH02-538	MH02-536	1,200	24	8.52	*9.79*	115%	*10.22*	120%	*10.72*	126%	*10.20*	*120%*	*11.21*	132%	*12.15*	143%	**2000**
GR	MH02-536	MH02-535	600	21	10.44	*9.79*	94%	*10.22*	98%	*10.72*	103%	*10.20*	*98%*	*11.21*	107%	*12.15*	116%	**2010-2020**
GR	MH02-535	MH02-532	841	21	10.44	*9.79*	94%	*10.22*	98%	*10.72*	103%	*10.20*	*98%*	*11.21*	107%	*12.15*	116%	**2010-2020**
GR	MH02-532	MH02-531A	65	36	12.19	*9.98*	82%	*10.42*	85%	*10.91*	89%	*10.39*	*85%*	*11.40*	94%	*12.34*	101%	2030-2060
GR	MH02-531A	MH02-519	4,363	36	12.19	*12.58*	103%	*13.07*	107%	*13.20*	108%	*12.93*	*106%*	*14.17*	116%	*15.27*	125%	**2000**
GR	MH02-519	MH02-518	465	36	25.85	*12.58*	49%	*13.07*	51%	*13.62*	53%	*12.93*	*50%*	*14.17*	55%	*15.27*	59%	> 2060
SI	MH02-518	MH02-516	204	36	12.19	*12.58*	103%	*13.07*	107%	*13.62*	112%	*12.93*	*106%*	*14.17*	116%	*15.27*	125%	2000
GR	MH02-516	MH08-228	10	36	12.19	*14.21*	117%	*14.66*	120%	*15.16*	124%	*14.45*	*119%*	*15.67*	129%	*16.75*	137%	2000
GR	MH08-228	MH02-513	1,112	36	12.19	*6.68*	55%	*6.89*	57%	*7.13*	58%	*6.79*	*56%*	*7.36*	60%	*7.87*	65%	> 2060
GR	MH02-513	MH08-209	2,175	36	12.19	*9.29*	76%	*9.77*	80%	*10.28*	84%	*9.47*	*78%*	*10.78*	88%	*11.92*	98%	> 2060
GR	MH08-209	MH08-207	625	36	12.19	*7.74*	63%	*8.01*	66%	*8.30*	68%	*7.80*	*64%*	*8.59*	70%	*9.42*	77%	> 2060
GR	MH08-207	MH02-503	463	36	12.19	*3.63*	30%	*3.76*	31%	*3.90*	32%	*3.66*	*30%*	*4.03*	33%	*4.40*	36%	> 2060
GR	MH02-503	MH02-502	142	36	12.19	*3.63*	30%	*3.76*	31%	*3.90*	32%	*3.66*	*30%*	*4.03*	33%	*4.40*	36%	> 2060
GR	MH02-502	MH02-014A	513	36	12.19	*5.34*	44%	*5.48*	45%	*5.63*	46%	*5.31*	*44%*	*5.78*	47%	*6.23*	51%	> 2060

Table A8-3: 2010 Average Daily Flows to West Interceptor Relief

	PS 15	PS 5	Gettle PS	Gravity Flow	Total Flow
2010 Measured Flows(1)	1.34	0.67	0.70	0.03	2.74
2010 CARPC Flows	1.58	0.61	0.85		3.05
Notes: (1). January, 2010 through June, 2010 (2). All values in units of ‘mgd’.					

It appears that CARPC's projections for 2010 flowrates are reasonable. The upstream terminus of the West Interceptor Relief receives flow from four major sources: (1). Pumping Station No. 15; (2). Pumping Station No. 5; (3). City of Madison's Gettle Pumping Station; and (4). Gravity flow near Whitney Way and Old Middleton Road. A summary of these flows based on MMSD pumping records and a comparison to CARPC's flow projections is shown in Table A8-3.

Hydraulic modeling of the West Interceptor Relief Sewer indicates that appreciable surcharging is expected to occur in sewer segments west of Shorewood Boulevard for CARPC's 2010 UF flows (see Appendix A8-2). Field monitoring of this interceptor during wet weather events and historical data does not confirm the surcharging indicated by CARPC's analysis or by the hydraulic model, however. It is possible that this interceptor is able to withstand a certain degree of surcharging without adverse effects due to the lack of local main and lateral connections between Whitney Way and Shorewood Boulevard.

The hydraulic model was used to simulate the effect of a 36" sewer built parallel to the West Interceptor Relief between Walnut Street and Whitney Way. This sewer should have adequate capacity to convey the flows projected by CARPC for 2060. No surcharging is observed in either of the 36" sewers for CARPC's 2060 peak flowrate of 15.3 mgd through the system (see Appendix A8-3).

The Campus Relief (Phase IV) project ended just east of Walnut Street. It is assumed that a new relief sewer for the West Interceptor Relief would begin on the west side of Walnut Street and that the existing siphon underneath Walnut Street would not receive additional capacity. Construction of a new siphon at this road crossing is not feasible due to the adjacent bridge abutments in the area. The hydraulic model estimates a difference in water surface elevation of approximately seven inches across the siphon for CARPC's 2060 projected flowrate of 15.3 mgd. Thus, the existing siphon should be adequate. The siphon was cleaned in 2008 and was found to be in reasonably good condition.

The most likely route for installation of a new relief sewer from the west side of Walnut Street to Whitney Way is parallel to the existing West Interceptor Relief. The new sewer would be located in or just outside the existing railroad corridor along the entire length. There are many existing utilities along this corridor and construction would be difficult. Additionally, the City of Madison has plans to install a new storm box culvert between Shorewood Drive and Walnut Street along this same corridor and the new relief sewer would need to be closely coordinated with that project.

Old West Interceptor

The Old West Interceptor (OWI) is one of the District's oldest facilities in the collection system. It was constructed in 1916 from Pumping Station No. 2 to the intersection of University Avenue and Farley Avenue and extended to the City of Middleton in 1931. Those portions of the OWI which are upstream of Pumping Stations No. 5 and 15 have sufficient capacity for projected 2060 flows. The OWI upstream of PS No. 5 and along the shore of Lake Mendota (MH05-011 to MH05-021) was rehabilitated with a cured-in-place-pipe (CIPP) in 2011. CARPC's analysis of the OWI (see Table A8-4) indicates two sections with capacity needs prior to 2030: (1). Approximately 4,000 feet of 18”-21” sewer on University Avenue between Farley Avenue and Paunack Place; and (2). Approximately 2,200 feet of 24” sewer on Regent Street between S. Orchard Street and N. Murray Street.

University Avenue Section

CARPC’s capacity evaluation estimates that capacity in the Old West Interceptor on University Avenue from Farley Avenue to Paunack Place will be reached between 2010 and 2020. Hydraulic modeling of this section shows moderate surcharging between two and three feet between MH02-032 (Walnut Street) and MH02-042 (Ridge Street) for 2020 UF flows (see Appendix A8-4).

The City of Madison has plans for a full reconstruction of University Avenue between Grand Avenue and Breese Terrace in 2011. Given the age and possible hydraulic constraints in this section prior to 2020, an opportunity exists for the District to replace or rehabilitate the Old West Interceptor as part of the City’s street reconstruction project. As mentioned in the previous section, there is also a need to provide additional capacity in the West Interceptor Relief Sewer in the near term. The West Interceptor Relief and Old West Interceptor run roughly parallel to each other from the western edge of the UW campus to Whitney Way. Rather than provide additional capacity in each system, it would be more cost effective for the District to interconnect portions of these two systems and build additional capacity in only one system (i.e. a parallel relief sewer to the West Interceptor Relief).

Downstream of Pumping Station No. 5 the Old West Interceptor serves the Village of Shorewood and the City of Madison. This includes flows from subbasins 8-H, 8-I, and 8-J on Figure A8-1. Most of the future growth and increased flows to this interceptor are estimated to occur in the Hilldale Mall area (Subbasin 8-H). In order to alleviate overloading of the OWI, flows from Subbasin 8-H could be diverted from the OWI to the West Interceptor Relief at MH02-043 near Ridge Street upon installation of a new 36” relief sewer. Thus, the section from MH02-060 to MH02-043 would connect to the West Interceptor Relief sewer at MH02-528. Under this scenario the section from MH02-042 to MH02-513 would receive flow only from subbasins 8-I and 8-J, which are both located entirely in the City of Madison. A comparison of the projected flowrates for existing conditions and the OWI flow diversion scenario is presented in Table A8-5.

Table A8-5 demonstrates that diverting flows in subbasin 8-H away from the OWI and into the West Interceptor Relief system will alleviate capacity exceedances in this section of the OWI through the year 2060. A new relief sewer for the West Interceptor Relief system would have to be designed to accept this additional flow. Even without the need to provide additional capacity in this section of the OWI, it should be rehabilitated with a cured-in-place liner given its age and condition history (numerous cracked sections of VC pipe). This rehabilitation should take place in conjunction or shortly after the City’s street reconstruction project in 2011.

The City of Madison has indicated a desire to provide direct connections from homes and businesses to the OWI along University Avenue. Given the proposed flow diversion in the OWI,

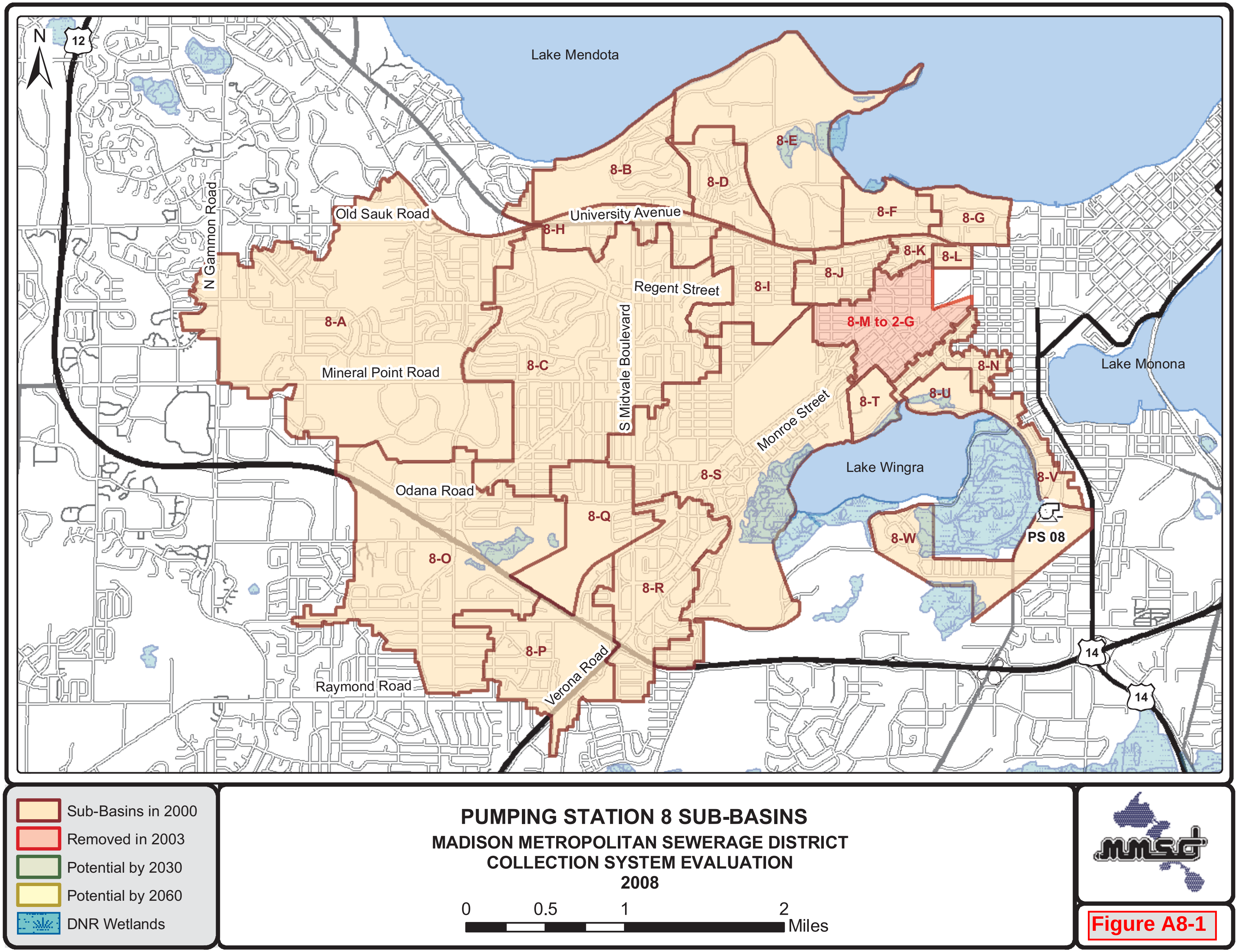
Table A8-4: Old West Interceptor (Downstream of PS 5 to PS 2)

Flow Type	From	To	Length (ft)	Pipe Dia. (in)	Nominal Capacity (mgd)	*Peak Flows (mgd)* / Percent Nominal Capacity: 2000		2010 UF		2020 UF		2030 TAZ		2030 UF		2060 UF		Capacity Reached
GR	MH02-060	MH02-047	5,066	12-18	2.09	*0.71*	34%	*0.89*	43%	*1.07*	51%	*0.82*	39%	*1.25*	60%	*1.84*	88%	> 2060
GR	MH02-047	MH02-041	1,914	18	2.71	*0.71*	26%	*0.89*	33%	*1.07*	39%	*0.82*	30%	*1.25*	46%	*1.84*	68%	> 2060
GR	MH02-041	MH02-038	1,063	18	2.71	*1.40*	52%	*1.67*	62%	*1.93*	71%	*1.49*	55%	*2.20*	81%	*2.93*	108%	2030-2060
GR	MH02-038	MH02-034	1,460	18	1.92	*1.40*	73%	*1.67*	87%	*1.93*	101%	*1.49*	78%	*2.20*	115%	*2.93*	153%	**2010-2020**
GR	MH02-034	MH02-032	816	20	2.84	*2.41*	85%	*2.76*	97%	*3.11*	110%	*2.49*	88%	*3.47*	122%	*4.28*	151%	**2010-2020**
GR	MH02-032	MH02-513	1,704	21	3.24	*2.41*	74%	*2.76*	85%	*3.11*	96%	*2.49*	77%	*3.47*	107%	*4.28*	132%	**2020-2030**
GR	MH02-021	MH02-014A	2,153	24	4.85	*3.44*	71%	*3.33*	69%	*3.22*	66%	*3.11*	64%	*3.11*	64%	*3.11*	64%	> 2060
GR	MH02-012	MH02-011	450	24	4.62	*0.00*	0%	*1.36*	29%	*1.52*	33%	*1.25*	27%	*1.68*	36%	*2.06*	45%	> 2060
GR	MH02-011	MH02-008	900	24	4.62	*5.65*	122%	*6.95*	150%	*7.32*	158%	*6.59*	143%	*7.69*	166%	*8.85*	192%	**2000**
GR	MH02-008	MH02-005A	1,260	24	5.27	*5.65*	107%	*6.95*	132%	*7.32*	139%	*6.59*	125%	*7.69*	146%	*8.85*	168%	**2000**
GR	MH02-005A	MH02-402	1,296	30	12.43	*5.65*	45%	*6.95*	56%	*7.32*	59%	*6.59*	53%	*7.69*	62%	*8.85*	71%	> 2060
GR	MH02-005	MH02-101	1,268	24	8.89	*0.23*	3%	*0.22*	2%	*0.22*	2%	*0.21*	2%	*0.21*	2%	*0.21*	2%	> 2060
GR	MH02-101	MH02-402	10	36	26.21	*7.38*	28%	*7.93*	30%	*8.47*	32%	*8.08*	31%	*9.01*	34%	*11.16*	43%	> 2060
GR	MH02-402	MH02-401	284	48	24.55	*11.97*	49%	*13.61*	55%	*14.43*	59%	*13.42*	55%	*15.25*	62%	*18.24*	74%	> 2060
GR	MH02-401	PS2	30	48	37.12	*12.83*	35%	*14.45*	39%	*15.25*	41%	*14.10*	38%	*16.04*	43%	*19.14*	52%	> 2060

Table A8-5: Comparison of Flows in Old West Interceptor on University Avenue

Section		Capacity (mgd)	Scenario	CARPC Projected Peak Flows (mgd): 2010 UF	2030 UF	2060 UF
MH02-041	MH02-038	2.71	Existing WI	1.67	2.20	2.93
			OWI Diversion	0.77	0.94	1.09
MH02-038	MH02-034	1.92	Existing WI	1.67	2.20	2.93
			OWI Diversion	0.77	0.94	1.09
MH02-034	MH02-032	2.84	Existing WI	2.76	3.47	4.28
			OWI Diversion	1.87	2.21	2.49
MH02-032	MH02-513	3.24	Existing WI	2.76	3.47	4.28
			OWI Diversion	1.87	2.21	2.49

2.93 Denotes capacity exceeded in section for specified time increment



PUMPING STATION 8 SUB-BASINS

MADISON METROPOLITAN SEWERAGE DISTRICT

COLLECTION SYSTEM EVALUATION

2008

Figure A8-1

the request to allow direct connections is feasible given that the sewer will act more like a local sewer with regards to flowrate. Additionally, since this section of the OWI will serve only City of Madison customers, it may make sense for the District to transfer ownership of this sewer to the City of Madison upon completion of the flow diversion project.

Regent Street Section

CARPC's analysis in Table A8-4 shows that capacity in the 24" section between S. Orchard Street and N. Murray Street was exceeded in the year 2000. This section of cast iron sewer was constructed in 1916 by the City of Madison and transferred to MMSD in 1933. The analysis for this system assumes that all flow from subbasin 2-B flows into the OWI at MH02-011 (see Figure A8-2). Basin 2-B comprises much of the flow from the west side of downtown Madison and is estimated to be 1.60 mgd for 2010 UF flows. The effect of inputting all of the flow from subbasin 2-B into MH02-011 is shown in Figure A8-3. The capacity in all segments downstream of MH02-011 in the OWI would be exceeded for this assumption.

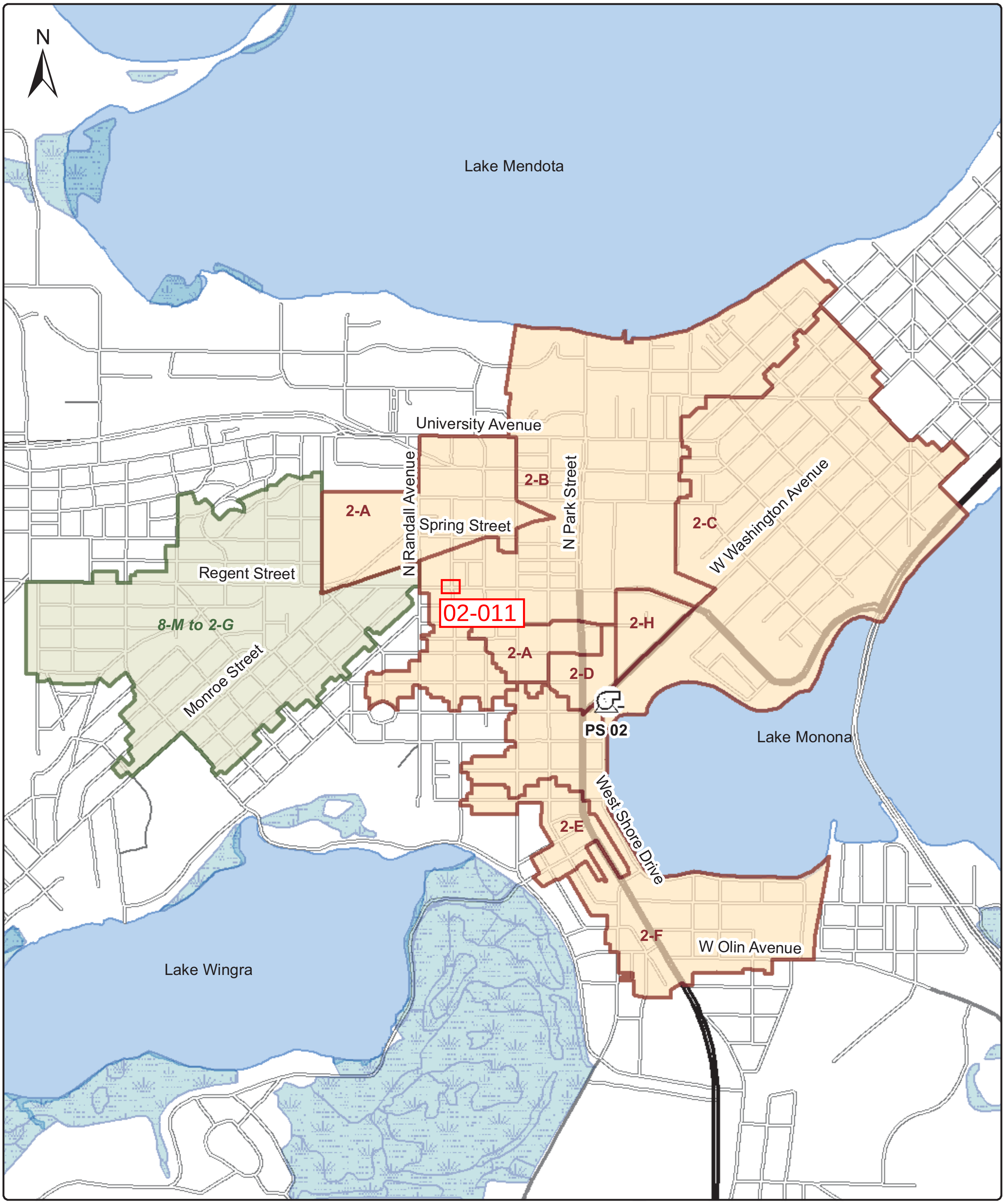
In looking at the City of Madison's sanitary sewer records, however, subbasin 2-B discharges to MMSD's West Interceptor primarily in two locations: (1). An 18" sewer on N. Park Street (MH02-006A); and (2). A 30" sewer along the southerly extension of East Campus Mall (MH02-402). Using a rough assumption that the total flow from subbasin 2-B is apportioned equally to these two discharge points, a revised analysis shows that capacity is not currently exceeded in the OWI on Regent Street (see Figure A8-4).

Previous inspection of this sewer section has revealed severe mineral deposits and tuberculation along its entire length. Therefore, the diameter and capacity of this sewer section may be somewhat smaller than the values indicated in Table A8-4 due to the deteriorated pipe condition. If it were assumed that mineral deposits had built up to a depth of one inch around the circumference of the 24" pipe, the capacity from N. Mills Street to N. Murray Street would be reduced from approximately 5.27 mgd to 4.19 mgd. A brief summary of revised flowrates for different time periods and diameters of the OWI are shown in Table A8-6.

Table A8-6: Revised Flowrate Analysis for Old West Interceptor on Regent Street

				Peak Flowrates (mgd)		
From	**To**	**Pipe diameter (in)**	**Pipe Capacity (mgd)**	**2010 UF**	**2030 UF**	**2060 UF**
MH02-012	MH02-008	24	4.62	1.36	1.68	2.06
		22	3.68			
MH02-008	MH02-005A	24	5.27	4.47	4.98	5.76
		22	4.19			
MH02-005A	MH02-402	30	12.43	6.95	7.69	8.85

12/03/2008



- Sub-Basins in 2000
- Added in 2003
- Potential by 2060
- DNR Wetlands

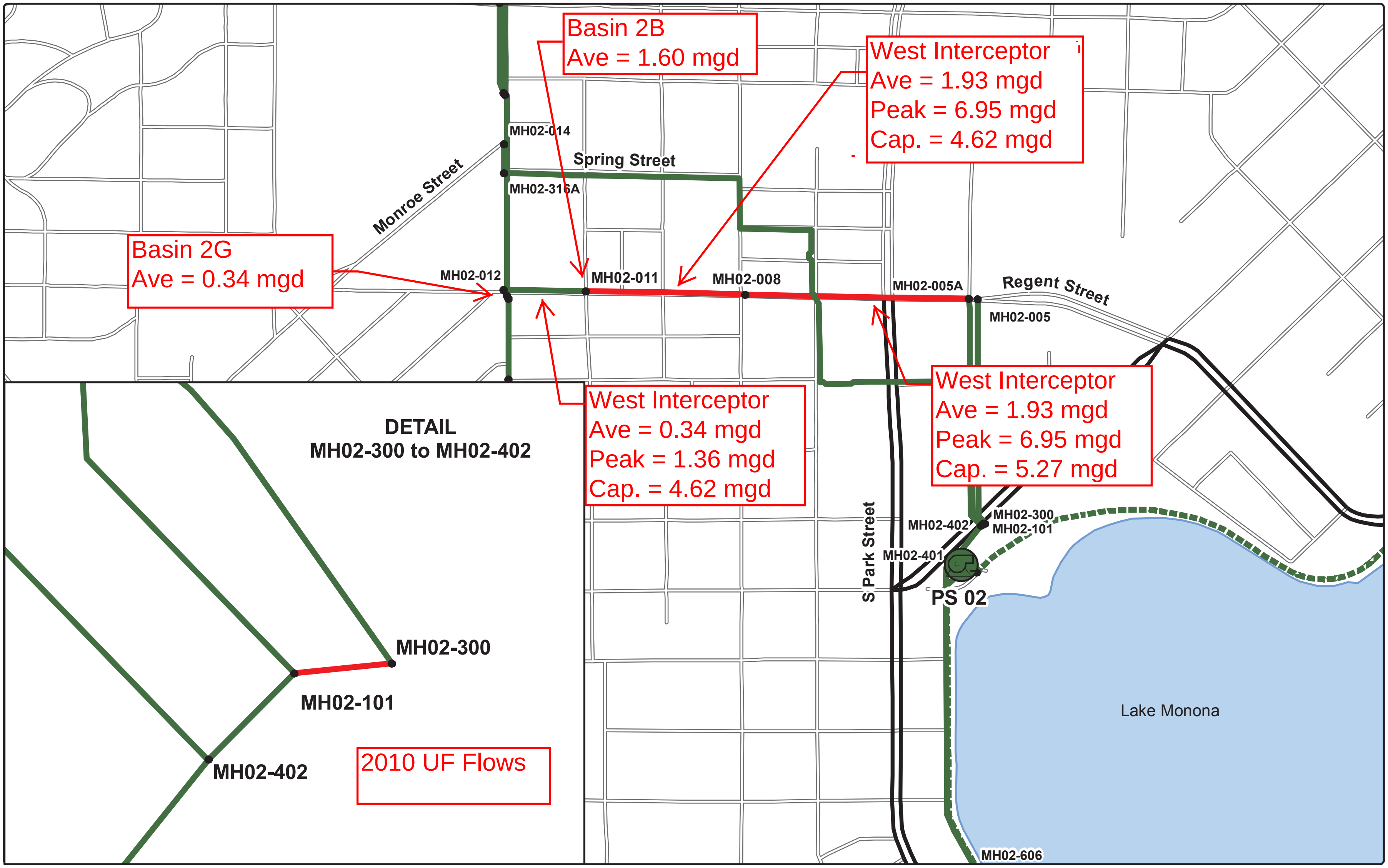
PUMPING STATION 2 SUB-BASINS

MADISON METROPOLITAN SEWERAGE DISTRICT
COLLECTION SYSTEM EVALUATION
2008

0 0.25 0.5 1 Miles

Figure A8-2

12/23/2008



Capacity Reached

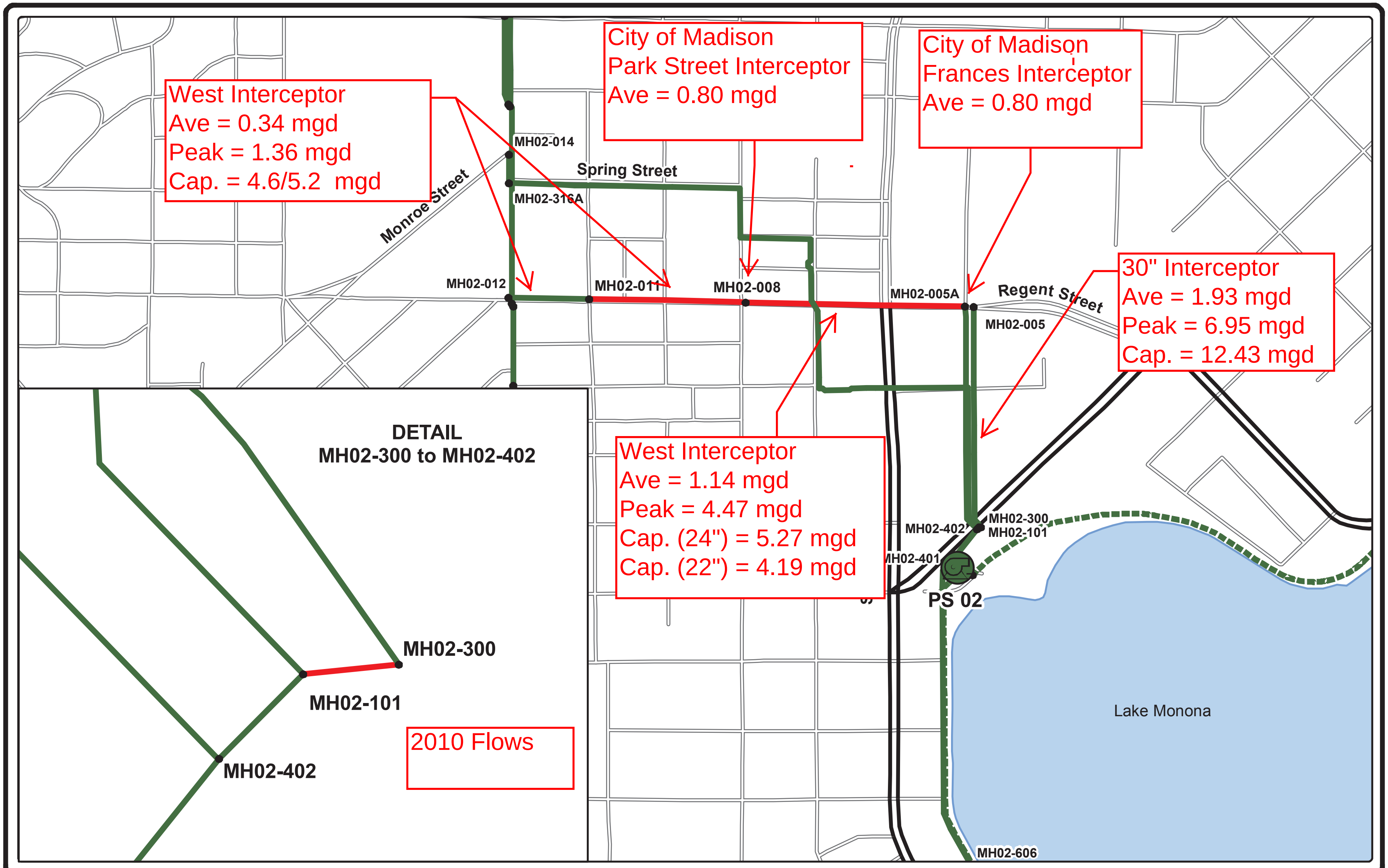
- > 2060
- 2030 - 2060
- 2020 - 2030
- 2010 - 2020
- 2000 - 2010

SPRING STREET RELIEF AND OLD WEST INTERCEPTOR

MADISON METROPOLITAN SEWERAGE DISTRICT
COLLECTION SYSTEM EVALUATION
2008

Figure A8-3

12/23/2008



Capacity Reached

- > 2060
- 2030 - 2060
- 2020 - 2030
- 2010 - 2020
- 2000 - 2010

SPRING STREET RELIEF AND OLD WEST INTERCEPTOR

MADISON METROPOLITAN SEWERAGE DISTRICT
COLLECTION SYSTEM EVALUATION
2008

REVISED ANALYSIS

Figure A8-4

Table A8-6 shows that the section from MH02-012 to MH02-008 has adequate capacity until 2060 for a 24" sewer and a 22" sewer in a deteriorated condition. The section from MH02-008 to MH02-005A should have adequate capacity to convey flows up to the year 2030, but may not have sufficient capacity if a deteriorated 22" pipe is assumed. A more thorough flow analysis is required to assess capacity needs in this section beyond 2030, however. No capacity upgrades are anticipated for the section from MH02-005A to MH02-402 prior to 2060.

In summary, it does not appear that the Old West Interceptor on Regent Street has imminent capacity needs as suggested by CARPC's *Collection System Evaluation.* While it appears that adequate capacity exists at this time, a more detailed study of this system should be performed. This study should include a more thorough analysis of the flow distribution between the City of Madison's N. Park Street and Frances Street Interceptors and a television inspection of the OWI to verify pipe condition and actual carrying capacity.

Midvale Relief

CARPC's analysis shows that capacity in the Midvale Relief will be reached sometime between 2020 and 2030 (Table A8-7). This 21" sewer is approximately 2,650 feet in length and extends along University Avenue from Shorewood Boulevard to Midvale Boulevard. Hydraulic modeling of this sewer section demonstrates that the water surface elevation is impacted by downstream conditions in the West Relief Interceptor. With the West Interceptor Relief flowing nearly full, the surcharge depth on segments in the Midvale Relief is modeled to be approximately one to two feet for CARPC's 2010 peak flow estimates (Appendix A8-5). In looking at Appendix A8-5, however, it should be noted that the hydraulic grade line for the Midvale Relief is below the elevation for most of the local sewers along its length.

Much of the surcharging problem can be attributed to the elevation at which the 21" Midvale Relief sewer connects to the 36" West Interceptor Relief sewer at MH02-531A. Normally in cases where sewers of different diameters connect the elevations would be set such that the crowns of the sewers are at the same elevation. In this instance, the sewer inverts are at the same elevation at the connection point.

This causes the smaller sewer to surcharge when the larger sewer is flowing full. Unfortunately there is no opportunity to match crowns at the connection point for these two sewers as the West Interceptor Relief sewer cannot be lowered any further between Shorewood Boulevard and Walnut Street. The surcharging situation is much improved in the Midvale Relief sewer with the addition of a new 36" relief sewer for the West Interceptor Relief system (Appendix A8-6). In this scenario there is little to no surcharging in the Midvale Relief for CARPC's 2010 peak flow projections. With only modest growth expected in the Midvale Relief basin until year 2060, the modeled surcharge is approximately one foot for 2060 flows with a new relief sewer for the West Interceptor Relief.

Table A8-7: West Interceptor – Midvale Relief

Flow Type	From	To	Length (ft)	Pipe Dia. (in)	Nominal Capacity (mgd)	*Peak Flows (mgd)* / Percent Nominal Capacity												Capacity Reached
						2000		2010 UF		2020 UF		2030 TAZ		2030 UF		2060 UF		
GR	MH02-531I	MH02-531A	2,653	21	3.55	*3.19*	90%	*3.32*	94%	*3.44*	97%	*3.16*	89%	*3.57*	101%	3.88	109%	**2020-2030**

Table A8-8: West Interceptor – Spring Street Relief

Flow Type	From	To	Length (ft)	Pipe Dia. (in)	Nominal Capacity (mgd)	*Peak Flows (mgd)* / Percent Nominal Capacity												Capacity Reached
						2000		2010 UF		2020 UF		2030 TAZ		2030 UF		2060 UF		
GR	MH02-014	MH02-316A	150	24	7.73	*2.23*	29%	*2.22*	29%	*2.20*	28%	*2.05*	27%	2.19	28%	*2.35*	30%	> 2060
GR	MH02-316A	MH02-300	4,577	24	6.54	*2.23*	34%	*2.22*	34%	*2.20*	34%	*2.05*	31%	2.19	33%	*2.35*	36%	> 2060
GR	MH02-300	MH02-101	3	24	6.54	*7.20*	110%	*7.76*	119%	*8.31*	127%	*7.92*	121%	8.86	135%	*11.01*	168%	**2000**

Table A8-8(1): West Interceptor – Spring Street Relief (Revised)[(1)]

Flow Type	From	To	Length (ft)	Pipe Dia. (in)	Nominal Capacity (mgd)	*Peak Flows (mgd)* / Percent Nominal Capacity												Capacity Reached
						2000		2010 UF		2020 UF		2030 TAZ		2030 UF		2060 UF		
GR	MH02-014	MH02-316A	150	24	7.73	*2.23*	29%	*2.22*	29%	*2.20*	28%	*2.05*	27%	2.19	28%	*2.35*	30%	> 2060
GR	MH02-316A	MH02-300	4,577	24	6.54	*2.23*	34%	*3.60*	55%	*3.58*	55%	*3.43*	52%	3.57	55%	*3.73*	57%	> 2060
GR	MH02-300	MH02-101	3	24	6.54	*7.20*	110%	*9.14*	140%	*9.69*	148%	*9.3*	142%	10.24	157%	*12.39*	189%	**2000**

Notes: (1). Includes intermittent peak wet weather flow from UW Charter Street Heating Plant. This permitted flow expected to cease in 2011-12.

Spring Street Relief

The Spring Street Relief was constructed in 1940 to provide relief for the West Interceptor on the near west side of the City of Madison. It extends from Pumping Station No. 2, travels through the Regent Street area, and connects to the OWI at the intersection of Spring Street and Randall Avenue. Per Table A8-8, adequate capacity exists in all segments of this relief sewer through the year 2060 except for a three-foot segment of 24" sewer just upstream of PS 2. Hydraulic modeling of this short segment of sewer shows negligible surcharge for 2010 flows and does not indicate a need or benefit to replacing this section in the near term (see Appendix A8-7).

The Spring Street Relief sewer receives flow from several unique sources on the UW campus, including Camp Randall stadium and the UW heating plant on Charter Street. The average daily flow from Camp Randall in 2000 was 41,016 gallons per day according to City of Madison Water Utility data. However, a peak instantaneous flowrate of 1.43 mgd was used for the design of the restrooms at the stadium. While it is unlikely that the peak flow from the stadium actually reaches this value, Table A8-8 suggests that the Spring Street Relief has adequate capacity to accommodate the flow if necessary.

In 2007 MMSD granted permission to representatives of the UW's Charter Street heating plant for a discharge of up to 1.38 mgd into the Spring Street Relief sewer at MH02-312A. The discharge is comprised primarily of stormwater runoff from an area surrounding the plant's coal unloading station. The dust created from the unloading operation is considered unsuitable for discharge into the public stormwater system. As shown in the revised analysis of system capacity in Table A8-8(1), this additional flow does not have an appreciable effect for much of the Spring Street Relief sewer. The University intends to cease the burning of coal at the plant in 2011 or 2012 and switch to natural gas as its primary fuel. It is expected that the District's permit to allow stormwater into the Spring Street Relief sewer will expire with the transition to a new fuel source.

Randall Relief

The Randall Relief was constructed in 1964 from Pumping Station No. 8 to the intersection of Dayton Street and Randall Avenue. CARPC's evaluation indicates that capacity is adequate for all sections of this interceptor through the year 2060 (Table A8-9). A small exceedance in capacity is projected for two 30" sewers passing underneath a City of Madison storm box at the intersection of Regent Street and Randall Avenue, although it is relatively minor and should not be a cause for concern at this time.

Table A8-9: West Interceptor - Randall Relief to PS 8

Flow Type	From	To	Length (ft)	Pipe Dia. (in)	Nominal Capacity (mgd)	*Peak Flows (mgd)* / Percent Nominal Capacity 2000		2010 UF		2020 UF		2030 TAZ		2030 UF		2060 UF		Capacity Reached
GR	MH02-014A	MH08-201	29	33	25.10	*7.97*	32%	*8.02*	32%	*8.08*	32%	*7.71*	31%	*8.15*	32%	*8.56*	34%	> 2060
GR	MH08-201	MH08-121	1,127	33	25.10	*19.93*	79%	*20.45*	81%	*21.02*	84%	*19.83*	79%	*21.58*	86%	*23.23*	93%	> 2060
GR	MH08-121	MH08-120	16	2@30	21.13	*19.93*	94%	*20.45*	97%	*21.02*	99%	*19.83*	94%	*21.58*	102%	*23.23*	110%	**2020-2030**
GR	MH08-120	MH08-119	473	42	25.17	*19.93*	79%	*20.45*	81%	*21.02*	84%	*19.83*	79%	*21.58*	86%	*23.23*	92%	> 2060
GR	MH08-119	MH08-117	1,201	42	25.17	*20.67*	82%	*20.45*	81%	*21.02*	84%	*19.83*	79%	*21.58*	86%	*23.23*	92%	> 2060
GR	MH08-117	MH08-113	1,479	42	25.17	*20.93*	83%	*20.70*	82%	*21.27*	85%	*20.08*	80%	*21.83*	87%	*23.48*	93%	> 2060
GR	MH08-113	MH08-109	1,237	48	27.84	*20.75*	75%	*20.61*	74%	*21.12*	76%	*20.01*	72%	*21.63*	78%	*23.22*	83%	> 2060
GR	MH08-109	MH08-106	1,279	48	27.84	*21.07*	76%	*20.94*	75%	*21.45*	77%	*20.34*	73%	*21.96*	79%	*23.54*	85%	> 2060
GR	MH08-106	PS 8	3,179	48	30.78	*24.90*	81%	*24.74*	80%	*25.34*	82%	*24.04*	78%	*25.94*	84%	*27.80*	90%	> 2060
FM	PS 8	RD08-13205	194	36	36.50	*25.13*	69%	*24.97*	68%	*25.57*	70%	*24.27*	66%	*26.17*	72%	*28.02*	77%	> 2060
FM	RD08-13205	WWTP	13,508	42	49.70	*25.13*	51%	*24.97*	50%	*25.57*	51%	*24.27*	49%	*26.17*	53%	*28.02*	56%	> 2060

Table A8-10: West Interceptor - Campus Relief

Flow Type	From	To	Length (ft)	Pipe Dia. (in)	Nominal Capacity (mgd)	*Peak Flows (mgd)* / Percent Nominal Capacity 2000		2010 UF		2020 UF		2030 TAZ		2030 UF		2060 UF		Capacity Reached
GR	MH08-228	MH08-223	1,933	36	15.04	*7.53*	50%	*7.77*	52%	*8.04*	53%	*7.66*	51%	*8.30*	55%	*8.88*	59%	> 2060
GR	MH08-223	MH08-221	161	36	15.04	*9.69*	64%	*9.90*	66%	*10.15*	67%	*9.70*	64%	*10.39*	69%	*11.01*	73%	> 2060
GR	MH08-221	MH08-220	118	2 @ 24	15.64	*9.69*	62%	*9.90*	63%	*10.15*	65%	*9.70*	62%	*10.39*	66%	*11.01*	70%	> 2060
GR	MH08-220	MH08-216	514	36	15.04	*9.69*	64%	*9.90*	66%	*10.15*	67%	*9.70*	64%	*10.39*	69%	*11.01*	73%	> 2060
GR	MH08-216	MH08-210	1,051	36	16.40	*9.69*	59%	*9.90*	60%	*10.15*	62%	*9.70*	59%	*10.39*	63%	*11.01*	67%	> 2060
GR	MH08-210	MH08-209	64	36	15.04	*9.69*	64%	*9.90*	66%	*10.15*	67%	*9.70*	64%	*10.39*	69%	*11.01*	73%	> 2060
GR	MH08-209	MH08-208	629	48	34.68	*9.52*	27%	*9.87*	28%	*10.25*	30%	*9.62*	28%	*10.63*	31%	*11.51*	33%	> 2060
GR	MH08-208	MH08-207	12	36	15.04	*9.52*	63%	*9.87*	66%	*10.25*	68%	*9.62*	64%	*10.63*	71%	*11.51*	77%	> 2060
GR	MH08-207	MH08-201	1,234	36	17.80	*13.64*	77%	*14.13*	79%	*14.66*	82%	*13.77*	77%	*15.18*	85%	*16.54*	93%	> 2060

City of Madison storm box at the intersection of Regent Street and Randall Avenue, although it is relatively minor and should not be a cause for concern at this time.

Campus Relief

The Campus Relief project was completed in four construction phases, beginning in 1999 and ending in 2004. The project added additional capacity to the West Intercepting system through the UW campus area from the intersection of Dayton Street and Randall Avenue to the intersection of Campus Drive and Walnut Street. As shown in Table A8-10, adequate capacity is available in this interceptor system through the year 2060.

Conclusions and Recommendations

The West Intercepting System is a complex network of parallel and interconnected sewers that has been constructed and continually updated to provide sewer service to the west side of the City of Madison and surrounding communities. According to CARPC's *2009 Collection System Capacity Evaluation* and analysis by District staff, adequate capacity is sufficient in several portions of the system through 2060, including:

- WI – Spring Street Relief
- WI – Randall Relief
- WI – Campus Relief

Other portions of the system require additional capacity prior to 2060. The following recommendations provided in Table A8-10 are a general guideline for improvements needed for the West Intercepting System within the next twenty years.

Table A8-10: Summary of Improvements for West Side Conveyance System

Facility	Limits		Improvements	Timeline
	From	To		
West Interceptor Relief	Walnut Street (MH02-517)	Whitney Way (MH02-547)	Construct 36" (or 42") interceptor parallel to existing interceptor	2015-2020
Old West Interceptor	Grand Avenue (MH02-037)	Forest Street (MH02-030)	Rehabilitate aging 18"-21" VP with cured-in-place liner	2011-2012
Old West Interceptor	University Ave & Ridge St (MH02-043)		Divert flow from old West Interceptor into West Interceptor Relief Sewer system (existing 36" WI Relief parallel to future 36" relief sewer)	2015-2020

Appendix A8-1

MADISON METROPOLITAN SEWERAGE DISTRICT
1610 MOORLAND ROAD
MADISON, WI 53713-3398
PHONE (608) 222-1201
FAX (608) 222-2703

MEMO

DATE: 7/16/08
TO: Bruce Borelli, DOE
From: Gerald Sachs, Municipal Engineer
RE: Collection System Evaluation-West Interceptor System Capacity

This Memo is a follow up to discussions regarding the capacity analysis of the West Interceptor System. The West Intercepting System consists of parallel and connecting interceptors built over time to serve the west side of the District. These interceptors extend from Pumping Stations 2 and 8 westward to Pumping Station 15 and are comprised of nine separate projects: West Interceptor-1916, West Interceptor Relief-1959, Randall Relief-1962, Midvale Relief-1971, Spring Harbor Relief-1972, and the four Campus Relief projects built from 1999 to 2004.

The interceptors, while parallel and connected in various locations, are not totally interconnecting allowing free flow from one to another. The system is best described as two parallel interceptors at different elevations with parallel interconnecting legs.
The original West Interceptor extends from Pumping Station 2 at Brittingham Park westerly along Regent St, Randall Ave, and University Ave ending at Shorewood Blvd. The Randall Relief extends northerly from Pumping Station 8 intersecting the West Interceptor in Randall Ave at MH02-014A. The West Interceptor Relief joins both the West Interceptor and Randall Relief at MH02-014A and extends northerly up Randall Ave, westerly along University Ave and the railroad corridor through Shorewood Hills to Whitney Way, then to Pumping Station 5. The Midvale Relief joins the West Interceptor Relief at MH02-531A in Shorewood Blvd. and extends west along University Ave to Midvale Blvd. The Spring Harbor Relief joins the West Interceptor Relief at the end of the Pumping Station 5 force main and extends westerly along University Ave, then north along Allen Blvd. to Pumping Station 15.

Summary:
An analysis of the interceptors that comprise the West Intercepting System identifying the flow diversion points, free flow connection points and cross connection points indicates the following:
All flow into Spring Harbor Relief, West Interceptor Relief, Midvale Relief, Campus Relief and West Interceptor upstream of manhole MH02-014A flows to Pumping Station 8 through the Randall Relief.
All flow into the West Interceptor downstream of manhole MH02-014A flows to Pumping Station 2 through the Spring Street Relief, West Interceptor and City of Madison's Francis Street Interceptor. See attached copy of connection points.

Connection Points

The following is a list of points where the interceptors either join or connect and comments relative to the direction of the flow.

1. Flow Diversion Points

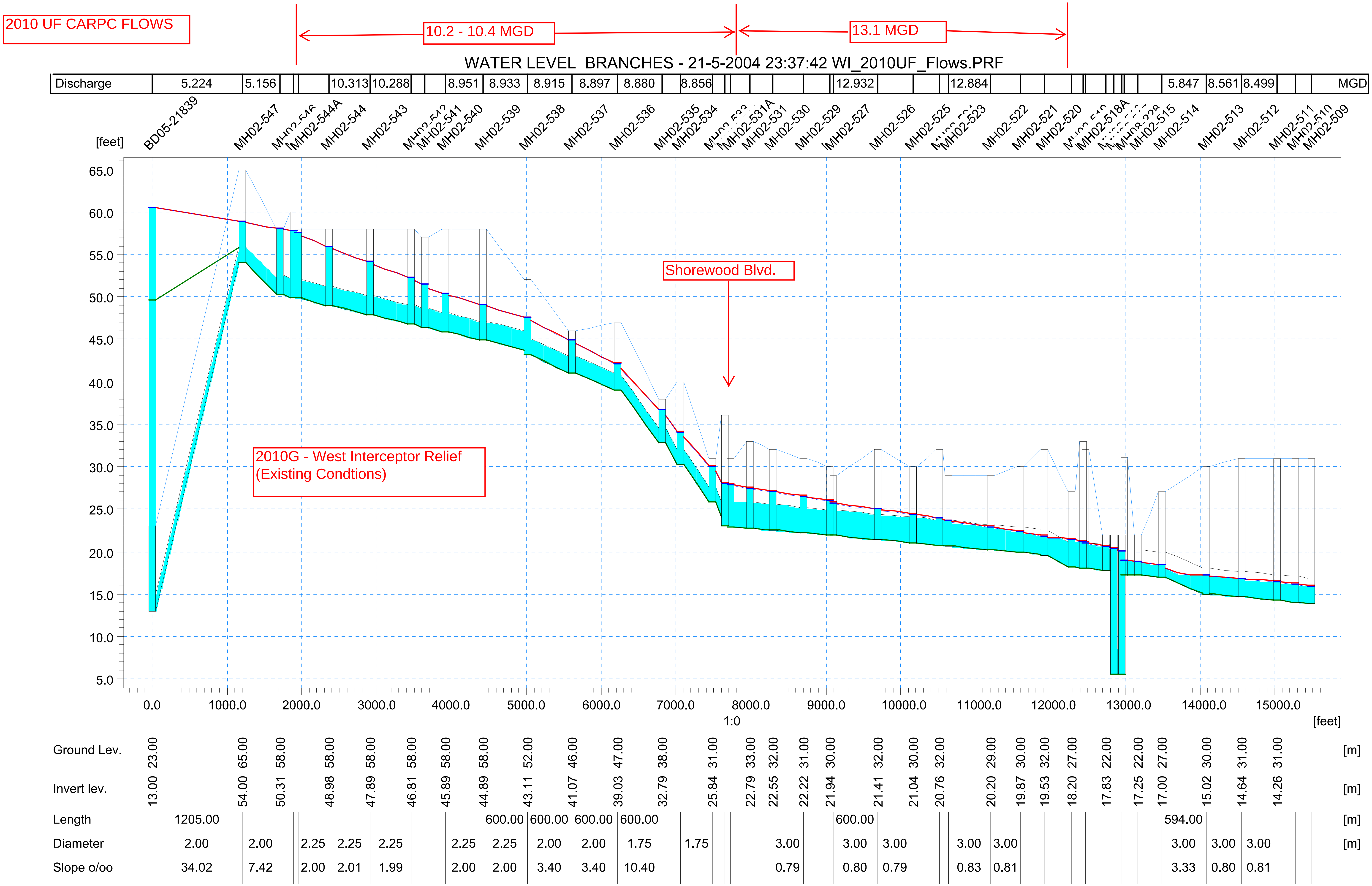
MH08-122/02-012B, Slide gate in MH08-122 allows flow to cross over into West Interceptor when removed.
MH02-316, Flow from WI drops into Spring Street Relief to PS2
MH02-014A, Slide gate in manhole forces flow from WI, WI Relief and Campus Relief into Randall Relief to PS8.
MH08-210, Flow from Campus Relief directed south to junction manhole MH08-209 between WI Relief and Campus Relief
MH02-513, Flow from WI along University Ave directed into WI Relief
MH02-531A, Flow from Midvale Relief into WI Relief
MH15-01360, Valve in manhole directs flow from PS15 into West Intercepting System

2. Free Flow Connection Points

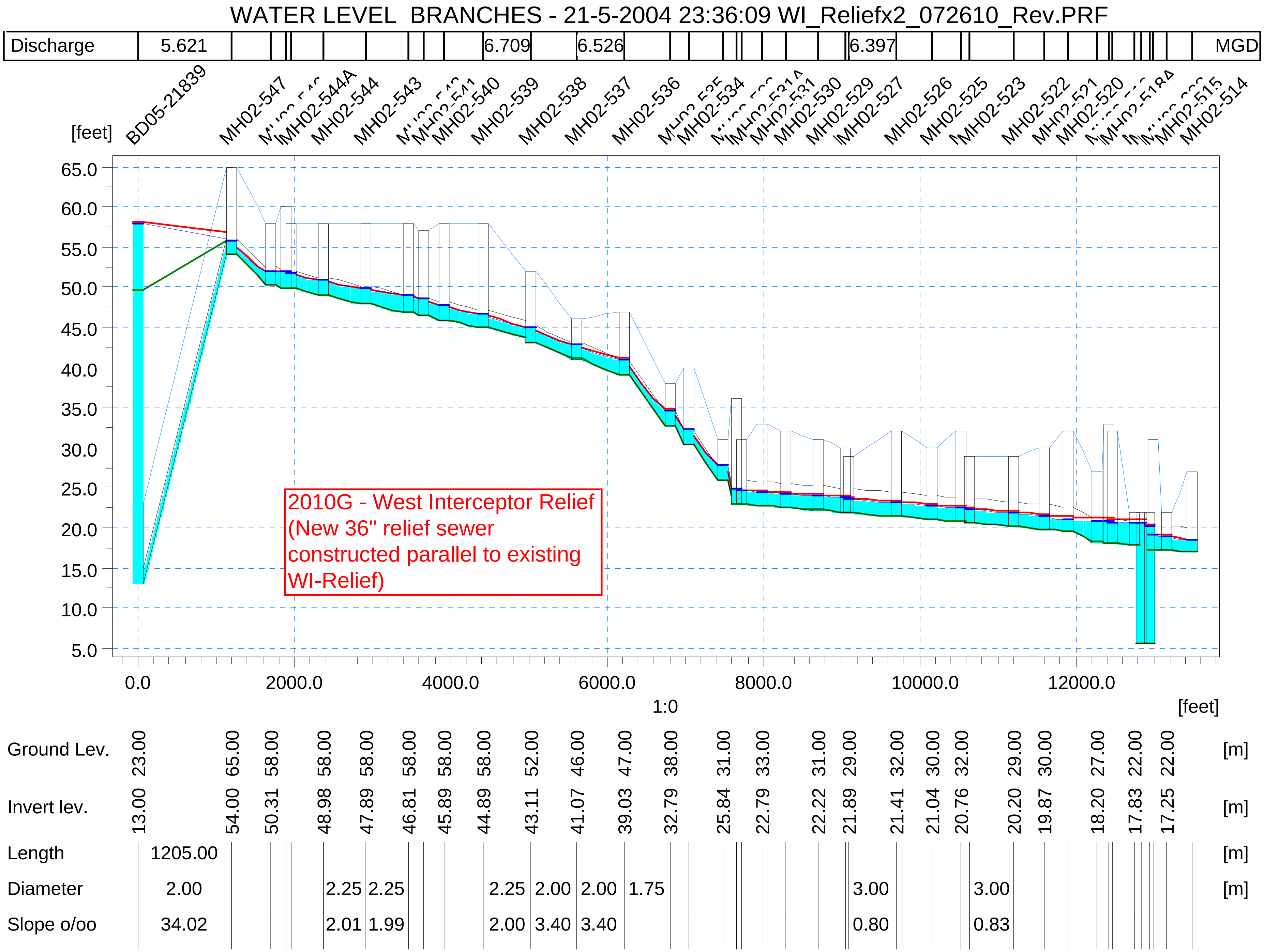
MH08-206, Free flow from WI in Campus area to Campus Relief
MH08-207, Free flow between WI Relief and Campus Relief
MH08-209, Free flow between WI Relief and Campus Relief
MH08-228, Free Flow between WI Relief and Campus Relief

3. Cross Connection Points

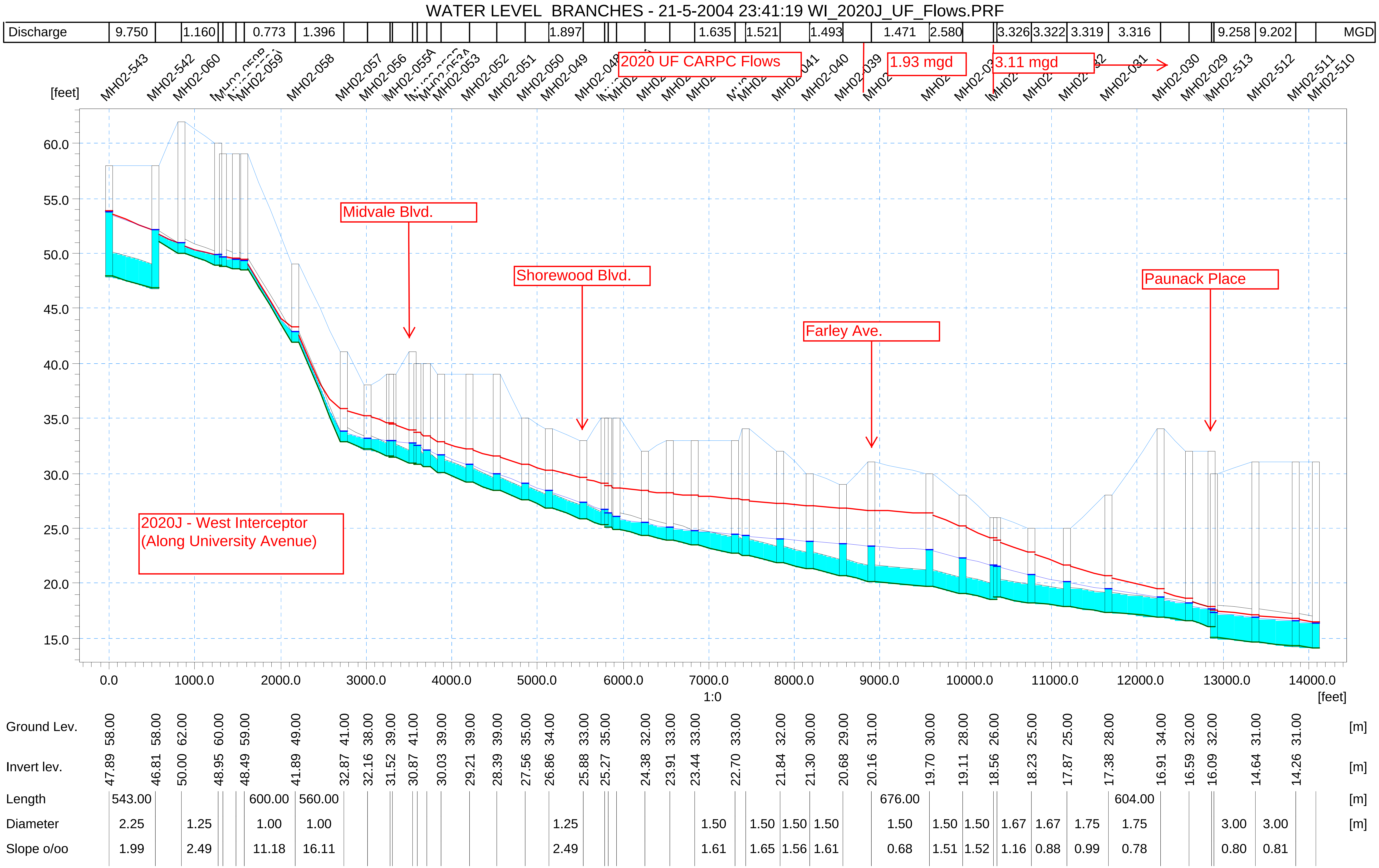
MH02-530/02-045, 8” Shorewood sewer between manholes. Cross flow will occur when WI Relief is surcharged ~1”, (El. ~26.2).
MH02-531/02-046, 12” and 10” Shorewood sewers between manholes. Cross flow will occur when WI Relief is surcharged ~1”, (El. ~26.8).
MH02-532/02-047, 12” Shorewood sewer between manholes. Cross flow will occur when WI Relief is surcharged ~1’, (El.27.0).
MH02-531I/02-054A, 12” City sewer between manholes. Cross flow will occur when Midvale is surcharged ~4-1/2’ or when WI is full, (El. Midvale MH02-531I is 26.2. El. WI MH02-054A is 30.8 cross connected by a City 12” EL. ~31.0). Siphon just downstream in WI can cause flow to be diverted over into Midvale Relief if surcharged.
MH02-542, Junction of WI Relief and WI. The WI upstream of this manhole is abandoned and does not exist. Cross flow from the WI Relief to the WI will occur when the WI Relief is surcharged ~5’, (WI Relief El. ~46.8 and WI El. ~52.0).



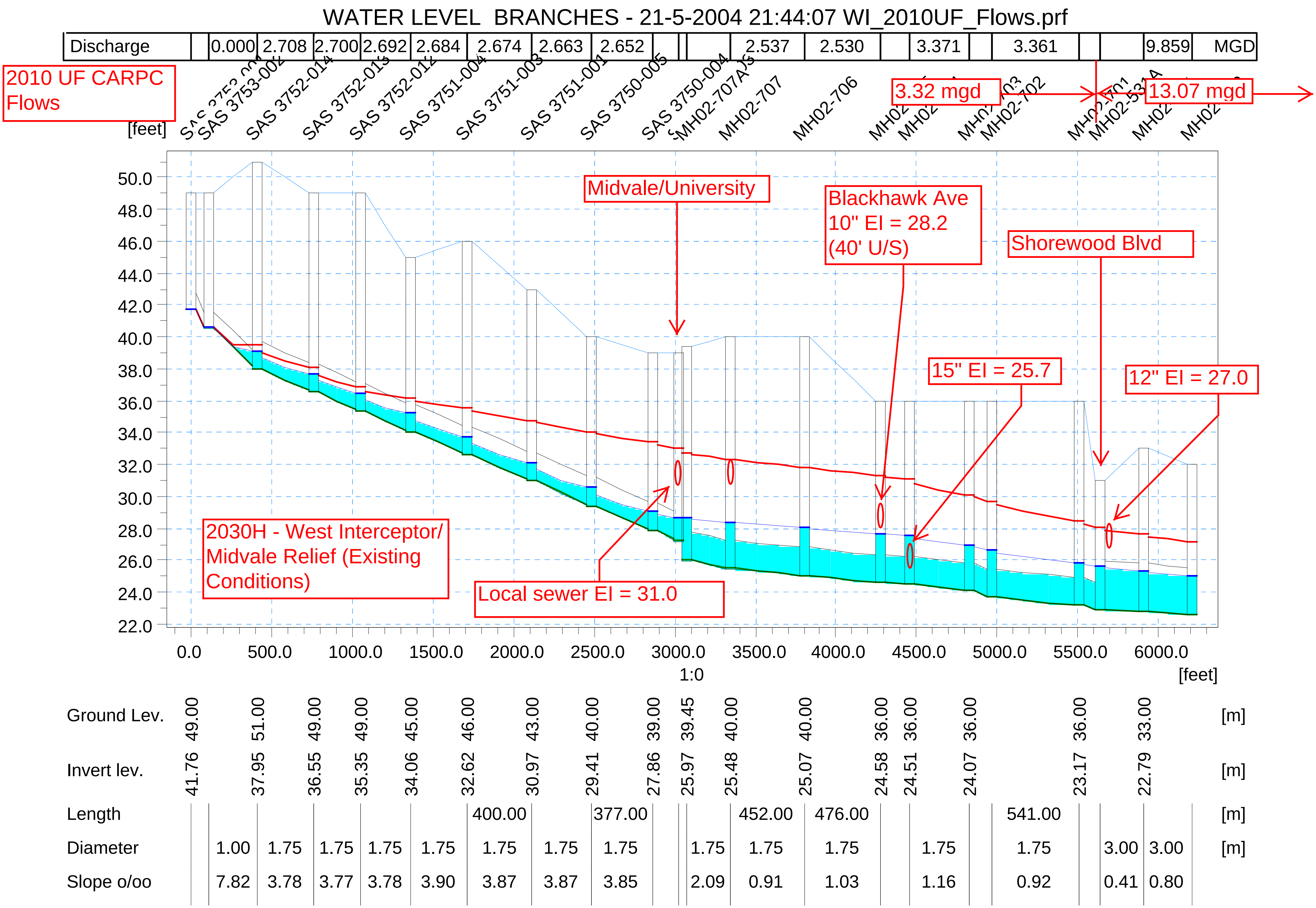
APPENDIX A8-2



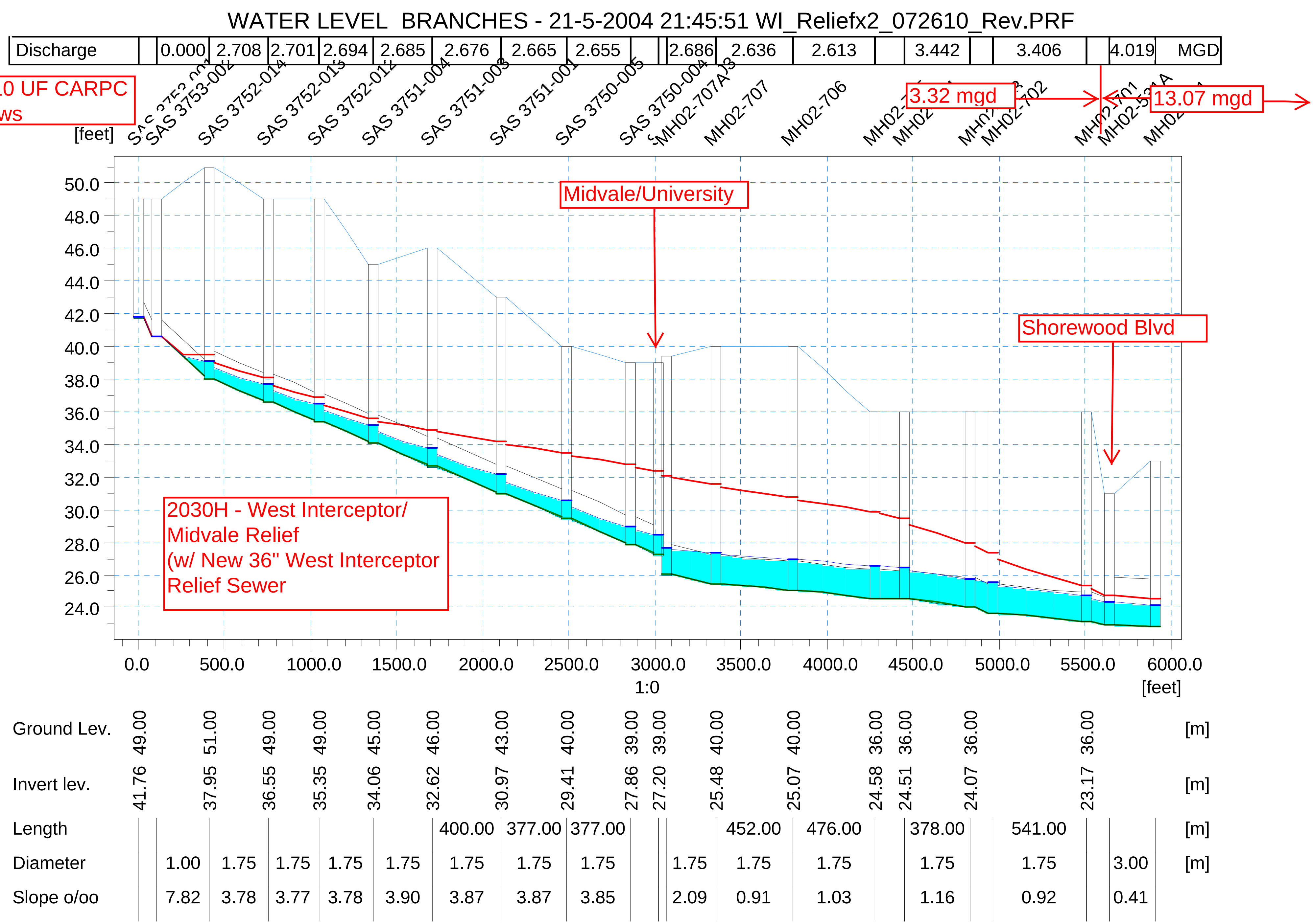
Appendix A8-3



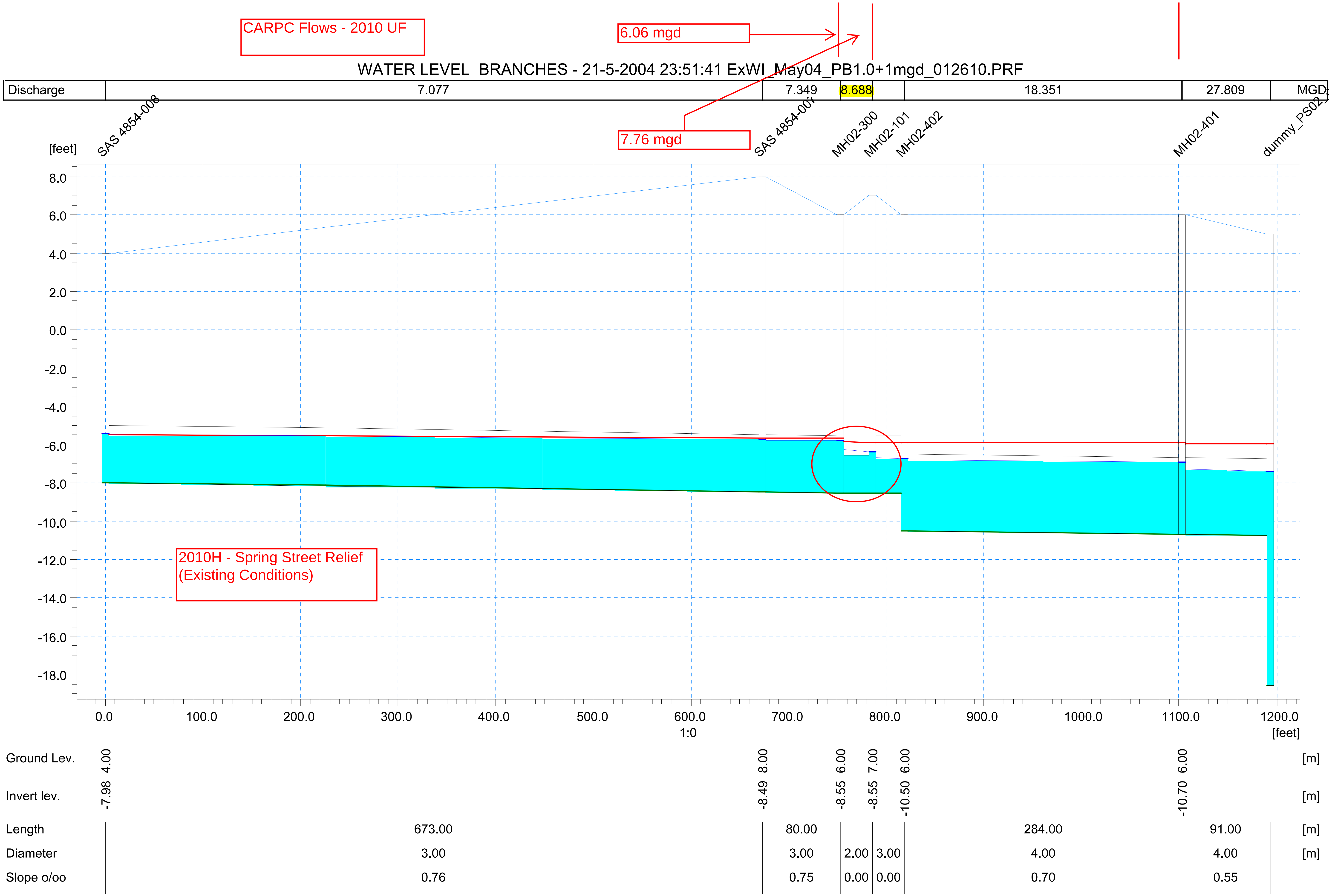
Appendix A8-4



Appendix A8-5



Appendix A8-6



Appendix A8-7

Appendix A9

Pumping Station 18 Feasibility Study, August 2010

Appendix A9
Pumping Station 18 Feasibility Study
Madison Metropolitan Sewerage District
August, 2010

Outline

This study is organized into the following sections:

- Introduction
- Background and History
- Purpose of Study
- Preliminary Design Flows and System Needs
- Siting of Pumping Station 18 and Related Improvements
- Schedule for Improvements
- Alternative Design Concepts for Pumping Station 18
- Peak Design Flow Assumptions for PS7 and New PS18
    - Madison Design Curve
    - Modified Madison Design Curve
- Discussion of Alternatives
    - Alternative 1
    - Alternative 2
    - Alternative 3
- Preliminary Sizing of PS18 Pumps and Force Main for Peak Flows
- Emergency Diversions
- Hydraulic Modeling of PS18
- Estimated Power Costs
- Summary and Recommendations

Introduction

CARPC's projected peak hourly flowrates at PS7 for 2030 and 2060 are 60 mgd and 72 mgd, respectively. The existing firm capacity at Station 7 is only 39 mgd, thus a major capacity upgrade would be required at this station to convey these future flowrates. Available space is limited in the pump room at PS7 and expansion at the site is not feasible. Additional conveyance would also need to be provided in the PS7 forcemain system and in the Southeast Interceptor from PS7 to its junction with the Northeast Interceptor. Relief or replacement for this section of the Southeast Interceptor would be very difficult and costly. Given these constraints and the District's preference not to convey such large flows through a single station with no diversion options, the District has proposed the construction of a new Pumping Station 18 to serve a portion of the Eastside collection system.

Background and History

A major feature of the *2002 Collection System Facilities Plan* was a study of the District's Crosstown Forcemain between Pumping Stations 1 and 2. Three alternative strategies for replacing or rehabilitating the old 20" forcemain were introduced: (1). Abandon the Crosstown forcemain and convey all flows from PS1 to PS6; (2). Reline the existing Crosstown forcemain to improve its condition and maintain 7.2 mgd of hydraulic relief from PS1 to PS2; and (3). Replace the Crosstown forcemain with a new 30" pipe and provide up to 21 mgd of hydraulic relief from PS1 to PS2. All three of the alternatives had implications with regard to relief of PS7 and the need and timing for a new PS18 as part of the Eastside collection system. Alternatives 1 and 2 required a new PS18 much sooner than Alternative 3.

The recommendation of the study was to replace and provide additional capacity in the Crosstown forcemain (Alternative 3). The District completed this project in 2003 and began pumping both average daily and peak flows from PS1 to PS2 at that time. This change in operation provided a considerable amount of relief in the Eastside collection system, primarily at PS6 and PS7.

Recognizing that hydraulic relief for PS7 was still needed within ten years of completion of this project, the District acquired land along East Broadway in the City of Monona in 2003 as a site for PS18. This vacant property is approximately 1.7 acres in area and is located near the intersection of the Southeast Interceptor and the Northeast Interceptor.

Purpose of Study

This study will explore the following issues related to the need, siting, timing, design, construction, and operation of a new PS18:

- Preliminary design flows and capacity of Eastside collection system
- Siting of pump station and routing of PS18 forcemain and Northeast Interceptor Relief sewer
- Timing of PS18 construction and related improvements
- Station capacity and alternative design concepts
- Emergency diversion with PS7

Each of these issues will be discussed at a general level as part of this study. It is anticipated that a detailed engineering study will be performed to further refine and expand on the ideas presented here.

Preliminary Design Flows and System Needs

The average flows used in this study are based on year 2010, 2030 and 2060 flow projections prepared by the Capital Area Regional Planning Commission (CARPC) as part of their 2009 report entitled *MMSD Collection System Evaluation.*

CARPC's report utilizes the "Madison Design Curve" (MDC) as a benchmark tool for determining the peak design capacity of the District's wastewater conveyance facilities. This curve was prepared for MMSD by the engineering consulting firm, Greeley and Hansen, in their *Report on Sewerage and Sewage Treatment* (1961) and has been standard MMSD design practice since that time. The Madison Design Curve is represented by the following formulas:

1. $Peaking\ Factor = 4 \Big/ \left(Q_{avg}\right)^{0.158}$ (Q in mgd)

   Note:
   - Peaking factor = 4.0 for $Q_{avg} \leq 1.0$ mgd
   - Peaking factor = 2.5 for $Q_{avg} \geq 20$ mgd

2. $Q_{peak} = 4 * \left(Q_{avg}\right)^{0.842}$ (Q in mgd)

The MDC provides a useful overall benchmark or reference for comparison of design flows. In general, it is considered by MMSD to be a reasonable design curve for a reasonably tight collection system.

Table 1 shows a summary of the projected flowrates and capacities for PS7 and related facilities over the next fifty years based on the Madison Design Curve and CARPC's report. Timing for the improvements is based on population estimates using high-growth rate (UF) and normal-growth rate (TAZ) scenarios. For purposes of this analysis the high-growth rate scenario is used. There is a near-term need to provide hydraulic relief at PS7 and in the Southeast Interceptor and Northeast Interceptor upstream of PS7. The firm capacity of 39 mgd at PS7 is currently exceeded and the maximum capacity of 45 mgd will be exceeded prior to 2030. Peak hourly flows in the 60" Southeast Interceptor immediately upstream of PS7 are approaching the nominal capacity of the interceptor. Similarly, peak flows in the 48" Northeast Interceptor are at or slightly greater than the nominal capacity of the interceptor.

Construction of a new PS18 and associated forcemain, with a capacity similar to PS7, will relieve the current capacity concerns at PS7. Likewise, a new PS18 will remove the need to provide additional capacity in the Southeast Interceptor as it is anticipated that all, or a significant portion, of the flow from the Northeast Interceptor will be intercepted by PS18. Construction of PS18 will not relieve the capacity shortfall in the Northeast Interceptor from MH07-215 to MH07-313. Additional capacity will have to be provided in that section and should be coordinated with the PS18 project.

Siting of Pumping Station 18 and Related Improvements

As mentioned previously, the District acquired land along East Broadway in the City of Monona in 2003 as a site for PS18. This vacant property is approximately 1.7 acres in area and is located near the intersection of the Southeast Interceptor and the Northeast Interceptor (see Figure 1 in attachments). Locating the pump station at this site will provide the opportunity to easily divert flows from the Northeast Interceptor to the new station. The site for the new station is also well situated to accept reverse flow from PS7 to PS18 along the Southeast Interceptor during high-flow events or emergency situations.

Table 1: Capacities and Projected Flowrates for PS7 and Related Facilities

Facility	Limits		Firm or Nominal Capacity (mgd)	CARPC Peak Hourly Flowrate by Year (mgd)						Capacity Reached	
	From	To		2000	2010 UF	2020 UF	2030 UF	2030 TAZ	2060	UF Estimate	TAZ Estimate
PS7	-	-	39.00	35.13	42.99	50.59	59.86	45.90	72.30	2005	2011
PS7 FM	PS7	NSWTP	55.00	35.13	42.99	50.59	59.86	45.90	72.30	2024	2032
SEI	PS7	MH07-211	37.62	30.09	38.01	45.63	53.01	40.74	65.62	2010	2021
SEI	MH07-211	MH07-215	37.62	29.44	37.33	44.93	52.28	40.10	64.74	2011	2023
NEI	MH07-215	MH07-313	32.14	26.75	33.21	39.44	45.50	35.94	53.68	2008	2018

Notes:
(1). TAZ = Traffic Analysis Zone
(2). UF = Uncertainity Factor
(3). SEI = Southeast Interceptor
(4). NEI = Northeast Interceptor

A new forcemain will need to be constructed from PS18 to the Nine Springs Wastewater Treatment Plant (NSWTP). The preliminary route for the new forcemain is shown in Figure 2 of the attachments . The forcemain is shown extending north from the new pumping station to East Broadway (Point A), at which point it turns to the west and travels along East Broadway approximately 6,500 feet (Point B). At this point the forcemain would shift direction and head southwest approximately 4,700 feet to the Wisconsin and Southern Railroad corridor (Point C). The stretch of forcemain from Point B to Point C would pass through the western edge of the wetlands surrounding Upper Mud Lake, just to the east of WPS Insurance and Business Park.

Alternate routes for the forcemain from Point B to Point C are problematic. An alternate route for consideration would be along West Broadway. This alternate route is challenging as West Broadway carries a high traffic volume, was reconstructed within the last ten years, and has a complex interchange with USH 12 & 18 that would make construction in this area expensive and disruptive to users of the transportation system.

From Point C the forcemain would extend to the west approximately 1,300 feet through lowlands to the NSWTP grounds (Point D). The forcemain would continue approximately 1,700 feet around the northerly and westerly boundaries of the plant grounds and connect to an existing 42" pipe (Point E) which was installed as part of the District's 10^{th} Addition project.

The other major project related to the construction of a new PS18 is capacity relief for the Northeast Interceptor from its junction with the Southeast Interceptor to its junction with the Far East Interceptor. The existing 48" sewer travels along Progress Road and Femrite Drive in the City of Madison and has several local main connections and direct lateral connections. Due to the number of these connections, it is thought that the existing sewer should remain in place and a relief sewer should be constructed to provide the additional capacity which is required. A preliminary route for the relief interceptor is shown in Figure 3 of the attachments.

The new interceptor would extend north from the new station to East Broadway and head east on East Broadway approximately 800 feet (Point A). From this point the interceptor would turn to the north approximately 1,500 feet along Copps Avenue to a stormwater drainage way (Point B). The interceptor would extend approximately 2,600 feet to the northeast from this point and parallel the drainage way, including a crossing of USH 51, until its junction with the Far East Interceptor at MH07-932 (Point C). It is anticipated that the existing and relief interceptors would have two or more junction structures along the route. Most of the proposed route for the relief interceptor is along paved roadways or adjacent to paved parking lots of existing businesses. Short wetland crossings would be needed for this route both west and east of USH 51. Crossing USH 51 at Femrite Drive is a possible option to avoid the wetlands crossing, but doing so may conflict with future interchange improvements that are being considered by the Wisconsin Department of Transportation at USH 12/18 and USH 51.

Schedule for Improvements

As can be seen in Table 1, there is a near-term need to provide hydraulic relief at PS7 and in the Southeast and Northeast Interceptors. Projects to provide relief for these facilities have been included in the District's ten year Capital Projects Budget. These projects include a new PS18, a new PS18 forcemain, and a relief sewer for the Northeast Interceptor. A preliminary schedule for the design, construction, and start-up of these facilities is summarized in Table 2.

It should be noted that a rehabilitation project at PS7 will be undertaken soon after the start-up of PS18. While the scope of this work has not been fully developed, it will likely include, at a minimum, installation of a full size impeller for Pump 7B and replacement of control panels.

Table 2: Schedule of Improvements for PS18 and Related Projects

Activity	Time Period
Prepare Request for Design Proposals (RFP)	Winter 2010
Mail RFP	February 2011
Notice to Proceed for engineering consultant(s)	April 2011
Pre-design report completed	Fall 2011
Detailed design	Fall 2011 to Fall 2012
Bid improvement projects	Winter 2012
Begin construction	Spring 2013
Completion of projects and start-up of facilities	Fall 2015

Alternative Design Concepts for Pumping Station 18

The following alternative design concepts will be considered for PS18 and the implications of each will be evaluated in turn.

Alternative 1: Station 18 would be sized to collect and pump all average daily and peak flows conveyed by the Northeast Interceptor upstream of its junction with the Southeast Interceptor. Under this scenario Station 7 would still receive flows from the Southeast Interceptor and East Interceptor (including Stations 6 and 9).

Alternative 2: Station 18 would be sized to collect and pump all average daily and peak flowrates conveyed by the Northeast Interceptor up to a maximum, pre-defined flowrate, such that peak flows would be split equally between PS7 and PS18. This alternative would require the installation of a flow splitting structure in the Northeast Interceptor near PS18 to divert flows in excess of the maximum PS18 flowrate to PS7.

Alternative 3: Station 18 would be used primarily to convey only average daily flows in the Northeast collection system. Flows in excess of the average daily flows from the Northeast Interceptor would be conveyed to Station 7, along with flows from Stations 6 and 9. As flows increase in the Northeast collection system over time, Station 18 would have to convey correspondingly greater flows to ensure that firm capacity at Station 7 is not exceeded.

Each of the alternatives has a direct effect on the pumping capacity and operational strategies at PS7. As such, one of the goals for each alternative should be to minimize or negate the need to provide additional capacity at PS7. While PS7 is in need of rehabilitation from a condition perspective, it would be desirable to not have to significantly increase the capacity at this station due to space constraints. In addition to providing hydraulic relief in the Eastside collection system, another goal for each of the alternatives should be to provide diversion capabilities and system flexibility. Thus, each station should be able to convey the 2060 average daily flowrate in the Eastside collection system, at a minimum, in the event of an outage at either of the stations.

Peak Design Flow Assumptions for PS7 and New PS18

Madison Design Curve

As mentioned previously in the section on Preliminary Flows and System Needs, CARPC has projected flowrates for the PS7 and PS18 service areas through the year 2060 based on the Madison Design Curve. Figures 4, 5 and 6 show these preliminary projected peak design flows for the years 2010, 2030, and 2060 for each of the three alternatives at key points in the collection system (see attachments).

The service areas for PS7 and all upstream pumping stations are shown in Figures 14-18 of the attachments. The development of peak hourly flows from average daily flows in these service areas for each pumping alternative are shown in Tables 8A-1, 8A-2, and 8A-3 of the attachments. Using the standard Madison Design Curve, the ultimate (Year 2060) peak capacity for PS18 for each alternative would be as follows:

- Alternative #1 = 54 mgd
- Alternative #2 = 37 mgd
- Alternative #3 = 22 mgd

Modified Madison Design Curve

Consideration should be given to the utilization of more conservative peaking factors for the long-term sizing of PS18. Several severe wet weather events in the past 10-15 years have stressed portions of the District's collection system. Recent investigation into the effects of climate change, as described in greater detail in Chapter 8, suggest that storms are becoming more intense and additional consideration needs to be given to the adjustment of peaking factors in service areas that are prone to inflow and infiltration, such as those for PS7 and PS18.

It is recommended that design peak hourly flows for PS18 be established from more conservative peaking factors. These conservative flow estimates should be used in establishing the ultimate footprint of the PS18 pump room and in sizing the associated suction and discharge piping so that the pumping capacity at the station is readily expandable and flexible in the event that actual future flowrates at PS18 are higher than estimated by the Madison Design Curve.

Currently the minimum peaking factor allowed by the Madison Design Curve is 2.5 and applies to average daily flowrates in excess of 20 mgd. To reinforce the Eastside Collection System and the new PS18 infrastructure improvements, it is proposed to modify the Madison Design Curve by restricting the minimum peaking factor to 3.0, rather than 2.5. Thus, the same formulas for computing peaking factors and peak hourly flows that were presented previously would still be used, but the minimum peaking factor would be limited to 3.0. The effect of this adjustment is that the ultimate capacity of PS18 would be increased from 54 mgd to 66 mgd under Alternative 1, an increase of approximately 22%. Figures 4A, 5A, and 6A show the distribution of peak hourly flows in the Eastside Collection System based on the modified peaking factors. It should be noted that the modifications shown in the figures apply only to the service areas upstream of PS7 and PS18. Calculations of the peak hourly flows using the modified peaking factors can be found in Tables 9A-1, 9A-2, and 9A-3 of the attachments.

Discussion of Alternatives

Alternative 1

Using the traditional Madison Design Curve, PS18 would have a peak pumping capacity of 54 mgd and would convey both average daily and peak hourly flows from the Northeast Interceptor. This capacity would be sufficient to serve flows in the Northeast Interceptor through year 2060. All flows from PS6, PS9, and the Southeast Interceptor (Blooming Grove Extension) would continue to be served by PS7 (Q_{avg} = 7.2 mgd; Q_{peak} = 21.0 mgd in 2060). Under the modified Madison Design Curve the ultimate peak pumping capacity would be increased to 66 mgd.

The primary advantage of this alternative is that it provides the greatest capacity and flexibility amongst the alternatives. PS18 would have significantly more pumping capacity than PS7 under this scenario. Thus it would provide excellent redundancy in the Eastside collection system in the event of a station outage at PS7. This alternative would also be relatively easy to construct and operate, with little need for advanced instrumentation and controls. All flows from the Northeast Interceptor would be directed to PS18. An overflow connection would need to be constructed to connect PS18 with the Southeast Interceptor (and PS7).

The disadvantages of this alternative are high construction costs to provide the required capacity and higher pumping costs during wet weather flow events relative to PS7. While the design of PS18 would be similar to PS7 with regards to flow capacity and building footprint, the PS18 force main will be approximately 7,000 feet longer than the PS7 force main. As a result the total dynamic head for PS18 will be higher than PS7, increasing the cost to pump flow to NSWWTP.

Alternative 2

Alternative 2 is similar to Alternative 1 in that average daily and peak hourly flows in the Northeast Interceptor would be directed and conveyed primarily to PS18. However, under this alternative the capacity of PS18 would be lowered and limited to 37 mgd (Madison Design Curve) or 44 mgd (modified Madison Design Curve). This is approximately one-half of the projected 2060 peak flows in the Eastside collection system. The balance of the flows would be conveyed to PS7.

A goal in developing this alternative is to provide two similar-sized stations that will each convey approximately one-half of the flow in the Eastside collection system. Operation of PS18 under this scenario would be slightly more challenging than Alternative 1. Careful consideration would have to be given to the design of a diversion structure such that flows in excess of the pre-determined capacity of PS18 would be directed towards PS7.

Alternative 3

Under Alternative 3 PS18 would convey average daily flows from the Northeast Interceptor similar to the two other alternatives, but all excess flows would be directed towards PS7. The conveyance of average daily flows from the Northeast Interceptor by PS18 would be enough to keep the pumping equipment in good working order and maintain adequate flushing velocities in the force main, while minimizing peak pumping costs relative to PS7.

Flows at PS18 would continue to grow over time with increased development and wastewater flows in the Eastside collection system. The primary advantages sought in developing this alternative are lower construction costs and energy efficiency. Only average daily flows from the Northeast Interceptor would be conveyed from PS18 in an effort to limit energy costs.

Several disadvantages are noted for this alternative. The primary disadvantage of this alternative is that the conveyance of *all* wet weather flows from the Northeast Interceptor would be directed to PS7. The firm capacity of PS7 would be exceeded prior to 2030 as a result. Another significant disadvantage to this alternative is that PS18 would have limited ability to convey flows diverted from PS7 during high flow or emergency events. For these reasons, it is not recommended that Alternative 3 be advanced for further study during preliminary design of the PS18 improvements.

Preliminary Sizing of PS18 Pumps and Force Main for Peak Flows

In order to assess the number and size of pumps that may be needed to achieve the maximum pumping capacity for PS18, a preliminary analysis of pump configurations was conducted. This analysis was performed for Alternative 1, which has the highest capacity requirements (54 mgd for traditional MDC and 66 mgd for modified MDC).

Given that nearly half of the District's pumping stations currently utilize pumps manufactured by Fairbanks Morse, maximum station capacity for Alternative 1 was evaluated for Fairbanks Morse centrifugal pumps in 20" and 24" outlet sizes. The 20" pumps used in the analysis are Fairbanks Model No. 5722, with 30" impellers, a two-vane impeller design, and an operating speed of 705 rpm. They are very similar to the pumps that were installed at PS8 in 2010. The 24" pumps are also Model No. 5722 with 36"- 40" impellers and an operating speed of 585 rpm. The 24" pumps have a five-vane impeller design, however, which raises concerns related to pump plugging with rags and other stringy material. The District's largest pumps in the collection system are currently 20" pumps. It would be preferable to use 20" pumps with a two-vane impeller at PS18 if possible, for purposes of familiarity, consistency, and reliability.

The maximum station capacity for PS18 was also analyzed for a 42" and 48" diameter forcemain from PS18 to the existing 42" forcemain at the Nine Springs Wastewater Treatment Plant. The scenarios that were analyzed and the results of this analysis are summarized briefly in Table 3.

For purposes of this analysis it was assumed that all pumps used to achieve the maximum pumping capacity were identical units. Further, it was assumed impractical to provide more than four pumping units to achieve maximum capacity.

Table 3: Analysis of PS18 Peak Flow Capacity (Alternative 1)

Peak Flow Scenario	Pump Size (in)	Forcemain diameter (in)	Adequate Capacity Available	Minimum Number of Pumps Needed	System & Pump Curves
Madison Design Curve (54 mgd)	20	42	No	>4	Fig 7
		48	Yes	3	Fig 7
	24	42	Yes	2	Fig 8
		48	Yes	2	Fig 8
Modified Madison Design Curve (66 mgd)	20	42	No	>4	Fig 7A
		48	Yes	4	Fig 7A
	24	42	Yes	3	Fig 8A
		48	Yes	3	Fig 8A

Peak Flows from Madison Design Curve (54 mgd)

Four equal-sized 20" pumps cannot deliver enough flow through a 42" forcemain to achieve a maximum capacity of 54 mgd. These three pumps would be sufficient for a 48" forcemain, however. Two equal-sized 24" pumps could provide the maximum capacity of 54 mgd in either a 42" or 48" forcemain.

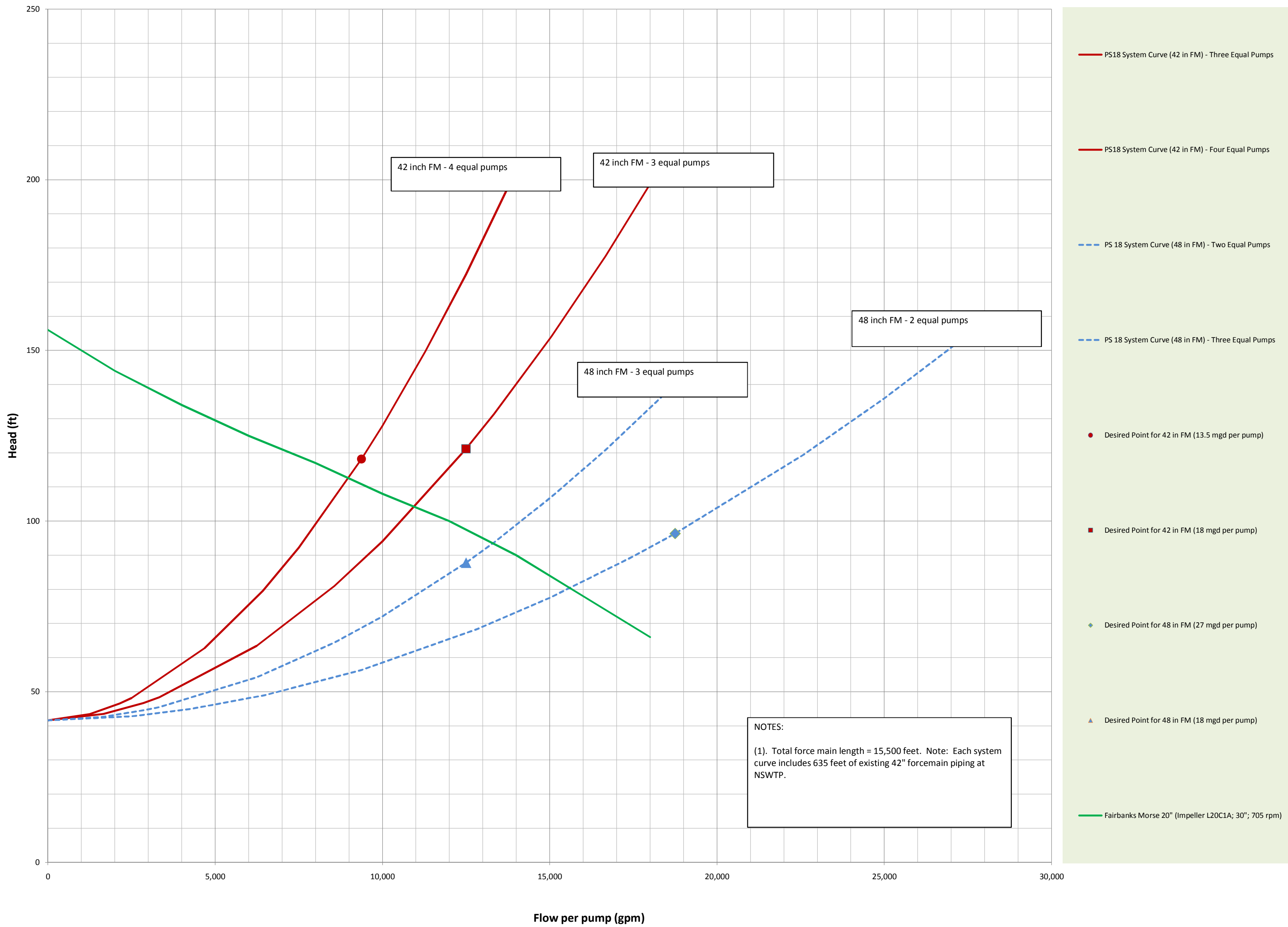
Peaks Flows from 'Modified' Madison Design Curve (66 mgd)

In order to achieve the maximum capacity of 66 mgd as required for the modified Madison Design Curve, four equal-sized 20" pumps and a 48" diameter forcemain would be needed. It would be impractical, however, to provide 20" pumps and a 42" diameter forcemain to achieve the desired capacity due to the number of pumps required. The maximum capacity could also be achieved using three equal-sized 24" pumps and either a 42" or 48" forcemain.

Conclusions

A summary of key design parameters for conveying peak flows at PS18 for Alternative 1 is provided in Table 4. The table includes a comparison of the parameters for both a 42" and a 48" forcemain. In looking at Table 4, it should be noted that the velocity in the 42" forcemain is in excess of 8 feet per second for both station capacities under Alternative 1 (54 and 66 mgd). Exceeding this value is not considered good design. For a station capacity of 66 mgd the maximum flowrate of 8.12 feet per second in a 48" forcemain would exceed the standard by only

Figure 7: Preliminary Pump Sizing for Alternative 1 (54 mgd)
20" Pumps



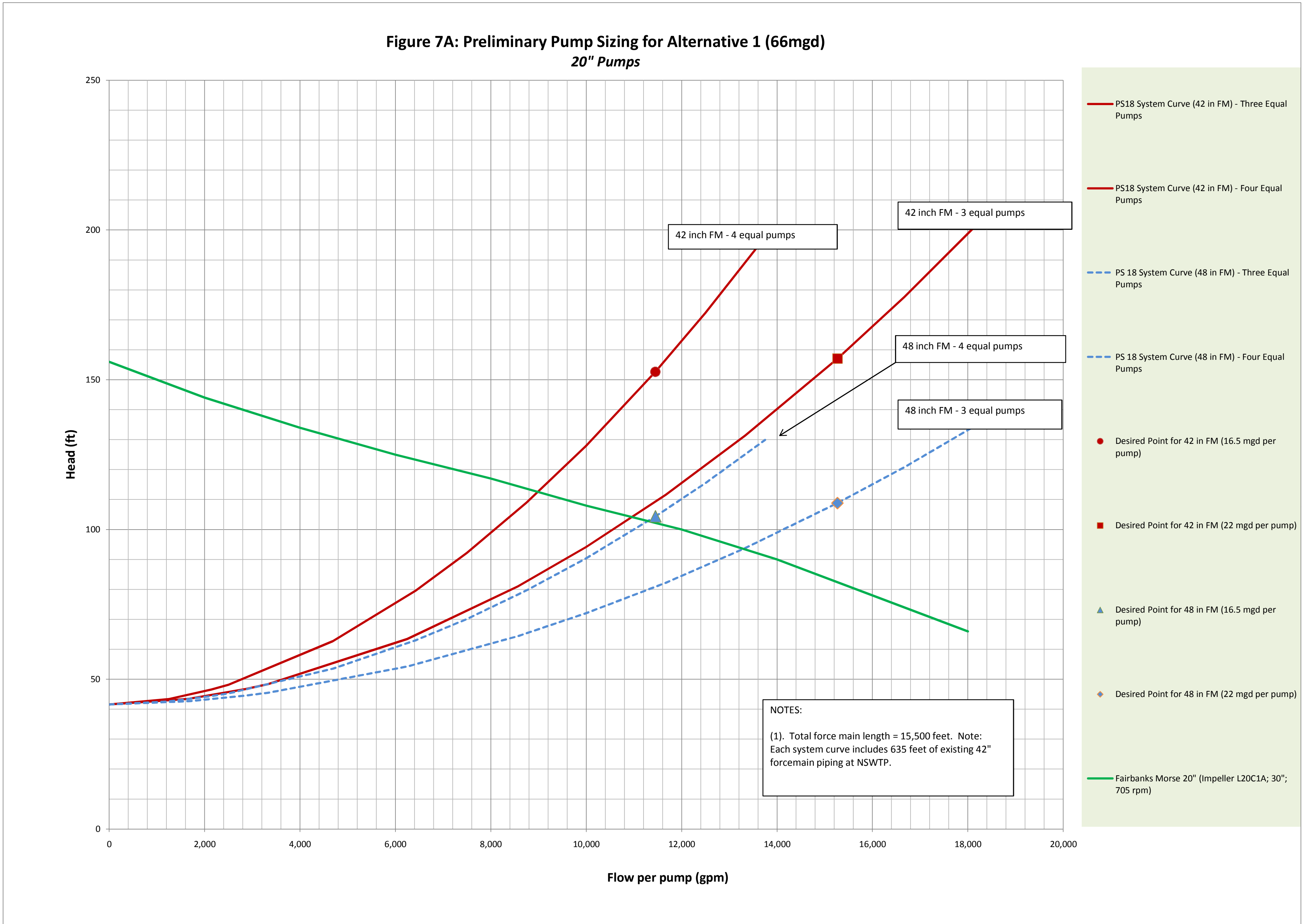


Figure 7A: Preliminary Pump Sizing for Alternative 1 (66mgd)
20" Pumps
Head (ft)
Flow per pump (gpm)
0
50
100
150
200
250
0
2,000
4,000
6,000
8,000
10,000
12,000
14,000
16,000
18,000
20,000
42 inch FM - 4 equal pumps
42 inch FM - 3 equal pumps
48 inch FM - 4 equal pumps
48 inch FM - 3 equal pumps
NOTES:
(1). Total force main length = 15,500 feet. Note: Each system curve includes 635 feet of existing 42" forcemain piping at NSWTP.
PS18 System Curve (42 in FM) - Three Equal Pumps
PS18 System Curve (42 in FM) - Four Equal Pumps
PS 18 System Curve (48 in FM) - Three Equal Pumps
PS 18 System Curve (48 in FM) - Four Equal Pumps
Desired Point for 42 in FM (16.5 mgd per pump)
Desired Point for 42 in FM (22 mgd per pump)
Desired Point for 48 in FM (16.5 mgd per pump)
Desired Point for 48 in FM (22 mgd per pump)
Fairbanks Morse 20" (Impeller L20C1A; 30"; 705 rpm)

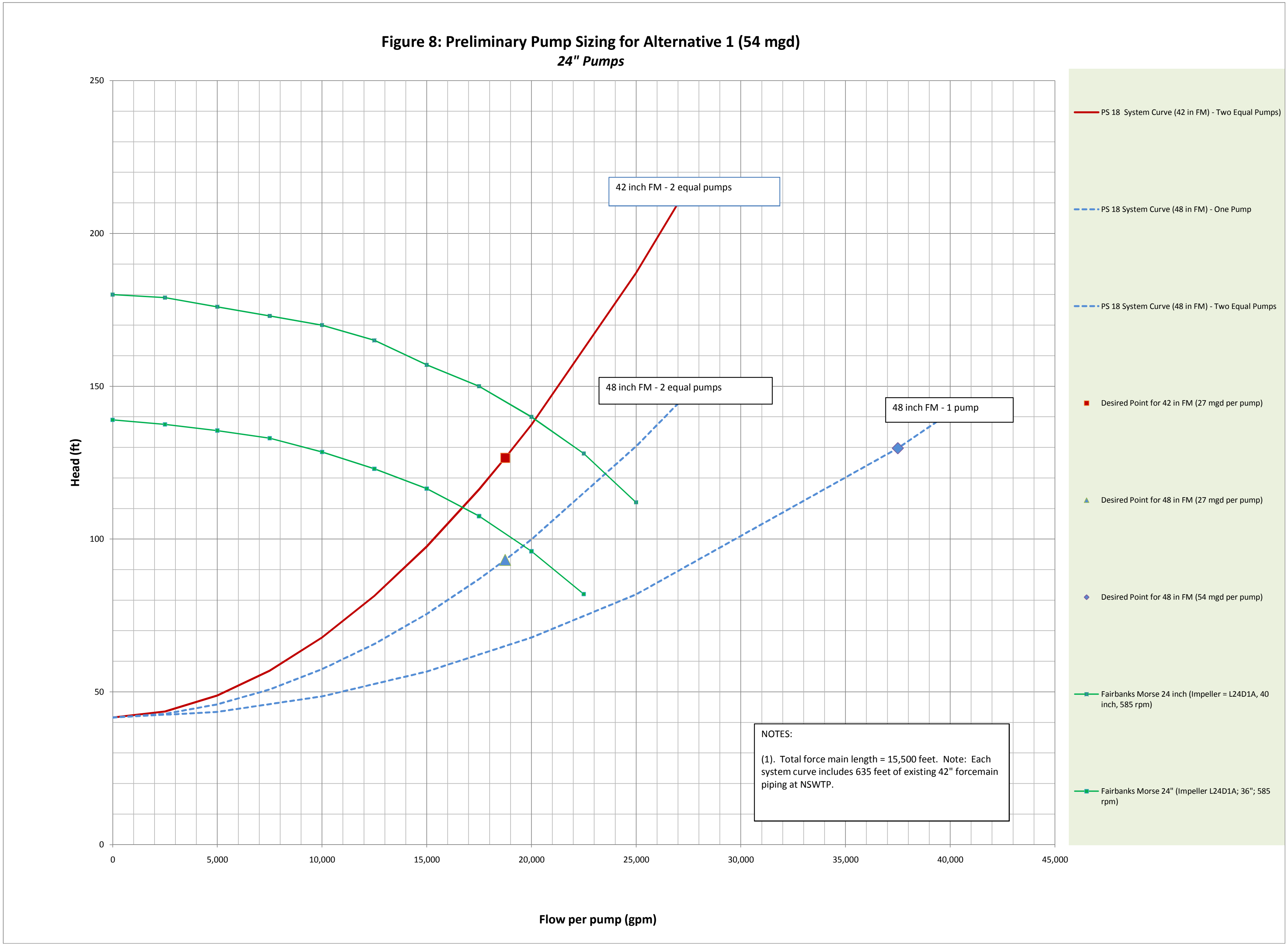


Figure 8: Preliminary Pump Sizing for Alternative 1 (54 mgd)
24" Pumps
Head (ft)
Flow per pump (gpm)
0
50
100
150
200
250
0
5,000
10,000
15,000
20,000
25,000
30,000
35,000
40,000
45,000
42 inch FM - 2 equal pumps
48 inch FM - 2 equal pumps
48 inch FM - 1 pump
NOTES:
(1). Total force main length = 15,500 feet. Note: Each system curve includes 635 feet of existing 42" forcemain piping at NSWTP.
PS 18 System Curve (42 in FM) - Two Equal Pumps)
PS 18 System Curve (48 in FM) - One Pump
PS 18 System Curve (48 in FM) - Two Equal Pumps
Desired Point for 42 in FM (27 mgd per pump)
Desired Point for 48 in FM (27 mgd per pump)
Desired Point for 48 in FM (54 mgd per pump)
Fairbanks Morse 24 inch (Impeller = L24D1A, 40 inch, 585 rpm)
Fairbanks Morse 24" (Impeller L24D1A; 36"; 585 rpm)

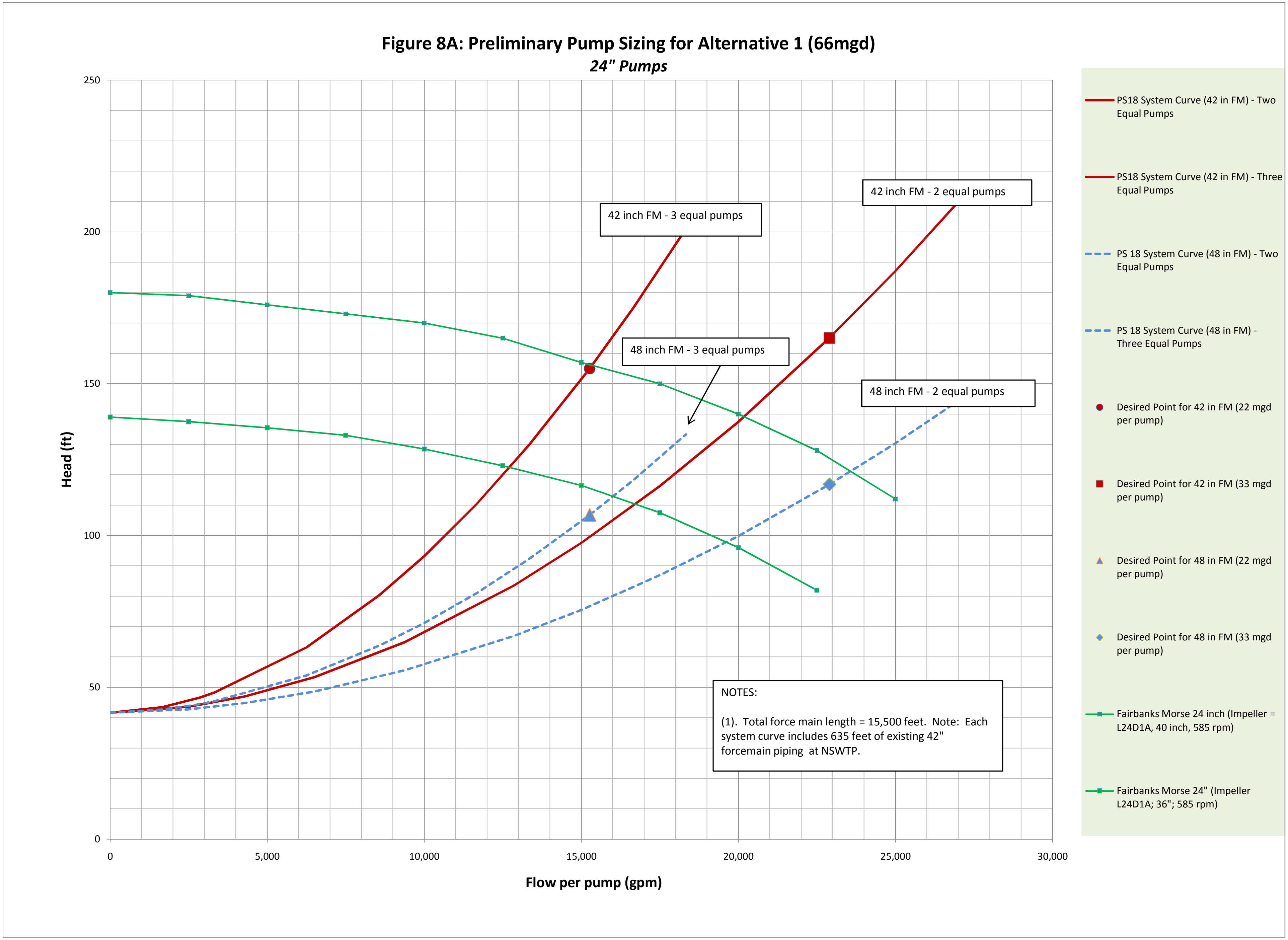


Figure 8A: Preliminary Pump Sizing for Alternative 1 (66mgd)
24" Pumps
Head (ft)
Flow per pump (gpm)
0
50
100
150
200
250
0
5,000
10,000
15,000
20,000
25,000
30,000
42 inch FM - 3 equal pumps
42 inch FM - 2 equal pumps
48 inch FM - 3 equal pumps
48 inch FM - 2 equal pumps
NOTES:
(1). Total force main length = 15,500 feet. Note: Each system curve includes 635 feet of existing 42" forcemain piping at NSWTP.
PS18 System Curve (42 in FM) - Two Equal Pumps
PS18 System Curve (42 in FM) - Three Equal Pumps
PS 18 System Curve (48 in FM) - Two Equal Pumps
PS 18 System Curve (48 in FM) - Three Equal Pumps
Desired Point for 42 in FM (22 mgd per pump)
Desired Point for 42 in FM (33 mgd per pump)
Desired Point for 48 in FM (22 mgd per pump)
Desired Point for 48 in FM (33 mgd per pump)
Fairbanks Morse 24 inch (Impeller = L24D1A, 40 inch, 585 rpm)
Fairbanks Morse 24" (Impeller L24D1A; 36"; 585 rpm)

a small margin. In addition, it can be seen that the headlosses due to friction are significantly higher in the 42" forcemain relative to the 48" forcemain, especially at higher flowrates.

Table 4: Design Parameters for PS18 Peak Flow Pumping (Alternative 1)

Parameter per pump or per forcemain	*Madison Design Curve (54 mgd)*		*'Modified Madison Design Curve (66 mgd)*	
	42" FM	48" FM	42" FM	48" FM
Number and size of pumps	2 – 24"	3 – 20"	3 – 24"	4 – 20"
Pump capacity (mgd)	27.0	18.0	22.0	16.5
Total pumping head (ft)	127	88	155	104
Maximum forcemain velocity (fps)	8.68	6.65	10.61	8.12
Motor horsepower	705	330	705	355
Forcemain velocity (fps) for 2010 ADF of 12.35 mgd	1.99	1.52	1.99	1.52
Forcemain velocity (fps) for 2020 ADF of 15.15 (fps)	2.44	1.87	2.44	1.87
Forcemain velocity (fps) for 2030 ADF of 17.95 mgd	2.89	2.21	2.89	2.21
Forcemain velocity (fps) for 2060 ADF of 21.85 mgd	3.51	2.69	3.51	2.69

Notes: ADF = Average daily flow

An additional factor to consider in selecting a 48" forcemain is that it favors the use of 20" pumps rather than 24" pumps to achieve the maximum capacity. This is an important consideration from an operational perspective. The use of smaller 20" pumps may also provide more flexibility in conveying average daily flows at PS18 by allowing them to be equipped with adjustable frequency drives.

Based on the information provided in Tables 3 and 4, it is recommended that a 48" forcemain be installed from PS18 to the existing 42" forcemain at the Nine Springs Wastewater Treatment Plant. The larger forcemain provides greater flexibility in meeting the peak flow requirements of PS18 under Alternative 1 and in conveying average daily flows. It should be noted that the velocity in a 48" forcemain will be below the recommended minimum velocity of 2 feet per second until approximately 2024 based on average daily flowrates. The lack of adequate flushing velocity will need to be evaluated during detailed design and a pumping strategy will need to be implemented during the initial years of operation to ensure that solids deposition does not occur. The District has utilized daily flushing cycles with large pumps at other pumping stations to prevent solids deposition and similar programming could be used at PS18.

It should also be stressed that the preliminary sizing discussed in this section considers only peak pumping considerations for Alternative 1 (worst case scenario) and not the conveyance of average daily flows. Final pump selection for PS18 will need to consider the peak flow requirements for other alternatives and pump sizes that are suitable for both everyday operation and for intermittent operation during peak flow events. Combinations of constant-speed pumps and pumps with adjustable frequency drives will likely be required to achieve the desired flowrates.

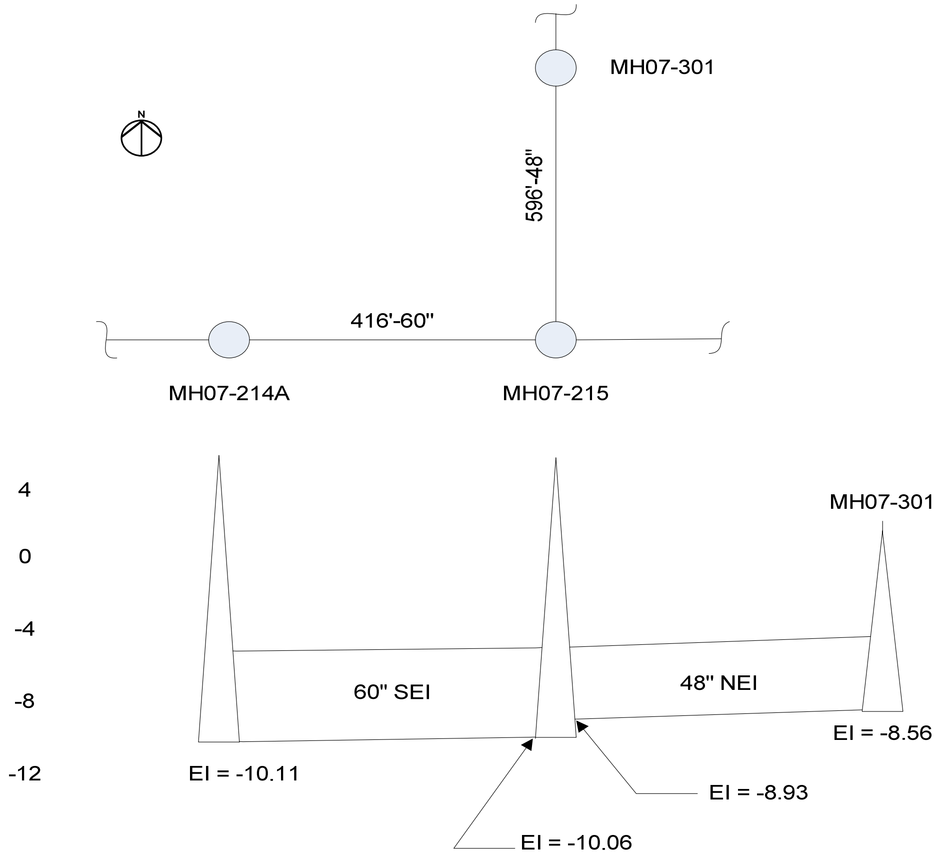
Emergency Diversions

Besides providing additional capacity for the Eastside collection system, a major feature of a new PS18 should be the ability to transfer flows with PS7 in emergency situations. In the event that PS7 has a loss of power or other type of failure and/or one or both of the PS7 forcemains become disabled, it would be desirable for PS18 to accommodate the flow that is normally conveyed through PS7. While it may not be possible to transfer all of the flow during high-flow events, it would be beneficial if dry weather and smaller wet weather flows from PS7 could be conveyed to PS18.

One critical factor in conveying flows from PS7 to a new PS18 is the elevation and size of the sewers at the junction of the Southeast Interceptor and Northeast Interceptor (see Figure 9). There is an elevation difference of approximately 1.1 feet in the inverts at the junction of the Southeast and Northeast interceptors (MH07-215). The 596 foot segment of 48” interceptor upstream of MH07-215 has significant headlosses relative to the 60” Southeast Interceptor and will force the water to rise higher in the PS7 wet well in order to drive the flow backwards along the Southeast Interceptor to the Northeast Interceptor and eventually to PS18. The design of a new PS18 should consider the construction of a new connector line from the Southeast Interceptor to PS18 that minimizes this elevation drop and increases the size of the connector line to PS18.

Preliminary calculations were performed to estimate the hydraulic grade line between PS7 and PS18 in the event of an outage at PS7 (see Table 5 in attachments for calculations). The analysis includes both existing conditions (48” NEI from MH07-215 to PS18) and proposed conditions (60” NEI from MH07-215 to PS18). The analysis assumes that MH07-301, located near PS18, is flowing full. It also assumes that PS18 has adequate capacity to convey all flows in the Eastside collection system, including those from PS7 to PS18. Table 6 is a summary of the water surface elevations in the PS7 wet well that can be expected for various peak flowrate scenarios at PS7.

Figure 9: Plan and Profile of Existing Southeast and Northeast Interceptor Junction



As can be seen in Table 6, for all the scenarios listed the water surface elevation in the PS7 wet well would exceed the high water alarm at the station. The District's *Emergency Response Manual* directs users to contact the City of Monona if the wet well reaches an elevation of -2.5 feet to alert them of possible flooding near PS7. This water surface elevation corresponds to a flowrate of approximately 24.5 mgd for existing conditions. At a water surface elevation of +1.00 in the PS7 wet well water would begin to overflow to the Yahara River, although it is likely that many basements in the Monona area would experience backups prior to reaching this elevation. It is estimated that a flowrate of 40 mgd could be achieved at the overflow elevation.

In summary, Alternative 1 provides the greatest diversion capacity. The water surface elevation would not rise above the level of anticipated basement flooding for any of the scenarios shown. Replacement of the 48" gravity interceptor from MH07-214A to PS18 would not be needed under this alternative. For Alternative 2, the water surface elevation at PS7 would exceed the expected level of basement flooding by the year 2025. Providing additional capacity between MH07-214A and PS18 could prolong exceedance of the flooding elevation by approximately five years to 2030. Alternative 3 could not provide sufficient wet weather diversion capacity for any of the scenarios.

The analysis shown in Table 6 is a theoretical exercise that was performed to: (1). Estimate the maximum wet well level elevation at PS7 for various rates of peak flow; and (2). Assess the need for a new interceptor segment from MH07-214A to PS18. This analysis has limited usefulness in that it is not able to accurately simulate the splitting and routing of flows with time in the Northeast Interceptor and Southeast Interceptor near PS18. A more practical simulation of the flow diversion capabilities between PS7 and PS18 is needed and is presented in the next section.

Table 6: Maximum Wet Well Elevations at PS7 for Emergency Diversion to PS18

PS 18 Alternative(1)	Year	PS18 Peak Hourly Flowrate (mgd)	PS7 Peak Hourly Flowrate (mgd)	Water Surface Elevation at PS 7 (ft)		PS 7 High Water Alarm (ft)
				Existing Conditions	Proposed Conditions(2)	
1	2010	33	14	-3.89	-3.98	-6.50
	2030	46	18	-3.45	-3.60	-6.50
	2060	54	21	-3.05	-3.26	-6.50
2	2010	33	14	-3.89	-3.98	-6.50
	2030	37	27	-2.06	-2.41	-6.50
	2060	37	37	+0.13	-0.52	-6.50
3	2010	12	35	-0.37	-0.94	-6.50
	2030	18	46	+2.69	+1.67	-6.50
	2060	22	53	+5.06	+3.71	-6.50

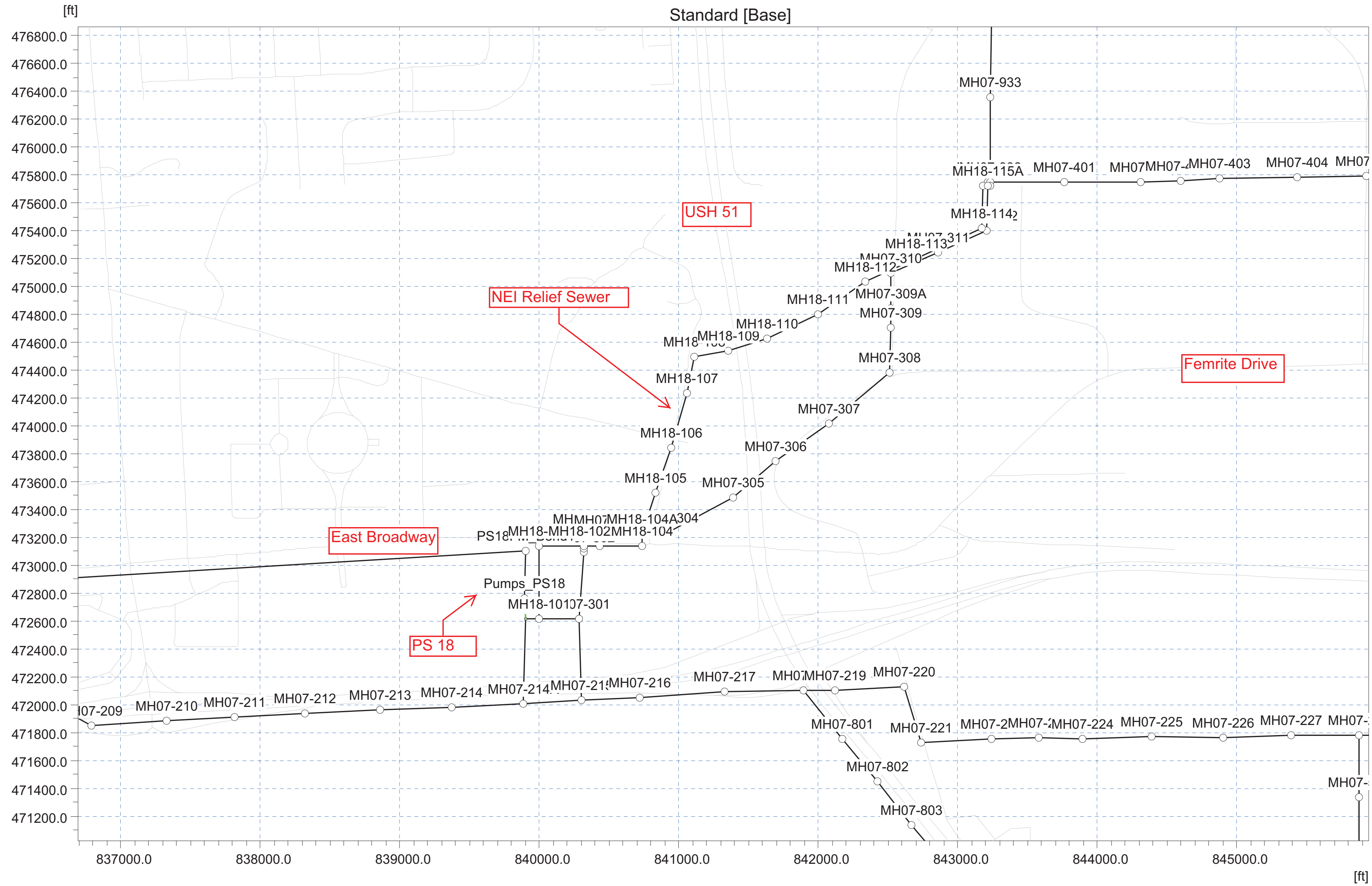
Notes:
(1). For peak hourly flows derived from Madison Design Curve only.
(2). Assumes a new 60” gravity line from Southeast Interceptor to PS18 to replace existing 48” line.

Hydraulic Modeling of PS18

The District’s hydraulic model was used to simulate the conveyance of peak flows in the Eastside collection system and the diversion capabilities between PS7 and PS18. The location and alignments for new PS18, the new PS18 forcemain, and the Northeast Interceptor relief sewer were input into the model as described previously in this study (see Figure 10). Recognizing that PS18 will be very similar to PS7 in terms of size and capacity, the general layout of the wet well and pump capacities for the PS18 model were set nearly identical to those for PS7 for modeling purposes.

The model was run to simulate a service outage at PS7 in order to estimate the well level rise at PS7 that could be expected under existing and proposed conditions. Existing conditions include operation of a new PS18 and continued operation of the 48” interceptor sewer segment from MH07-215 to MH07-301. Proposed conditions include operation of a new PS18 and the

Figure 10: Hydraulic Model for Pump Station 18



replacement of the aforementioned sewer segment with a new 60” sewer from MH07-214A to PS18. For each condition the model was evaluated at 2010 flowrates for periods of dry and wet weather.

PS7 Out of Service - Dry Weather Simulation

The results of the dry weather simulation are shown in Figure 11. The average daily dry weather flowrate from PS7 to PS18 was modeled at approximately 2.4 mgd. At this flowrate the average wet well levels at PS7 for existing and proposed conditions are -7.1 and -8.5, respectively. These elevations are both below the current high water alarm elevation of -6.5. As a result, basement flooding in the PS7 service area should not be a concern while diverting flows from PS7 to PS18 in periods of dry weather. It should be pointed out that replacing the 48” NEI segment from MH07-215 to PS18 with a 60” sewer will keep the PS7 wet well level approximately 1.4 feet lower during the diversion of flows from PS7 to PS18.

PS7 Out of Service – Wet Weather Simulation

Modeling of the inter-tie between PS7 and PS18 during wet weather is shown in Figure 12. To simulate the effect of wet weather, storm data for the period of May 19-22, 2004, was used. During this storm event the District’s collection system received approximately 5.96 inches of rain (as measured at the Dane County Regional Airport). A plot of the rainfall distribution and the modeled pumping rate at PS18 can be found in Figure 13.

For existing conditions the modeled wet well level at PS7 rose to a maximum elevation of -5.9 and remained slightly above the high water alarm elevation for approximately a one-day period. Under proposed conditions the modeled wet well level at PS7 rose gradually but never exceeded Elevation -7.5.

PS18 flowrate information is also shown on Figure 12 for the modeled wet weather event. Prior to the storm event the average daily flowrate at PS18 was approximately 17.3 mgd. This modeled flowrate agrees very well to the actual 2010 average daily flowrate at PS7 (16.8 mgd). Approximately 2.5 mgd of this flow prior to the storm was being diverted from PS7 to PS18, similar to the dry weather simulation. The modeled flowrate at PS18 rose steadily during the wet weather simulation and reached peaks of approximately 60 mgd. During the storm the average flowrate from PS7 to PS18 increased from 2.5 mgd to 4.7 mgd.

Summary

Hydraulic modeling suggests that 2010 dry weather flows can safely be conveyed from PS7 to PS18 during a loss of power or other operational problems at PS7 that require the station to be taken out of service. For existing conditions the well level at PS7 should remain below the high water alarm elevation during diversion events. Replacing the Northeast Interceptor segment directly upstream of the Southeast Interceptor will provide an additional level of comfort for diversion of flow during dry weather. It is expected that the well level at PS7 will be approximately 1.5 feet lower for these conditions.

Figure 11: Hydraulic Modeling of Dry Weather Flow Diversion - 2010 (PS 7 to PS 18)

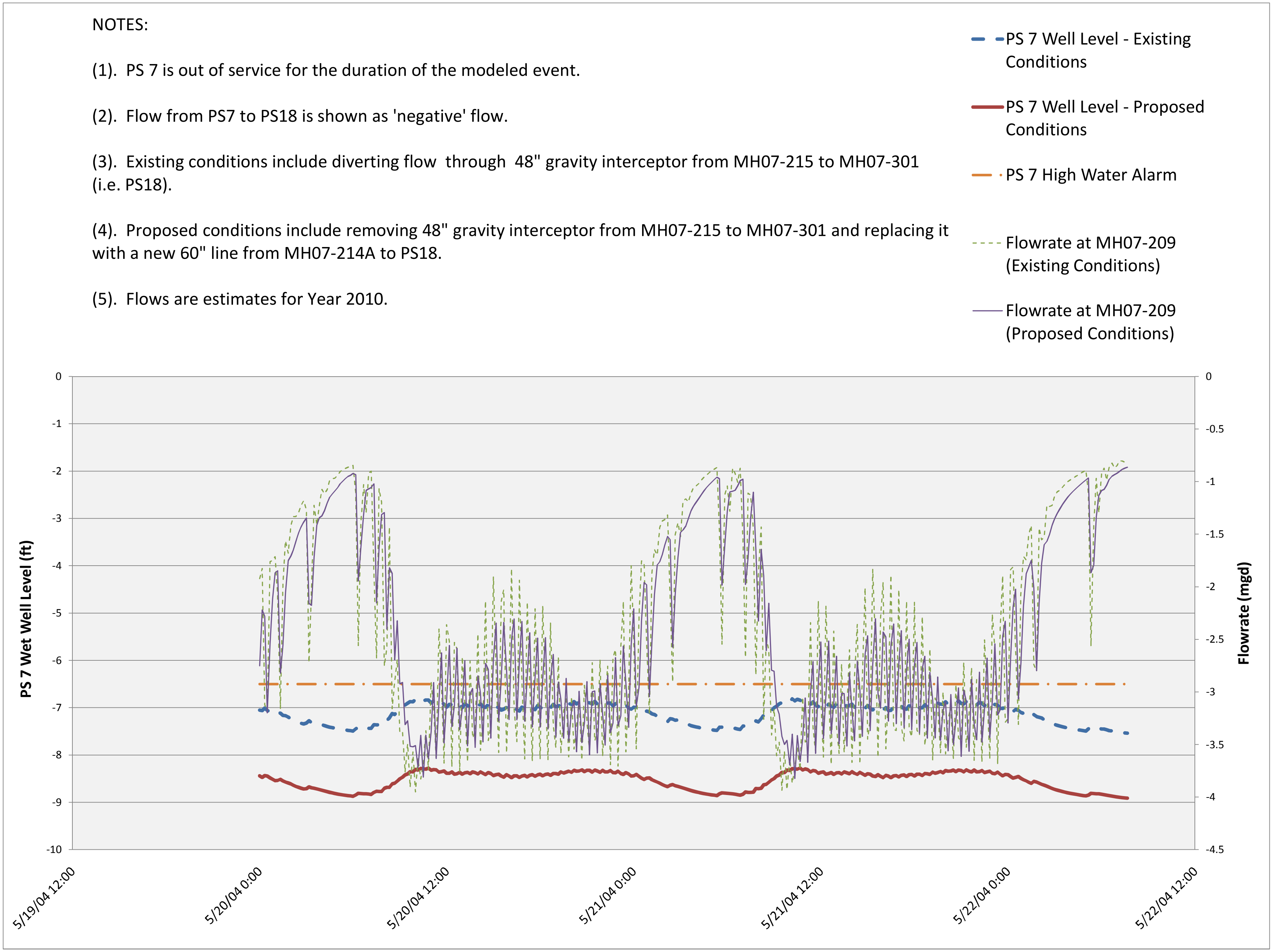


Figure 11A: Hydraulic Modeling of Dry Weather Flow Diversion - 2060 (PS 7 to PS 18)

NOTES:

(1). PS 7 is out of service for the duration of the modeled event.

(2). Flow from PS7 to PS18 is shown as 'negative' flow.

(3). Existing conditions include diverting flow through 48" gravity interceptor from MH07-215 to MH07-301 (i.e. PS18).

(4). Proposed conditions include removing 48" gravity interceptor from MH07-215 to MH07-301 and replacing it with a new 60" line from MH07-214A to PS18.

(5). Flows are estimates for Year 2060.

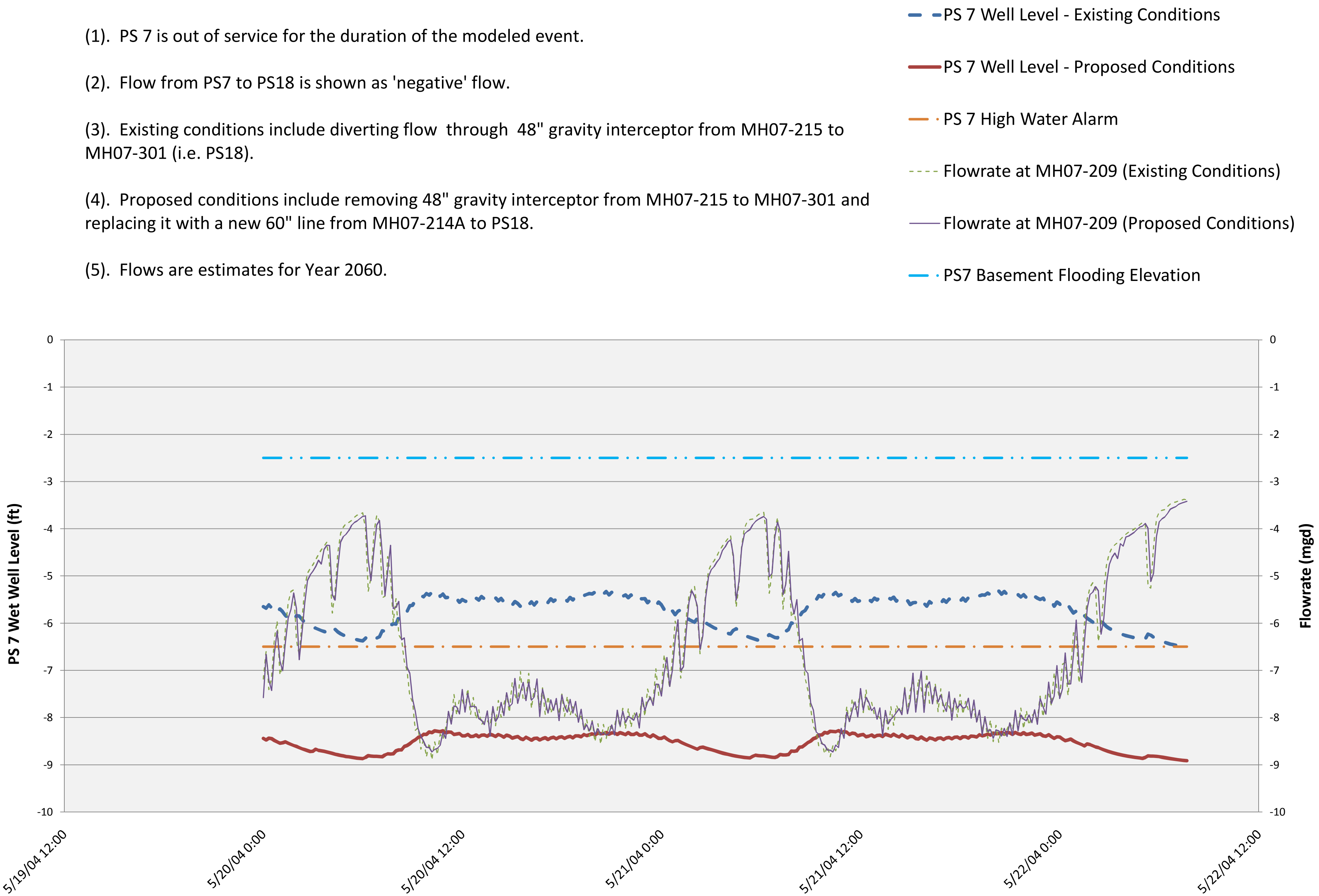


Figure 12: Hydraulic Modeling of Wet Weather Flow Diversion (PS 7 to PS 18)

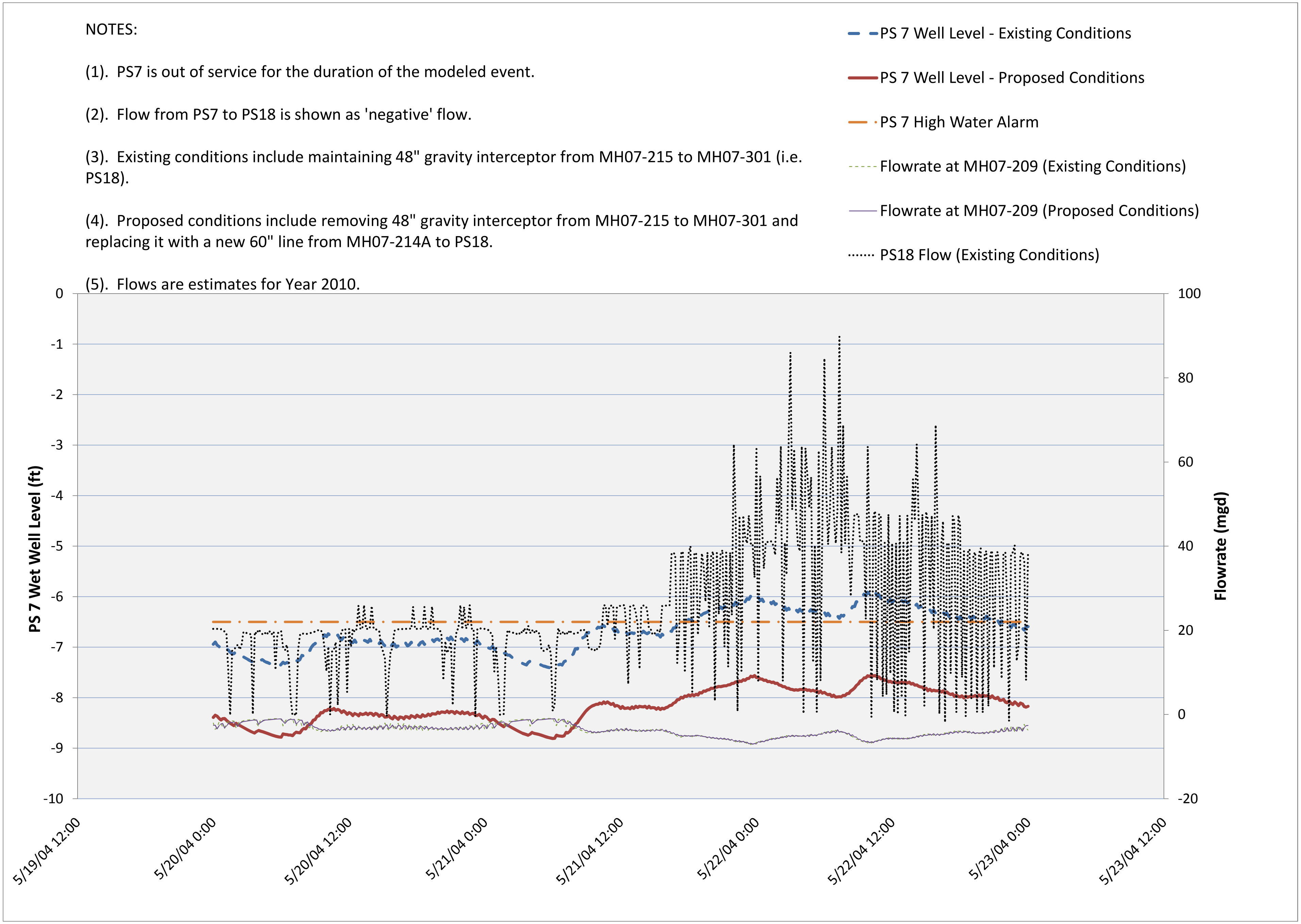
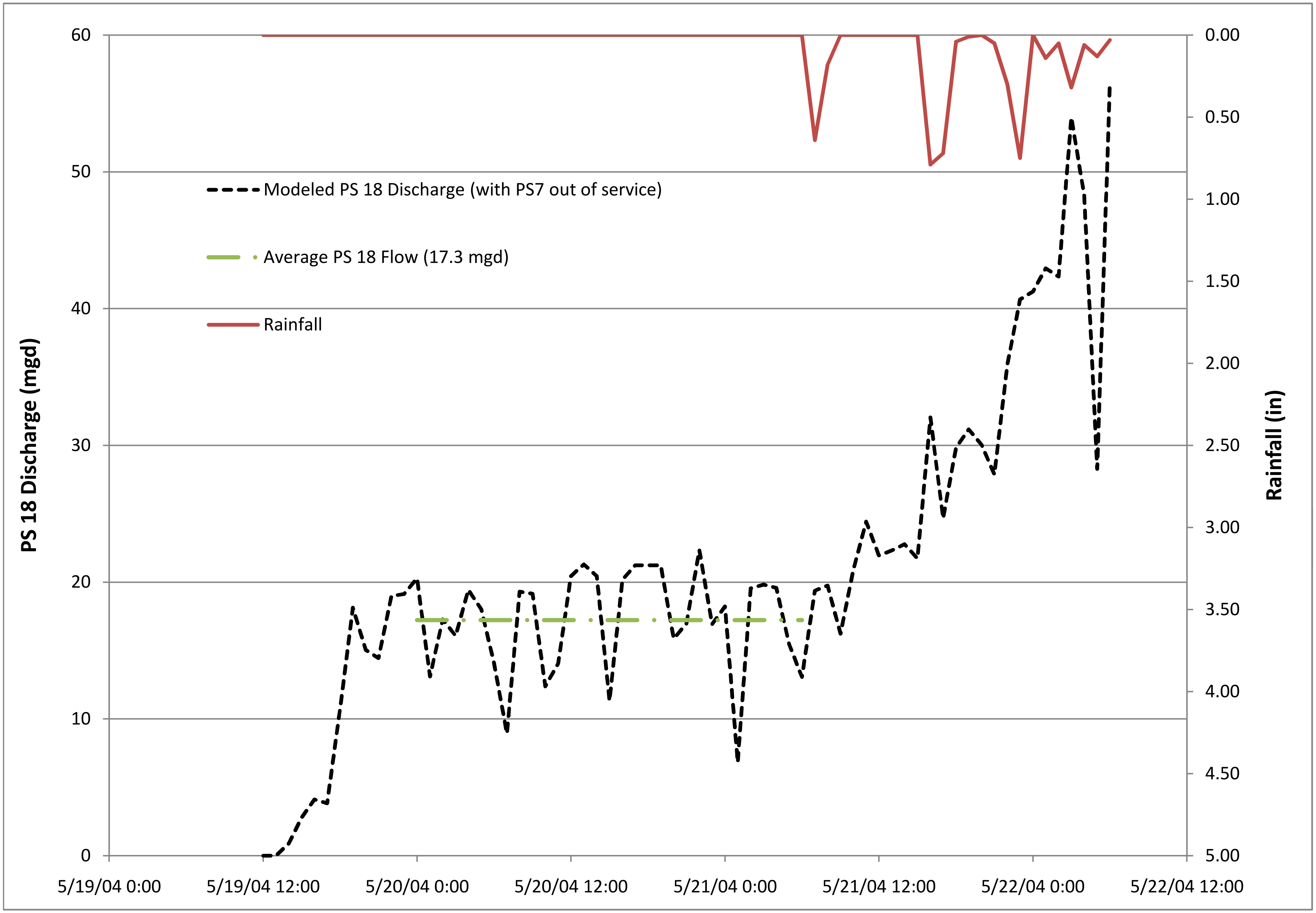


Figure 13: Wet Weather Flow (May 19-22, 2004 Storm)



Hydraulic modeling for 2060 dry weather flows shows that the wet well at PS7 will be above the high water alarm elevation for existing conditions but below the anticipated level at which basement flooding would occur (Figure 11A). The wet well level at PS7 for 2060 dry weather flows and proposed conditions would remain well below the high alarm level for a service outage at PS7.

Diverting flow from PS7 to PS18 during moderate wet weather events appears to be feasible without significant basement flooding in the PS7 service area. Diverting flows for long durations and/or for extreme wet weather events may not be possible without some basement flooding near PS7.

Replacing the existing 48” interceptor sewer segment (MH07-215 to MH07-301) with a new 60” sewer has an appreciable benefit during the diversion of flows from PS7 to PS18 during dry weather. It is estimated that the wet well at PS7 will be approximately 1.4 feet lower during diversion events if the 60” sewer is installed. It should be noted that installation of the 60” sewer will result in more flow being diverted from the Southeast Interceptor to PS18 during normal operations due to the lower invert elevation at MH07-214A. A cost-benefit analysis is recommended during the preliminary design phase to investigate this issue further.

Estimated Power Costs

The average daily flow at each pumping station is the same across all of the described operating alternatives. The alternatives differ in how the peak flows are distributed between the two pumping stations. Thus, for purposes of estimating annual energy use, only the average daily flowrates will be considered in this section.

Table 7 shows the approximate annual energy costs for PS7 and PS18 for existing and proposed conditions across three time periods. In this analysis it is assumed that the pumping rate is equal to the average daily flowrate and that the pump and motor efficiencies are the same for all conditions. As can be seen in this simplified calculation, the annual costs to pump average daily flows at PS7 and PS18 for all operating scenarios are very similar to the annual pumping costs for existing conditions at PS7. It is assumed in this analysis that a 48” force main is installed for PS18.

Energy costs associated with the pumping of wet weather flows were not considered in this analysis. While these costs are relevant, this preliminary analysis of pumping costs suggests that energy use may not be a primary factor in selection of the preferred alternative for the operation of PS18. Electric demand charges and back-up power requirements are important considerations that will need to be considered during preliminary design, however.

Table 7: Estimated Power Use for Pumping Scenarios at PS7 & PS18

	PS7				PS18				
Year	Average Daily Flow (mgd)	Pump Head (ft)	Energy Usage (kw·hr per Mgal)	Total Annual Energy Use ($/yr)	Average Daily Flow (mgd)	Pump Head (ft)	Energy Usage (kw·hr per Mgal)	Total Annual Energy Use ($/yr)	Total Annual Energy Use ($/yr)
Proposed conditions - PS7 + PS18									
2010	4.4	44.1	175.17	$28,000	12.4	49.4	196.23	$89,000	$117,000
2030	6.0	44.6	177.16	$70,000	18.0	55.6	220.85	$262,000	$332,000
2060	7.0	44.9	178.35	$200,000	22.0	61.2	243.10	$856,000	$1,056,000
Existing Conditions - PS7									
2010	16.8	50.6	200.99	$123,000	-	-	-	-	$123,000
2030	24.0	51.1	202.98	$321,000	-	-	-	-	$321,000
2060	29.0	54.4	216.09	$1,003,000	-	-	-	-	$1,003,000

Constants/Assumptions

(1). Pump Efficiency	0.85
(2). Motor Efficiency	0.93
(3). Unit Energy Cost ($/kw·hr)	$0.10
(4). Energy Escalation Rate (%)	3%
(5). PS18 force main diameter (in)	48

Summary and Recommendations

The District has identified a need to upgrade capacity and provide redundancy in its Eastside collection system. Immediate needs include an upgrade to firm pumping capacity at PS7, capacity relief in the Southeast Interceptor from PS7 to the Northeast Interceptor junction, and capacity relief for the Northeast Interceptor between the Southeast Interceptor and Far East Interceptor. The District also wishes to provide additional pumping capacity in this system to lessen the reliance on PS7 and provide more flexibility and diversion capabilities.

A new Pumping Station 18, located approximately 6,300 feet southeast of PS7, will accomplish the following goals:

1. Allow firm capacity pumping requirements at PS7 to be met, thus eliminating the need to increase the size of PS7 and the potential addition of another forcemain from PS7 to the NSWWTP.

2. Provide benchmark capacity for the Southeast Interceptor between PS7 and Northeast Interceptor junction, thus eliminating the need to provide a relief sewer from PS7 to the NEI junction.

3. Provide system redundancy and improve reliability for the Eastside collection system during service interruptions at PS7 for dry weather and small wet weather events.

It is expected that the capacity of PS18 will be very similar to that of PS7. Three alternate operating strategies have been proposed in this study for PS18 with regard to the conveyance of peak flows. Alternatives 1 and 2 are similar in their approach and propose that PS18 convey both average daily and peak flows from the Northeast Interceptor near MH07-301. Alternative 3 involves the pumping of primarily average daily flows from the Northeast Interceptor in an attempt to minimize construction costs and reduce annual pumping costs.

A preliminary analysis of pumping energy costs shows that Alternative 3 does not result in significant energy savings. This alternative does not alleviate the need for future capacity upgrades to PS7 and does provide sufficient diversion capacity for PS7. It is not recommended that Alternative 3 be advanced for further study.

Alternatives 1 and 2 provide peak flow capacity at PS18 that would provide the most benefit to PS7 and the Southeast Interceptor in both the near and long term. These alternatives are also capable of providing the required redundancy with PS7. Alternative 1 provides the greatest pumping capacity at PS18 (54 mgd) and is derived using MMSD's traditional peaking factors. It is recommended that PS18 be sized for an ultimate pumping capacity of 66 mgd based on the use of more conservative peaking factors.

Hydraulic modeling suggests that emergency diversion of flows from PS7 to PS18 can be performed safely during dry weather and possibly some moderate rain events. This diversion should not be relied upon for severe wet weather events or for extended outages in wet weather.

Further analysis is needed to determine how much peak flow capacity should be provided at PS18 relative to PS7 and the best method to split flows at PS18. This analysis should include a detailed investigation of the hydraulic inter-tie and the control strategies needed for splitting flows.

Attachments

1. Figure 1: Pumping Station 18 Site
2. Figure 2: Preliminary Route for Pumping Station 18 Forcemain
3. Figure 3: Preliminary Route for Northeast Interceptor Relief Sewer
4. Figure 4: Preliminary Peak Design Flow Schematic for Pump Station 18 Alternatives (2010 Flows) – Peak Flows from Madison Design Curve
5. Figure 4A: Preliminary Peak Design Flow Schematic for Pump Station 18 Alternatives (2010 Flows) – Peak Flows from 'Modified' Madison Design Curve
6. Figure 5: Preliminary Peak Design Flow Schematic for Pump Station 18 Alternatives (2030 Flows) – Peak Flows from Madison Design Curve
7. Figure 5A: Preliminary Peak Design Flow Schematic for Pump Station 18 Alternatives (2030 Flows) – Peak Flows from 'Modified' Madison Design Curve
8. Figure 6: Preliminary Peak Design Flow Schematic for Pump Station 18 Alternatives (2060 Flows) – Peak Flows from Madison Design Curve
9. Figure 6A: Preliminary Peak Design Flow Schematic for Pump Station 18 Alternatives (2060 Flows) – Peak Flows from 'Modified' Madison Design Curve
10. Table 5: Diversion from PS7 to PS18 (MH07-301)
11. Table 8A-1: Peak Hourly Flows for PS18 and PS7 – Alternative 1 (Madison Design Curve)
12. Table 8A-2: Peak Hourly Flows for PS18 and PS7 – Alternative 2 (Madison Design Curve)
13. Table 8A-3: Peak Hourly Flows for PS18 and PS7 – Alternative 3 (Madison Design Curve)
14. Table 9A-1: Peak Hourly Flows for PS18 and PS7 – Alternative 1 (Modified Madison Design Curve)
15. Table 9A-2: Peak Hourly Flows for PS18 and PS7 – Alternative 2 (Modified Madison Design Curve)
16. Table 9A-3: Peak Hourly Flows for PS18 and PS7 – Alternative 3 (Modified Madison Design Curve)

17. Figure 14: Pumping Station 14 Sub-basins
18. Figure 15: Pumping Station 13 Sub-basins
19. Figure 16: Pumping Station 10 Sub-basins
20. Figure 17: Pumping Station 9 Sub-basins
21. Figure 18: Pumping Station 7 Sub-basins

	Madison Metropolitan Sewerage District	**Figure 1** **Pumping Station 18 Site**	Prepared by: TWG Date: August, 2011

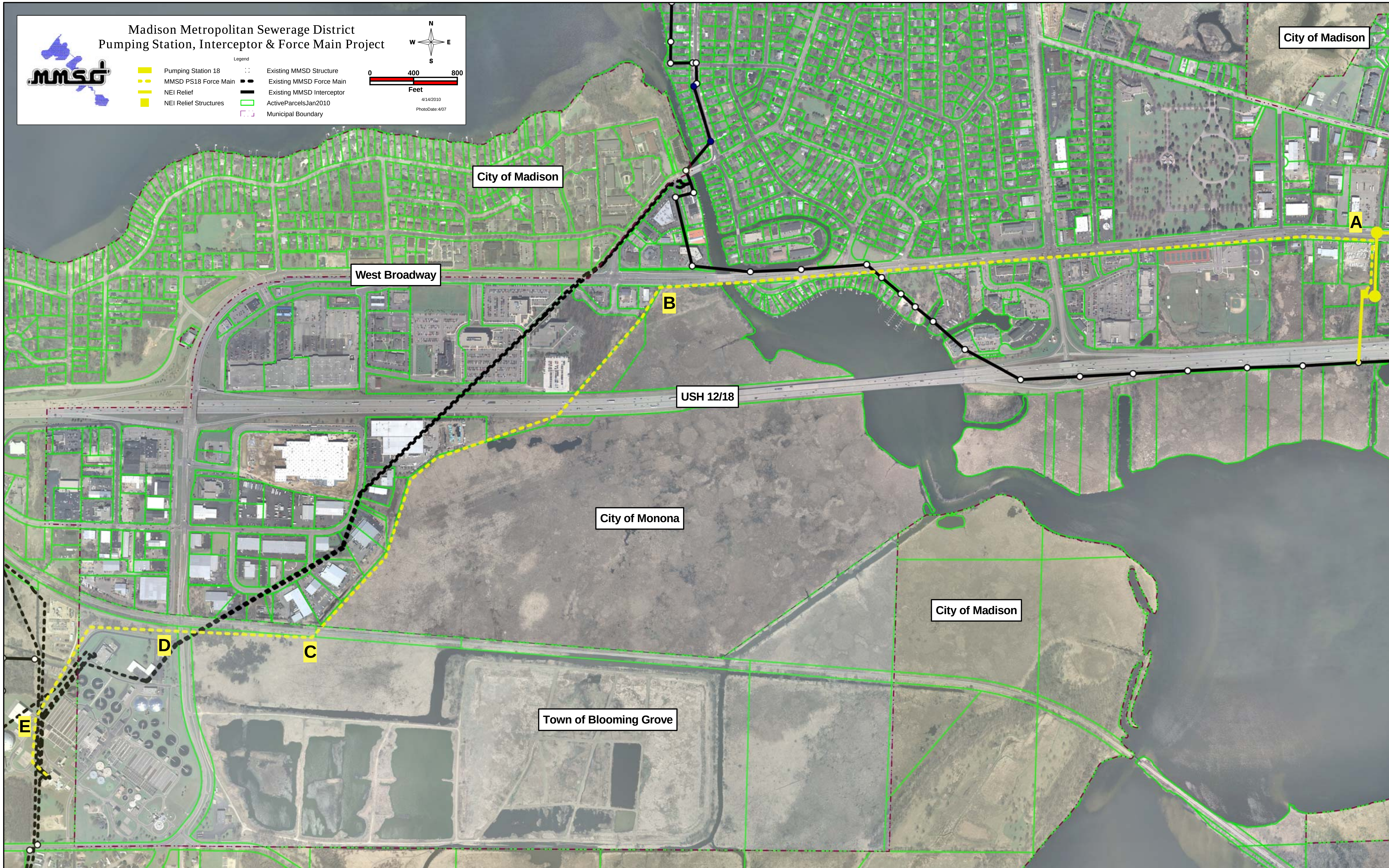


Figure 2 - Preliminary Route for Pumping Station 18 Forcemain

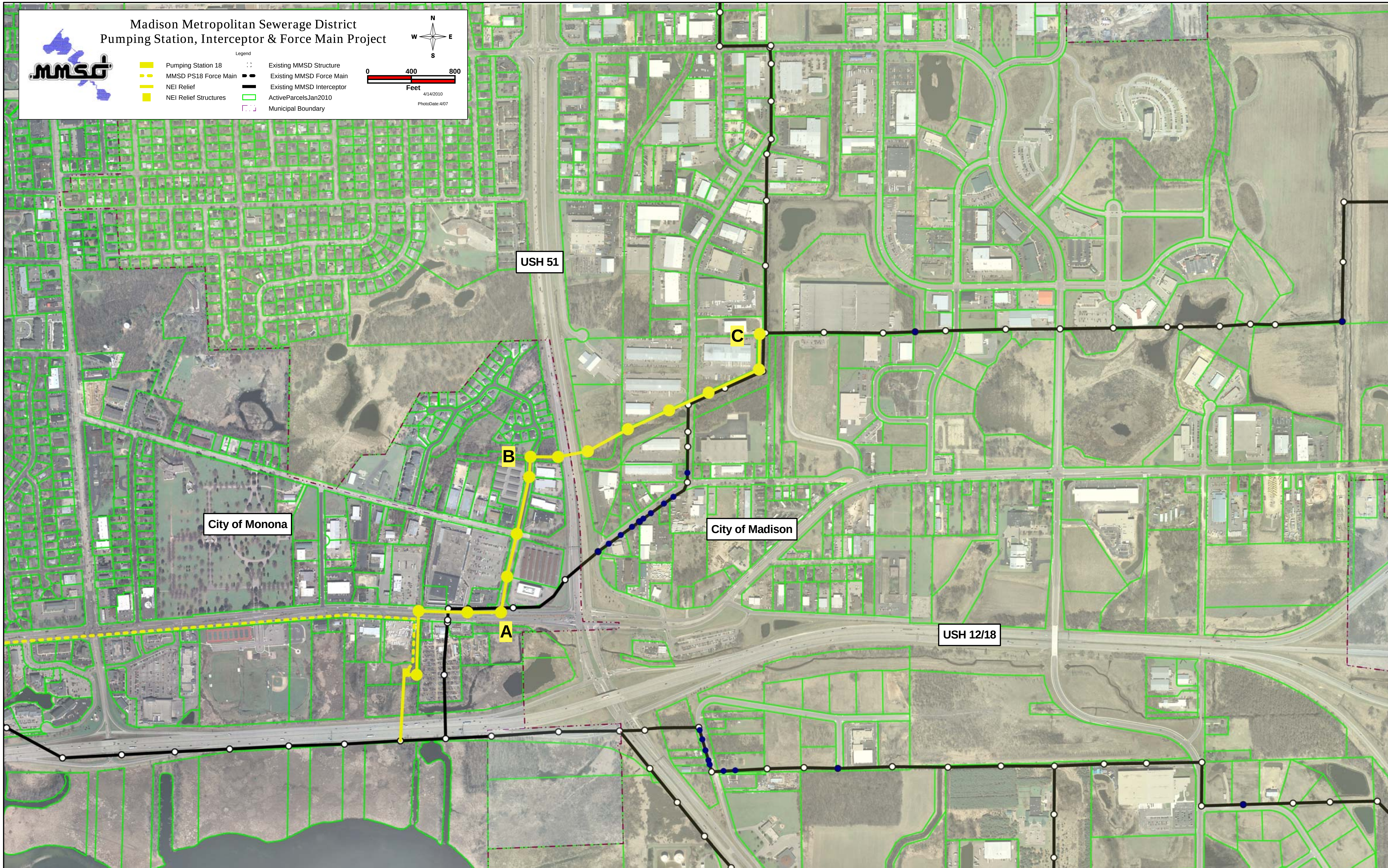
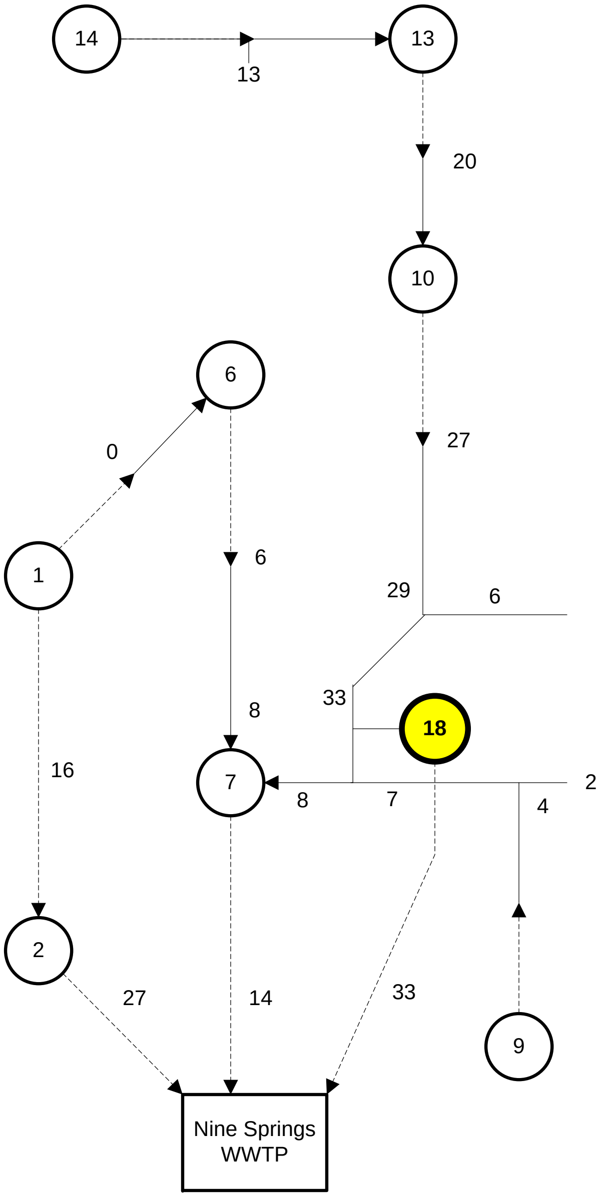
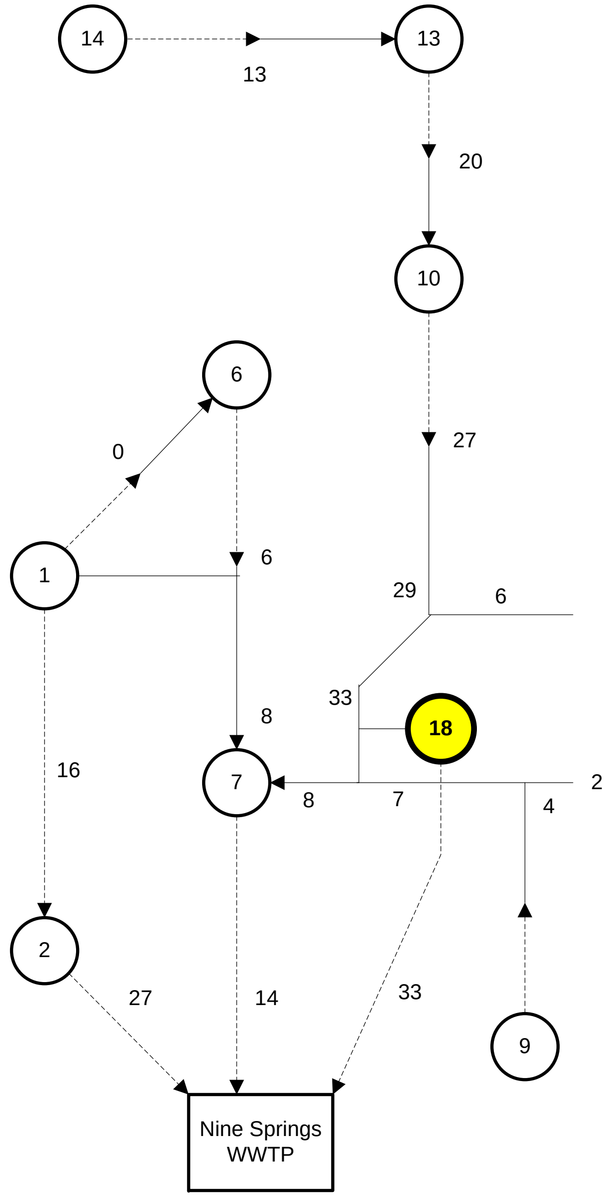


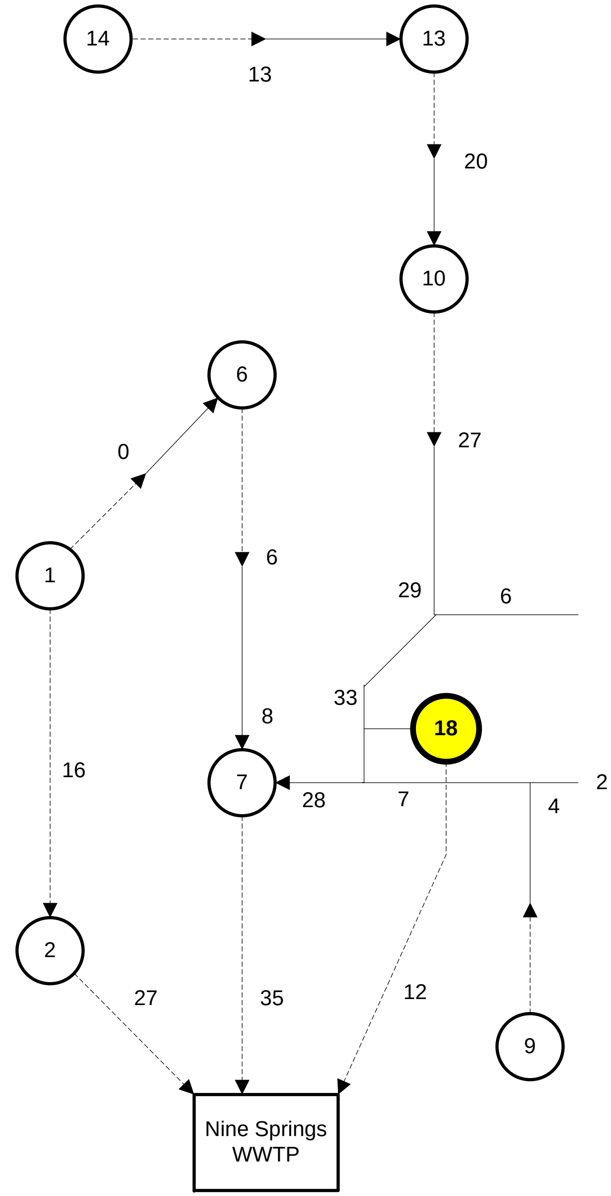
Figure 3 - Preliminary Route for Northeast Interceptor Relief Sewer



Alternative 1
PS 18 = 54 mgd peak capacity



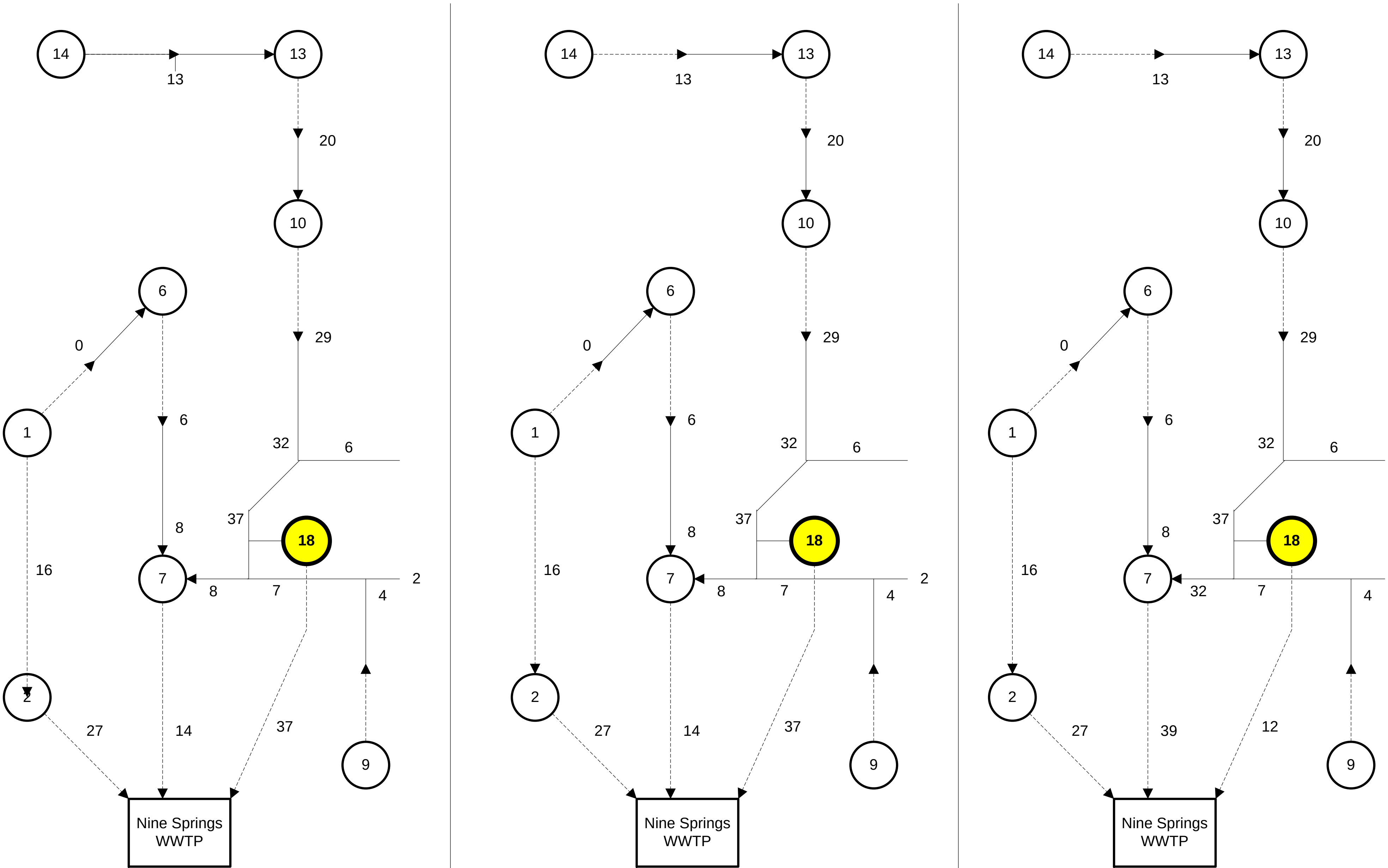
Alternative 2
PS 18 = 37 mgd peak capacity



Alternative 3
PS 18 = Northeast Interceptor Average Daily Flow

Figure 4 – Preliminary Peak Design Flow Schematic for Pump Station 18 Alternatives (2010 Flows)

Peak flows derived from Madison Design Curve

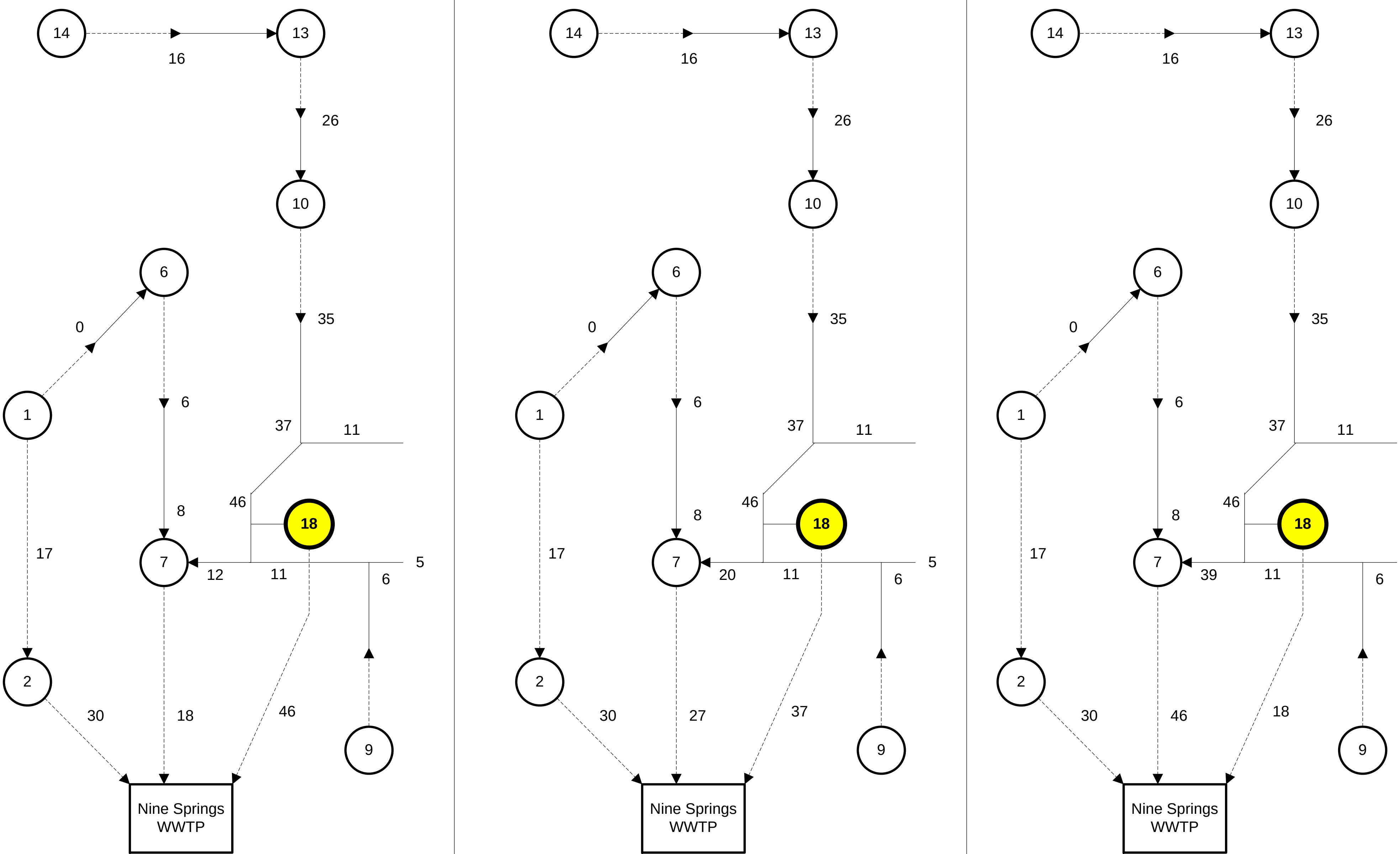


Alternative 1
PS 18 = 66 mgd peak capacity

Alternative 2
PS 18 = 44 mgd peak capacity

Alternative 3
PS 18 = Northeast Interceptor
Average Daily Flow

Figure 4A – Preliminary Peak Design Flow Schematic for Pump Station 18 Alternatives (2010 Flows)
Peak flows derived from 'Modified' Madison Design Curve



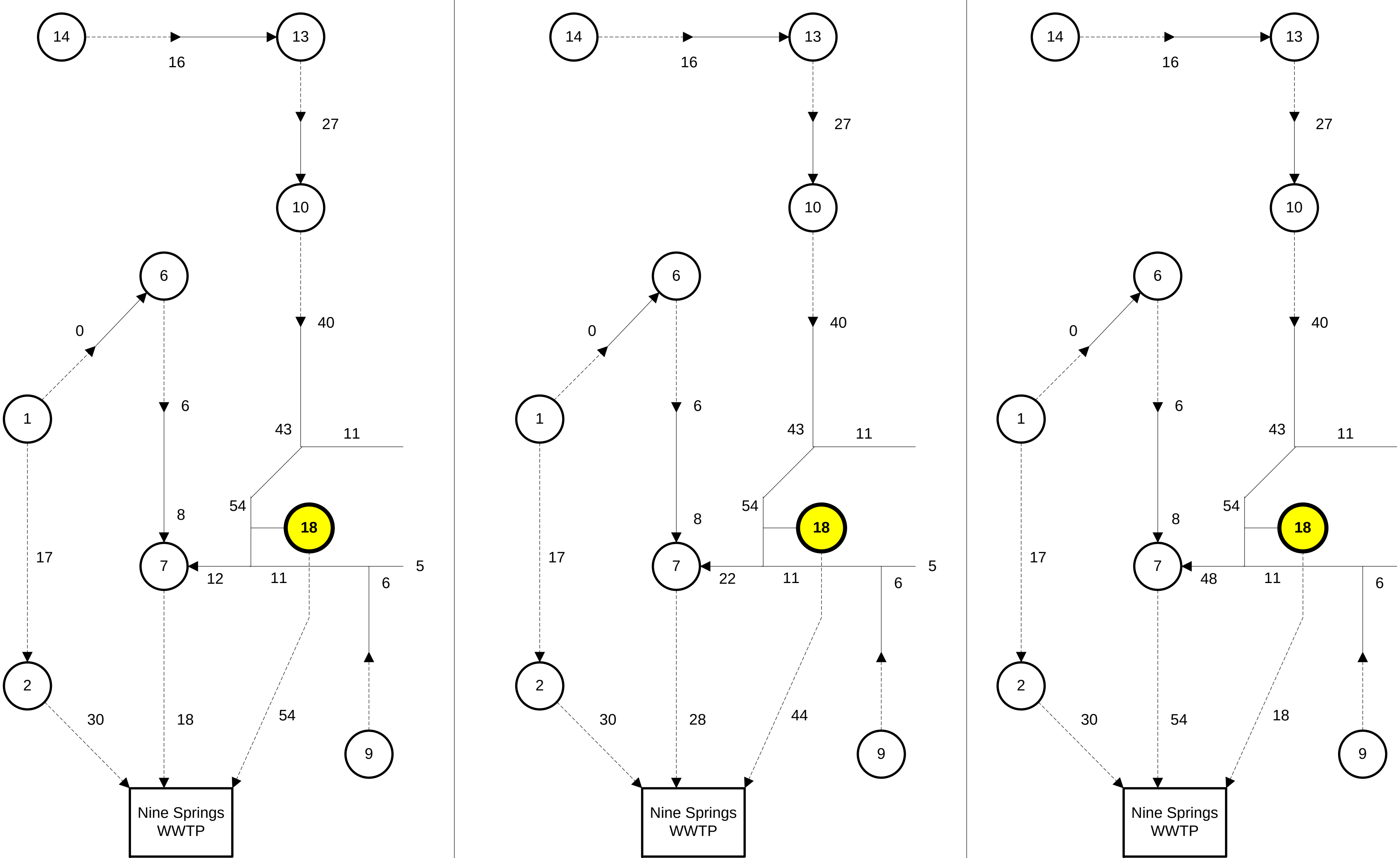
Alternative 1
PS 18 = 54 mgd peak capacity

Alternative 2
PS 18 = 37 mgd peak capacity

Alternative 3
PS 18 = Northeast Interceptor
Average Daily Flow

Figure 5 – Preliminary Peak Design Flow Schematic for Pump Station 18 Alternatives (2030 Flows)

Peak flows derived from Madison Design Curve



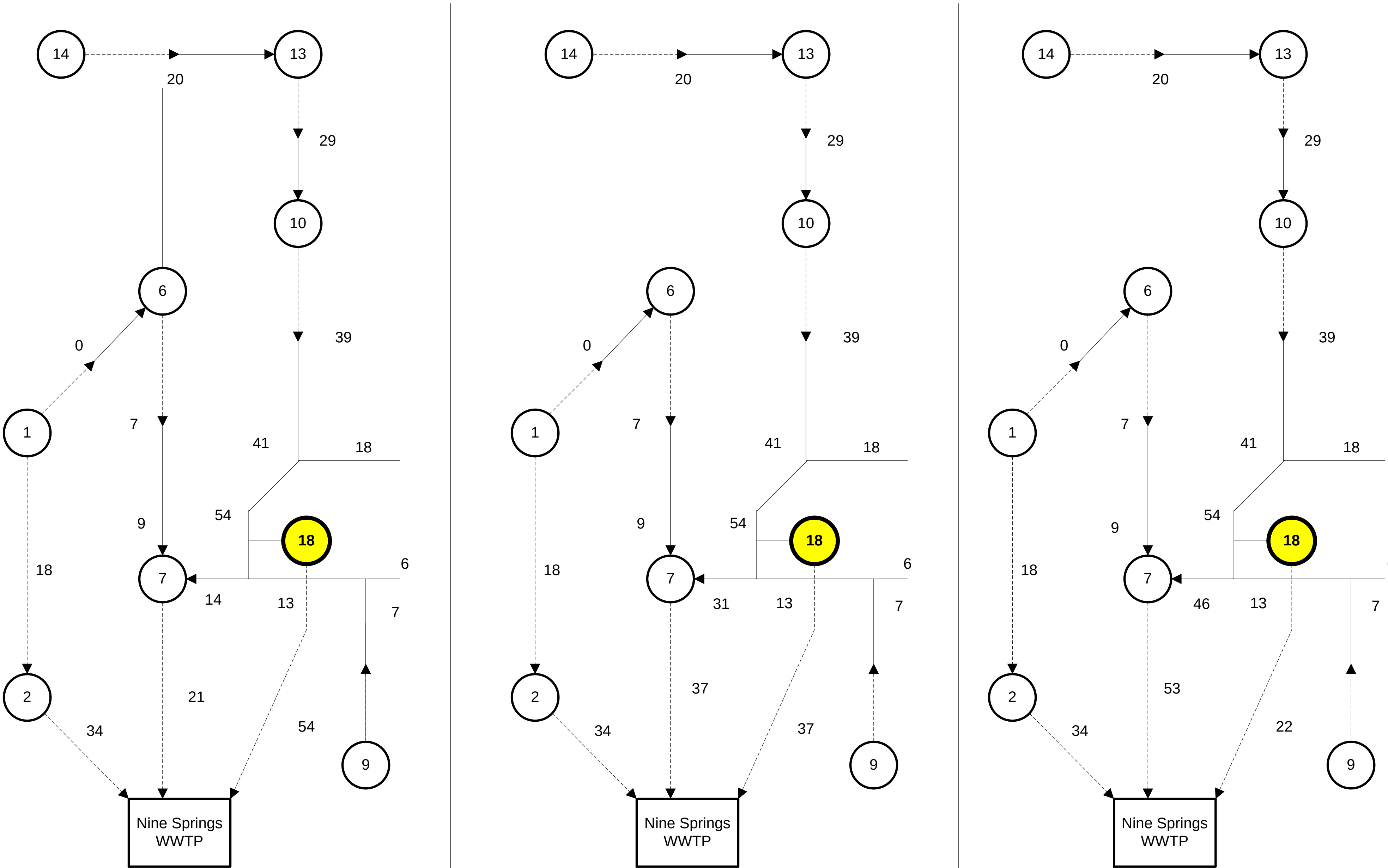
Alternative 1
PS 18 = 66 mgd peak capacity

Alternative 2
PS 18 = 44 mgd peak capacity

Alternative 3
PS 18 = Northeast Interceptor Average Daily Flow

Figure 5A – Preliminary Peak Design Flow Schematic for Pump Station 18 Alternatives (2030 Flows)

Peak flows derived from 'Modified' Madison Design Curve



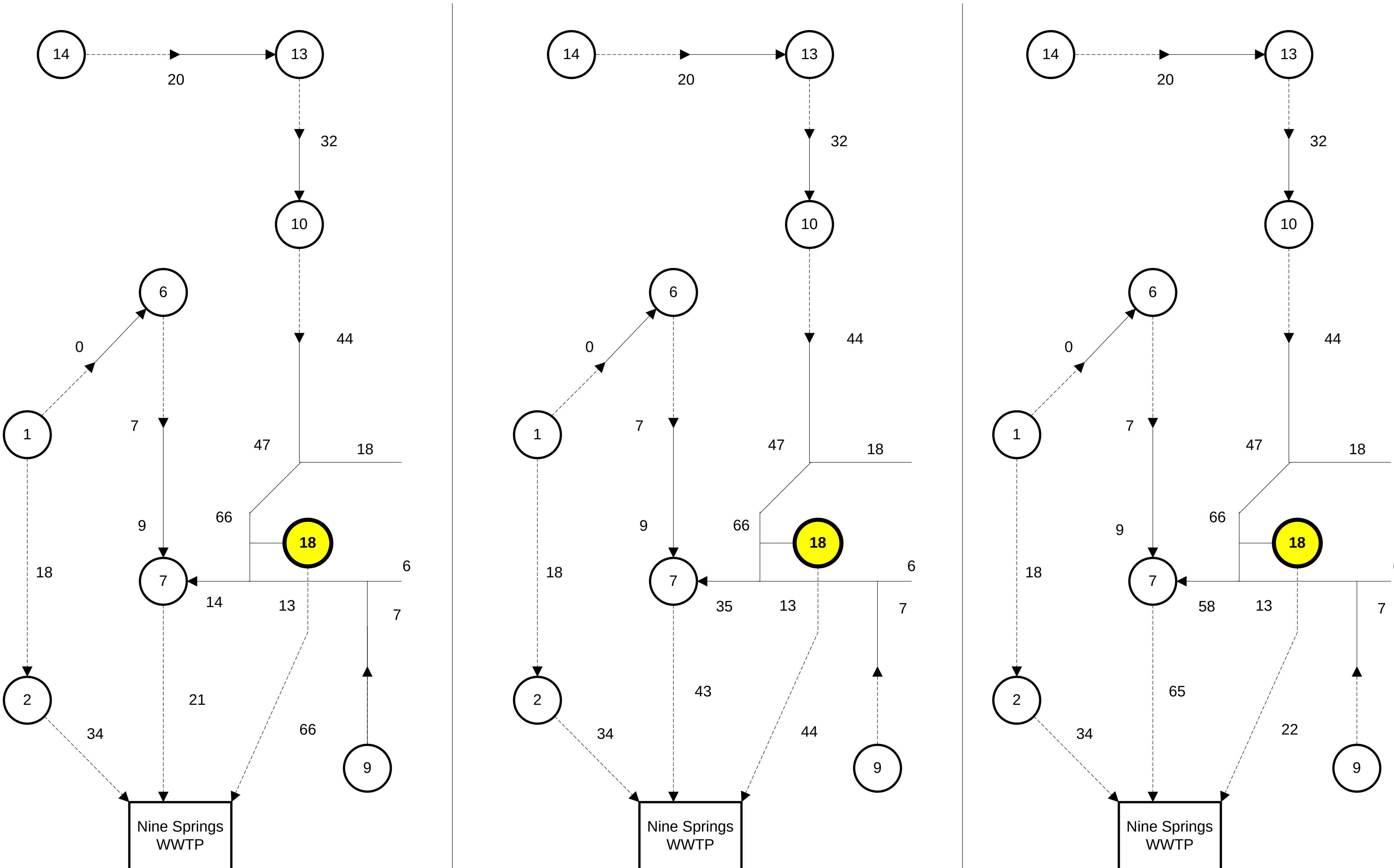
Alternative 1 – PS 18 = 54 mgd peak capacity

Alternative 2 – PS 18 = 37 mgd peak capacity

Alternative 3 – PS 18 = Northeast Interceptor Average Daily Flow

Figure 6 – Preliminary Peak Design Flow Schematic for Pump Station 18 Alternatives (2060 Flows)

Peak flows derived from Madison Design Curve



Alternative 1 – PS 18 = 66 mgd peak capacity

Alternative 2 – PS 18 = 44 mgd peak capacity

Alternative 3 – PS 18 = Northeast Interceptor Average Daily Flow

Figure 6A – Preliminary Peak Design Flow Schematic for Pump Station 18 Alternatives (2060 Flows)

Peak flows derived from 'Modified' Madison Design Curve

TABLE 5 - EMERGENCY DIVERSION FROM PS7 TO PS18 (MH07-301)
Existing Conditions

FIND: Wet well elevations at Pump Station No. 7 for various rates of "backflow" from PS 7 to PS18 (MH07-301

I. PHYSICAL CHARACTERISICS OF DIVERSION

NEI Section		**SEI Section**	
Length, L, of 48" overflow (ft) =	596	Length, L, of 60" overflow (ft) =	7,810
Pipe diameter, D (ft) =	4.00	Pipe diameter, D (ft) =	5.00
Pipe area, A (ft^2) =	12.566	Pipe area, A (ft^2) =	19.635
Hydraulic radius, R (ft) =	1.00	Hydraulic radius, R (ft) =	1.25
Manning's n =	0.013	Manning's n =	0.013

Water surface elevation at MH07-301 = -4.56 (assume 48" NEI is flowing full)

II. CALCULATE FLOW BY MANNING'S EQUATION

$Q = (1.49/n) * A*R^{2/3} *S^{1/2}$

$\Delta H = ((Q*n)/(1.49*A*R^{2/3}))^2 * L$

Diversion Flow, Q (mgd)	Diversion Flow, Q (cfs)	Head Loss, ΔH, in NEI Section (ft)	Water Surface Elevation at MH07-215 (ft)	Head Loss, ΔH, in SEI Section (ft)	Water Surface Elevation at PS7 (ft)
53.0	81.99	1.93	-2.63	7.69	5.06
52.0	80.44	1.86	-2.70	7.40	4.70
51.0	78.90	1.79	-2.77	7.12	4.35
50.0	77.35	1.72	-2.84	6.84	4.00
49.0	75.80	1.65	-2.91	6.57	3.66
48.0	74.26	1.58	-2.98	6.31	3.33
47.0	72.71	1.52	-3.04	6.05	3.00
46.0	71.16	1.45	-3.11	5.79	2.69
45.0	69.62	1.39	-3.17	5.54	2.37
44.0	68.07	1.33	-3.23	5.30	2.07
43.0	66.52	1.27	-3.29	5.06	1.77
42.0	64.97	1.21	-3.35	4.83	1.48
41.0	63.43	1.16	-3.40	4.60	1.20
40.0	61.88	1.10	-3.46	4.38	0.92
39.0	60.33	1.05	-3.51	4.16	0.65
38.0	58.79	0.99	-3.57	3.95	0.38
37.0	57.24	0.94	-3.62	3.75	0.13
36.0	55.69	0.89	-3.67	3.55	-0.12
35.0	54.15	0.84	-3.72	3.35	-0.37
34.0	52.60	0.79	-3.77	3.16	-0.60
33.0	51.05	0.75	-3.81	2.98	-0.83
32.0	49.50	0.70	-3.86	2.80	-1.05
31.0	47.96	0.66	-3.90	2.63	-1.27
30.0	46.41	0.62	-3.94	2.46	-1.48
29.0	44.86	0.58	-3.98	2.30	-1.68
28.0	43.32	0.54	-4.02	2.15	-1.88
27.0	41.77	0.50	-4.06	2.00	-2.06
26.0	40.22	0.46	-4.10	1.85	-2.25
25.0	38.68	0.43	-4.13	1.71	-2.42
24.0	37.13	0.40	-4.16	1.58	-2.59
23.0	35.58	0.36	-4.20	1.45	-2.75
22.0	34.03	0.33	-4.23	1.32	-2.90
21.0	32.49	0.30	-4.26	1.21	-3.05
20.0	30.94	0.28	-4.28	1.09	-3.19
19.0	29.39	0.25	-4.31	0.99	-3.32
18.0	27.85	0.22	-4.34	0.89	-3.45
17.0	26.30	0.20	-4.36	0.79	-3.57
16.0	24.75	0.18	-4.38	0.70	-3.68
15.0	23.21	0.15	-4.41	0.62	-3.79
14.0	21.66	0.13	-4.43	0.54	-3.89
13.0	20.11	0.12	-4.44	0.46	-3.98

TABLE 5 - EMERGENCY DIVERSION FROM PS7 TO PS18
Proposed Conditions

FIND: Wet well elevations at Pump Station No. 7 for various rates of "backflow" from PS 7 to PS18

I. PHYSICAL CHARACTERISICS OF DIVERSION

PS18 Overflow		**SEI Section**	
Length, L, of 60" overflow (ft) =	620	Length, L, of 60" overflow (ft) =	7,810
Pipe diameter, D (ft) =	5.00	Pipe diameter, D (ft) =	5.00
Pipe area, A (ft^2) =	19.635	Pipe area, A (ft^2) =	19.635
Hydraulic radius, R (ft) =	1.25	Hydraulic radius, R (ft) =	1.25
Manning's n =	0.013	Manning's n =	0.013

Water surface elevation at PS18 = -4.56

II. CALCULATE FLOW BY MANNING'S EQUATION

$Q = (1.49/n) * A*R^{2/3} *S^{1/2}$

$\Delta H = ((Q*n)/(1.49*A*R^{2/3}))^2 * L$

Diversion Flow, Q (mgd)	Diversion Flow, Q (cfs)	Head Loss, ΔH, in NEI Section (ft)	Water Surface Elevation at MH07-215 (ft)	Head Loss, ΔH, in SEI Section (ft)	Water Surface Elevation at PS7 (ft)
53.0	81.99	0.61	-3.95	7.69	3.74
52.0	80.44	0.59	-3.97	7.40	3.43
51.0	78.90	0.57	-3.99	7.12	3.12
50.0	77.35	0.54	-4.02	6.84	2.82
49.0	75.80	0.52	-4.04	6.57	2.53
48.0	74.26	0.50	-4.06	6.31	2.25
47.0	72.71	0.48	-4.08	6.05	1.97
46.0	71.16	0.46	-4.10	5.79	1.69
45.0	69.62	0.44	-4.12	5.54	1.42
44.0	68.07	0.42	-4.14	5.30	1.16
43.0	66.52	0.40	-4.16	5.06	0.90
42.0	64.97	0.38	-4.18	4.83	0.65
41.0	63.43	0.37	-4.19	4.60	0.41
40.0	61.88	0.35	-4.21	4.38	0.17
39.0	60.33	0.33	-4.23	4.16	-0.07
38.0	58.79	0.31	-4.25	3.95	-0.29
37.0	57.24	0.30	-4.26	3.75	-0.52
36.0	55.69	0.28	-4.28	3.55	-0.73
35.0	54.15	0.27	-4.29	3.35	-0.94
34.0	52.60	0.25	-4.31	3.16	-1.15
33.0	51.05	0.24	-4.32	2.98	-1.34
32.0	49.50	0.22	-4.34	2.80	-1.54
31.0	47.96	0.21	-4.35	2.63	-1.72
30.0	46.41	0.20	-4.36	2.46	-1.90
29.0	44.86	0.18	-4.38	2.30	-2.08
28.0	43.32	0.17	-4.39	2.15	-2.24
27.0	41.77	0.16	-4.40	2.00	-2.41
26.0	40.22	0.15	-4.41	1.85	-2.56
25.0	38.68	0.14	-4.42	1.71	-2.71
24.0	37.13	0.13	-4.43	1.58	-2.86
23.0	35.58	0.11	-4.45	1.45	-3.00
22.0	34.03	0.11	-4.45	1.32	-3.13
21.0	32.49	0.10	-4.46	1.21	-3.26
20.0	30.94	0.09	-4.47	1.09	-3.38
19.0	29.39	0.08	-4.48	0.99	-3.49
18.0	27.85	0.07	-4.49	0.89	-3.60
17.0	26.30	0.06	-4.50	0.79	-3.71
16.0	24.75	0.06	-4.50	0.70	-3.80
15.0	23.21	0.05	-4.51	0.62	-3.90
14.0	21.66	0.04	-4.52	0.54	-3.98
13.0	20.11	0.04	-4.52	0.46	-4.06

Table 8A-1
Peak Hourly Flows for PS18 and PS7 - Alternative 1 (Madison Design Curve)

			2010 U.F.				2030 U.F.				2060			
Pumping Station Sub Basin	From	To	Sub-Basin Flow (gpd)	Cumulative Flow (MGD)	Peak Factor	Cumulative Peak Flow (MGD)	Sub-Basin Flow (gpd)	Cumulative Flow (MGD)	Peak Factor	Cumulative Peak Flow (MGD)	Sub-Basin Flow (gpd)	Cumulative Flow (MGD)	Peak Factor	Cumulative Peak Flow (MGD)
14-A	MH 14-209	MH14-196	498,879	0.50	4.0	2.00	589,606	0.59	4.0	2.36	772,585	0.77	4.0	3.09
14-B	MH14-196	MH14-193	249,667	0.75	4.0	2.99	312,984	0.90	4.0	3.61	588,677	1.36	3.8	5.19
14-C	MH14-193	MH14-182	62,225	0.81	4.0	3.24	97,850	1.00	4.0	4.00	97,850	1.46	3.8	5.50
14-D	MH14-182	MH14-171	49,884	0.86	4.0	3.44	95,650	1.10	3.9	4.32	95,650	1.55	3.7	5.80
14-E	MH14-171	MH14-166	38,588	0.90	4.0	3.60	38,534	1.13	3.9	4.45	38,534	1.59	3.7	5.92
14-F	MH14-166	MH14-162	198,077	1.10	3.9	4.33	278,829	1.41	3.8	5.35	440,112	2.03	3.6	7.27
14-G	MH14-162	MH14-156	47,461	1.14	3.9	4.48	116,120	1.53	3.7	5.72	116,120	2.15	3.5	7.62
14-H	MH14-156	MH14-143	241,874	1.39	3.8	5.27	257,963	1.79	3.6	6.52	257,963	2.41	3.5	8.38
14-I	MH14-143	MH14-134	64,346	1.45	3.8	5.47	101,606	1.89	3.6	6.83	132,023	2.54	3.5	8.77
14-J	MH 14-416	MH14-134	308,576	0.31	4.0	1.23	519,368	0.52	4.0	2.08	624,919	0.62	4.0	2.50
14-K	MH14-134	MH14-102	53,627	1.81	3.6	6.60	66,727	2.48	3.5	8.58	66,727	3.23	3.3	10.74
14-Q	MH14-362	MH14-358	356,101	0.36	4.0	1.42	395,964	0.40	4.0	1.58	450,369	0.45	4.0	1.80
14-L	MH14-359	MH14-358	621,271	0.62	4.0	2.49	811,364	0.81	4.0	3.25	1,074,825	1.07	4.0	4.25
	MH14-358	MH14-356		0.98	4.0	3.91		1.21	3.9	4.69		1.53	3.7	5.71
14-M	MH14-356	MH14-323	429,812	1.41	3.8	5.33	747,196	1.95	3.6	7.03	1,094,496	2.62	3.4	9.00
14-N	MH14-323	MH14-315	153,514	1.56	3.7	5.82	204,977	2.16	3.5	7.65	261,387	2.88	3.4	9.75
14-O	MH14-315	MH14-102	194,823	1.76	3.7	6.42	214,995	2.37	3.5	8.28	305,002	3.19	3.3	10.61
	MH14-102	MH14-101		3.57	3.3	11.68		4.85	3.1	15.12		6.42	3.0	19.14
14-P	MH14-101	PS 14	400,678	3.97	3.2	12.77	409,746	5.26	3.1	16.18	409,746	6.83	3.0	20.16
PS 14				3.97	3.2	12.77		5.26	3.1	16.18		6.83	3.0	20.16
	PS 14	TE14-11057		3.97	3.2	12.77		5.26	3.1	16.18		6.83	3.0	20.16
13-F	TE14-11057	MH13-132	265,790	4.24	3.2	13.49	275,917	5.54	3.1	16.90	275,917	7.10	2.9	20.84
13-G	MH13-132	MH13-122A	122,964	4.36	3.2	13.82	160,919	5.70	3.0	17.31	160,919	7.26	2.9	21.24
13-A	MH13-122A	MH13-105A	351,739				367,673				367,673			
13-B	MH13-122A	MH13-105A	49,458				66,873				66,873			
13-C	MH13-122A	MH13-105A	639,164				730,012				730,012			
13-D	MH13-122A	MH13-105A	708,753				726,821				726,821			
13-E	MH13-122A	MH13-105A	188,234	6.30	3.0	18.83	196,939	7.78	2.9	22.52	196,939	9.35	2.8	26.28
13-H	MH13-105A	PS 13	468,068	6.76	3.0	20.00	1,353,883	9.14	2.8	25.77	1,353,883	10.71	2.8	29.44
PS 13				6.76	3.0	20.00		9.14	2.8	25.77		10.71	2.8	29.44
	PS 13	MH10-145		6.76	3.0	20.00		9.14	2.8	25.77		10.71	2.8	29.44
10-A	MH10-145	MH10-121	932,249	7.70	2.9	22.30	1,149,110	10.29	2.8	28.47	1,149,110	11.86	2.7	32.08
10-B	MH10-121	MH10-201	412,216	8.11	2.9	23.30	461,286	10.75	2.7	29.54	461,286	12.32	2.7	33.13
10-C	MH10-220	MH10-214	325,867	0.33	4.0	1.30	964,209	0.96	4.0	3.86	964,209	0.96	4.0	3.86
10-D	MH10-214	MH10-201	392,316	0.72	4.0	2.87	554,722	1.52	3.7	5.69	554,722	1.52	3.7	5.69
	MH10-201	MH10-115		8.83	2.8	25.03		12.27	2.7	33.02		13.84	2.6	36.54
10-E	MH10-115	MH10-104A	173,558	9.00	2.8	25.44	185,986	12.45	2.7	33.44	185,986	14.02	2.6	36.95
10-F	MH10-305	MH10-104A	188,221	0.19	4.0	0.75	190,971	0.19	4.0	0.76	190,971	0.19	4.0	0.76
	MH10-104A	MH10-102A		9.19	2.8	25.89		12.64	2.7	33.87		14.21	2.6	37.38
10-G	MH10-102A	MH10-101	11,479	9.20	2.8	25.91	17,319	12.66	2.7	33.91	17,319	14.23	2.6	37.42
10-H	MH10-101	PS 10	579,684	9.78	2.8	27.28	599,396	13.26	2.7	35.26	599,396	14.83	2.6	38.74
PS 10				9.78	2.8	27.28		13.26	2.7	35.26		14.83	2.6	38.74
	PS 10	MH07-955		9.78	2.8	27.28		13.26	2.7	35.26		14.83	2.6	38.74

Table 8A-1
Peak Hourly Flows for PS18 and PS7 - Alternative 1 (Madison Design Curve)

Pumping Station Sub Basin	From	To	2010 U.F. Sub-Basin Flow (gpd)	2010 U.F. Cumulative Flow (MGD)	2010 U.F. Peak Factor	2010 U.F. Cumulative Peak Flow (MGD)	2030 U.F. Sub-Basin Flow (gpd)	2030 U.F. Cumulative Flow (MGD)	2030 U.F. Peak Factor	2030 U.F. Cumulative Peak Flow (MGD)	2060 Sub-Basin Flow (gpd)	2060 Cumulative Flow (MGD)	2060 Peak Factor	2060 Cumulative Peak Flow (MGD)
7-A	MH07-955	MH07-932	871,342	10.65	2.8	29.32	980,335	14.24	2.6	37.44	980,335	15.81	2.6	40.88
7-C	MH07-734	MH07-426	693,680	0.69	4.0	2.77	1,989,624	1.99	3.6	7.14	3,771,162	3.77	3.2	12.23
7-B	MH07-437	MH07-426	550,457	0.55	4.0	2.20	1,018,340	1.02	4.0	4.06	1,566,335	1.57	3.7	5.84
7-D	MH07-426	MH07-415	157,183	1.40	3.8	5.31	357,817	3.37	3.3	11.11	357,817	5.70	3.0	17.31
7-E	MH07-415	MH07-932	85,825	1.49	3.8	5.59	116,336	3.48	3.3	11.44	116,336	5.81	3.0	17.60
7-F	MH07-932	MH07-215	213,427	12.35	2.7	33.21	226,623	17.95	2.5	45.50	226,623	21.85	2.5	53.68
PS 18 - Alternative 1				12.35	2.7	33.21		17.95	2.5	45.50		21.85	2.5	53.68
	PS 18	WWTP		12.35	2.7	33.21		17.95	2.5	45.50		21.85	2.5	53.68
7-J	MH07-249	MH07-228	518,417	0.52	4.0	2.07	1,368,622	1.37	3.8	5.21	1,734,576	1.73	3.7	6.36
9-A	MH09-108	MH09-104	647,586	0.65	4.0	2.59	918,416	0.92	4.0	3.67	1,380,367	1.38	3.8	5.25
9-B	MH09-104	PS 9	317,105	0.96	4.0	3.86	364,702	1.28	3.8	4.93	364,702	1.75	3.7	6.39
PS 9				0.96	4.0	3.86		1.28	3.8	4.93		1.75	3.7	6.39
	PS 9	MH07-517		0.96	4.0	3.86		1.28	3.8	4.93		1.75	3.7	6.39
7-G	MH07-517	MH07-512	10,080	0.97	4.0	3.90	25,880	1.31	3.8	5.02	25,880	1.77	3.7	6.47
7-H	MH07-618	MH07-512	77,097	0.08	4.0	0.31	141,857	0.14	4.0	0.57	141,857	0.14	4.0	0.57
7-I	MH07-512	MH07-228	56,267	1.11	3.9	4.36	141,304	1.59	3.7	5.92	141,304	2.05	3.6	7.33
	MH07-228	MH07-224		1.63	3.7	6.02		2.96	3.4	9.98		3.79	3.2	12.28
7-K	MH07-224	MH07-218	121,062	1.75	3.7	6.40	156,277	3.12	3.3	10.42	156,277	3.94	3.2	12.70
7-L	MH07-823	MH07-218	94,512	0.09	4.0	0.38	104,614	0.10	4.0	0.42	104,614	0.10	4.0	0.42
	MH07-218	MH07-215		1.84	3.6	6.69		3.22	3.3	10.71		4.05	3.2	12.99
	MH07-215	MH07-211		1.84	3.6	6.69		3.22	3.3	10.71		4.05	3.2	12.99
7-M	MH07-211	PS 7	305,045	2.15	3.5	7.61	350,317	3.57	3.3	11.68	350,317	4.40	3.2	13.93
6-A	MH06-209	MH06-108A	180,399	0.18	4.0	0.72	178,257	0.18	4.0	0.71	196,459	0.20	4.0	0.79
6-B	MH06-122	MH06-108A	156,634	0.16	4.0	0.63	201,410	0.20	4.0	0.81	209,378	0.21	4.0	0.84
6-C	MH06-108A	PS 6	36,339	0.37	4.0	1.49	35,643	0.42	4.0	1.66	44,024	0.45	4.0	1.80
6-D	NA	PS 6	1,235,750	1.24	3.9	4.78	1,321,888	1.32	3.8	5.06	1,540,062	1.54	3.7	5.75
PS 6				1.61	3.7	5.97		1.74	3.7	6.37		1.99	3.6	7.14
	PS 6	MH07-129		1.61	3.7	5.97		1.74	3.7	6.37		1.99	3.6	7.14
7-N	MH07-129	PS 7	675,724	2.28	3.5	8.02	682,620	2.42	3.5	8.42	682,620	2.67	3.4	9.15
PS 7 - Alternative 1				4.43	3.2	14.01		5.99	3.0	18.06		7.07	2.9	20.77
	PS 7	WWTP		4.43	3.2	14.01		5.99	3.0	18.06		7.07	2.9	20.77

Table 8A-2

Peak Hourly Flows for PS18 and PS7 - Alternative 2 (Madison Design Curve)

Pumping Station Sub Basin	From	To	2010 U.F. Sub-Basin Flow (gpd)	2010 U.F. Cumulative Flow (MGD)	2010 U.F. Peak Factor	2010 U.F. Cumulative Peak Flow (MGD)	2030 U.F. Sub-Basin Flow (gpd)	2030 U.F. Cumulative Flow (MGD)	2030 U.F. Peak Factor	2030 U.F. Cumulative Peak Flow (MGD)	2060 Sub-Basin Flow (gpd)	2060 Cumulative Flow (MGD)	2060 Peak Factor	2060 Cumulative Peak Flow (MGD)
14-A	MH 14-209	MH14-196	498,879	0.50	4.0	2.00	589,606	0.59	4.0	2.36	772,585	0.77	4.0	3.09
14-B	MH14-196	MH14-193	249,667	0.75	4.0	2.99	312,984	0.90	4.0	3.61	588,677	1.36	3.8	5.19
14-C	MH14-193	MH14-182	62,225	0.81	4.0	3.24	97,850	1.00	4.0	4.00	97,850	1.46	3.8	5.50
14-D	MH14-182	MH14-171	49,884	0.86	4.0	3.44	95,650	1.10	3.9	4.32	95,650	1.55	3.7	5.80
14-E	MH14-171	MH14-166	38,588	0.90	4.0	3.60	38,534	1.13	3.9	4.45	38,534	1.59	3.7	5.92
14-F	MH14-166	MH14-162	198,077	1.10	3.9	4.33	278,829	1.41	3.8	5.35	440,112	2.03	3.6	7.27
14-G	MH14-162	MH14-156	47,461	1.14	3.9	4.48	116,120	1.53	3.7	5.72	116,120	2.15	3.5	7.62
14-H	MH14-156	MH14-143	241,874	1.39	3.8	5.27	257,963	1.79	3.6	6.52	257,963	2.41	3.5	8.38
14-I	MH14-143	MH14-134	64,346	1.45	3.8	5.47	101,606	1.89	3.6	6.83	132,023	2.54	3.5	8.77
14-J	MH 14-416	MH14-134	308,576	0.31	4.0	1.23	519,368	0.52	4.0	2.08	624,919	0.62	4.0	2.50
14-K	MH14-134	MH14-102	53,627	1.81	3.6	6.60	66,727	2.48	3.5	8.58	66,727	3.23	3.3	10.74
14-Q	MH14-362	MH14-358	356,101	0.36	4.0	1.42	395,964	0.40	4.0	1.58	450,369	0.45	4.0	1.80
14-L	MH14-359	MH14-358	621,271	0.62	4.0	2.49	811,364	0.81	4.0	3.25	1,074,825	1.07	4.0	4.25
	MH14-358	MH14-356		0.98	4.0	3.91		1.21	3.9	4.69		1.53	3.7	5.71
14-M	MH14-356	MH14-323	429,812	1.41	3.8	5.33	747,196	1.95	3.6	7.03	1,094,496	2.62	3.4	9.00
14-N	MH14-323	MH14-315	153,514	1.56	3.7	5.82	204,977	2.16	3.5	7.65	261,387	2.88	3.4	9.75
14-O	MH14-315	MH14-102	194,823	1.76	3.7	6.42	214,995	2.37	3.5	8.28	305,002	3.19	3.3	10.61
	MH14-102	MH14-101		3.57	3.3	11.68		4.85	3.1	15.12		6.42	3.0	19.14
14-P	MH14-101	PS 14	400,678	3.97	3.2	12.77	409,746	5.26	3.1	16.18	409,746	6.83	3.0	20.16
PS 14				3.97	3.2	12.77		5.26	3.1	16.18		6.83	3.0	20.16
	PS 14	TE14-11057		3.97	3.2	12.77		5.26	3.1	16.18		6.83	3.0	20.16
13-F	TE14-11057	MH13-132	265,790	4.24	3.2	13.49	275,917	5.54	3.1	16.90	275,917	7.10	2.9	20.84
13-G	MH13-132	MH13-122A	122,964	4.36	3.2	13.82	160,919	5.70	3.0	17.31	160,919	7.26	2.9	21.24
13-A	MH13-122A	MH13-105A	351,739				367,673				367,673			
13-B	MH13-122A	MH13-105A	49,458				66,873				66,873			
13-C	MH13-122A	MH13-105A	639,164				730,012				730,012			
13-D	MH13-122A	MH13-105A	708,753				726,821				726,821			
13-E	MH13-122A	MH13-105A	188,234	6.30	3.0	18.83	196,939	7.78	2.9	22.52	196,939	9.35	2.8	26.28
13-H	MH13-105A	PS 13	468,068	6.76	3.0	20.00	1,353,883	9.14	2.8	25.77	1,353,883	10.71	2.8	29.44
PS 13				6.76	3.0	20.00		9.14	2.8	25.77		10.71	2.8	29.44
	PS 13	MH10-145		6.76	3.0	20.00		9.14	2.8	25.77		10.71	2.8	29.44
10-A	MH10-145	MH10-121	932,249	7.70	2.9	22.30	1,149,110	10.29	2.8	28.47	1,149,110	11.86	2.7	32.08
10-B	MH10-121	MH10-201	412,216	8.11	2.9	23.30	461,286	10.75	2.7	29.54	461,286	12.32	2.7	33.13
10-C	MH10-220	MH10-214	325,867	0.33	4.0	1.30	964,209	0.96	4.0	3.86	964,209	0.96	4.0	3.86
10-D	MH10-214	MH10-201	392,316	0.72	4.0	2.87	554,722	1.52	3.7	5.69	554,722	1.52	3.7	5.69
	MH10-201	MH10-115		8.83	2.8	25.03		12.27	2.7	33.02		13.84	2.6	36.54
10-E	MH10-115	MH10-104A	173,558	9.00	2.8	25.44	185,986	12.45	2.7	33.44	185,986	14.02	2.6	36.95
10-F	MH10-305	MH10-104A	188,221	0.19	4.0	0.75	190,971	0.19	4.0	0.76	190,971	0.19	4.0	0.76
	MH10-104A	MH10-102A		9.19	2.8	25.89		12.64	2.7	33.87		14.21	2.6	37.38
10-G	MH10-102A	MH10-101	11,479	9.20	2.8	25.91	17,319	12.66	2.7	33.91	17,319	14.23	2.6	37.42
10-H	MH10-101	PS 10	579,684	9.78	2.8	27.28	599,396	13.26	2.7	35.26	599,396	14.83	2.6	38.74
PS 10				9.78	2.8	27.28		13.26	2.7	35.26		14.83	2.6	38.74
	PS 10	MH07-955		9.78	2.8	27.28		13.26	2.7	35.26		14.83	2.6	38.74

Table 8A-2
Peak Hourly Flows for PS18 and PS7 - Alternative 2 (Madison Design Curve)

Pumping Station Sub Basin	From	To	2010 U.F. Sub-Basin Flow (gpd)	2010 U.F. Cumulative Flow (MGD)	2010 U.F. Peak Factor	2010 U.F. Cumulative Peak Flow (MGD)	2030 U.F. Sub-Basin Flow (gpd)	2030 U.F. Cumulative Flow (MGD)	2030 U.F. Peak Factor	2030 U.F. Cumulative Peak Flow (MGD)	2060 Sub-Basin Flow (gpd)	2060 Cumulative Flow (MGD)	2060 Peak Factor	2060 Cumulative Peak Flow (MGD)
7-A	MH07-955	MH07-932	871,342	10.65	2.8	29.32	980,335	14.24	2.6	37.44	980,335	15.81	2.6	40.88
7-C	MH07-734	MH07-426	693,680	0.69	4.0	2.77	1,989,624	1.99	3.6	7.14	3,771,162	3.77	3.2	12.23
7-B	MH07-437	MH07-426	550,457	0.55	4.0	2.20	1,018,340	1.02	4.0	4.06	1,566,335	1.57	3.7	5.84
7-D	MH07-426	MH07-415	157,183	1.40	3.8	5.31	357,817	3.37	3.3	11.11	357,817	5.70	3.0	17.31
7-E	MH07-415	MH07-932	85,825	1.49	3.8	5.59	116,336	3.48	3.3	11.44	116,336	5.81	3.0	17.60
7-F	MH07-932	MH07-215	213,427	12.35	2.7	33.21	226,623	17.95	2.5	45.50	226,623	21.85	2.5	53.68
				12.35	2.7	33.21		17.95	2.5	45.50		21.85	2.5	53.68
	PS 18	WWTP		12.35	2.7	33.21		17.95	2.5	45.50		21.85	2.5	53.68
PS 18 - Alternative 2				12.41	2.7	33.21		14.12	2.6	37.00		14.12	2.6	37.00
	PS 18	WWTP		12.41	2.7	33.21		14.12	2.6	37.00		14.12	2.6	37.00
Excess peak flow to PS7						0.00				8.50				16.68
7-J	MH07-249	MH07-228	518,417	0.52	4.0	2.07	1,368,622	1.37	3.8	5.21	1,734,576	1.73	3.7	6.36
9-A	MH09-108	MH09-104	647,586	0.65	4.0	2.59	918,416	0.92	4.0	3.67	1,380,367	1.38	3.8	5.25
9-B	MH09-104	PS 9	317,105	0.96	4.0	3.86	364,702	1.28	3.8	4.93	364,702	1.75	3.7	6.39
PS 9				0.96	4.0	3.86		1.28	3.8	4.93		1.75	3.7	6.39
	PS 9	MH07-517		0.96	4.0	3.86		1.28	3.8	4.93		1.75	3.7	6.39
7-G	MH07-517	MH07-512	10,080	0.97	4.0	3.90	25,880	1.31	3.8	5.02	25,880	1.77	3.7	6.47
7-H	MH07-618	MH07-512	77,097	0.08	4.0	0.31	141,857	0.14	4.0	0.57	141,857	0.14	4.0	0.57
7-I	MH07-512	MH07-228	56,267	1.11	3.9	4.36	141,304	1.59	3.7	5.92	141,304	2.05	3.6	7.33
	MH07-228	MH07-224		1.63	3.7	6.02		2.96	3.4	9.98		3.79	3.2	12.28
7-K	MH07-224	MH07-218	121,062	1.75	3.7	6.40	156,277	3.12	3.3	10.42	156,277	3.94	3.2	12.70
7-L	MH07-823	MH07-218	94,512	0.09	4.0	0.38	104,614	0.10	4.0	0.42	104,614	0.10	4.0	0.42
	MH07-218	MH07-215		1.84	3.6	6.69		3.22	3.3	10.71		4.05	3.2	12.99
	MH07-215	MH07-211		1.84	3.6	6.69		3.22	3.3	19.21		4.05	3.2	29.67
7-M	MH07-211	PS 7	305,045	2.15	3.5	7.61	350,317	3.57	3.3	20.18	350,317	4.40	3.2	30.61
6-A	MH06-209	MH06-108A	180,399	0.18	4.0	0.72	178,257	0.18	4.0	0.71	196,459	0.20	4.0	0.79
6-B	MH06-122	MH06-108A	156,634	0.16	4.0	0.63	201,410	0.20	4.0	0.81	209,378	0.21	4.0	0.84
6-C	MH06-108A	PS 6	36,339	0.37	4.0	1.49	35,643	0.42	4.0	1.66	44,024	0.45	4.0	1.80
6-D	NA	PS 6	1,235,750	1.24	3.9	4.78	1,321,888	1.32	3.8	5.06	1,540,062	1.54	3.7	5.75
PS 6				1.61	3.7	5.97		1.74	3.7	6.37		1.99	3.6	7.14
	PS 6	MH07-129		1.61	3.7	5.97		1.74	3.7	6.37		1.99	3.6	7.14
7-N	MH07-129	PS 7	675,724	2.28	3.5	8.02	682,620	2.42	3.5	8.42	682,620	2.67	3.4	9.15
				4.43	3.2	14.01		5.99	3.0	26.56		7.07	2.9	37.45
	PS 7	WWTP		4.43	3.2	14.01		5.99	3.0	26.56		7.07	2.9	37.45
PS 7 - Alternative 2				4.43	3.2	14.01		5.99	4.4	26.56		7.07	5.3	37.45
	PS 7	WWTP		4.43	3.2	14.01		5.99	4.4	26.56		7.07	5.3	37.45

Table 8A-3

Peak Hourly Flows for PS18 and PS7 - Alternative 3 (Madison Design Curve)

Pumping Station Sub Basin	From	To	2010 U.F.				2030 U.F.				2060			
			Sub-Basin Flow (gpd)	Cumulative Flow (MGD)	Peak Factor	Cumulative Peak Flow (MGD)	Sub-Basin Flow (gpd)	Cumulative Flow (MGD)	Peak Factor	Cumulative Peak Flow (MGD)	Sub-Basin Flow (gpd)	Cumulative Flow (MGD)	Peak Factor	Cumulative Peak Flow (MGD)
14-A	MH 14-209	MH14-196	498,879	0.50	4.0	2.00	589,606	0.59	4.0	2.36	772,585	0.77	4.0	3.09
14-B	MH14-196	MH14-193	249,667	0.75	4.0	2.99	312,984	0.90	4.0	3.61	588,677	1.36	3.8	5.19
14-C	MH14-193	MH14-182	62,225	0.81	4.0	3.24	97,850	1.00	4.0	4.00	97,850	1.46	3.8	5.50
14-D	MH14-182	MH14-171	49,884	0.86	4.0	3.44	95,650	1.10	3.9	4.32	95,650	1.55	3.7	5.80
14-E	MH14-171	MH14-166	38,588	0.90	4.0	3.60	38,534	1.13	3.9	4.45	38,534	1.59	3.7	5.92
14-F	MH14-166	MH14-162	198,077	1.10	3.9	4.33	278,829	1.41	3.8	5.35	440,112	2.03	3.6	7.27
14-G	MH14-162	MH14-156	47,461	1.14	3.9	4.48	116,120	1.53	3.7	5.72	116,120	2.15	3.5	7.62
14-H	MH14-156	MH14-143	241,874	1.39	3.8	5.27	257,963	1.79	3.6	6.52	257,963	2.41	3.5	8.38
14-I	MH14-143	MH14-134	64,346	1.45	3.8	5.47	101,606	1.89	3.6	6.83	132,023	2.54	3.5	8.77
14-J	MH 14-416	MH14-134	308,576	0.31	4.0	1.23	519,368	0.52	4.0	2.08	624,919	0.62	4.0	2.50
14-K	MH14-134	MH14-102	53,627	1.81	3.6	6.60	66,727	2.48	3.5	8.58	66,727	3.23	3.3	10.74
14-Q	MH14-362	MH14-358	356,101	0.36	4.0	1.42	395,964	0.40	4.0	1.58	450,369	0.45	4.0	1.80
14-L	MH14-359	MH14-358	621,271	0.62	4.0	2.49	811,364	0.81	4.0	3.25	1,074,825	1.07	4.0	4.25
	MH14-358	MH14-356		0.98	4.0	3.91		1.21	3.9	4.69		1.53	3.7	5.71
14-M	MH14-356	MH14-323	429,812	1.41	3.8	5.33	747,196	1.95	3.6	7.03	1,094,496	2.62	3.4	9.00
14-N	MH14-323	MH14-315	153,514	1.56	3.7	5.82	204,977	2.16	3.5	7.65	261,387	2.88	3.4	9.75
14-O	MH14-315	MH14-102	194,823	1.76	3.7	6.42	214,995	2.37	3.5	8.28	305,002	3.19	3.3	10.61
	MH14-102	MH14-101		3.57	3.3	11.68		4.85	3.1	15.12		6.42	3.0	19.14
14-P	MH14-101	PS 14	400,678	3.97	3.2	12.77	409,746	5.26	3.1	16.18	409,746	6.83	3.0	20.16
PS 14				3.97	3.2	12.77		5.26	3.1	16.18		6.83	3.0	20.16
	PS 14	TE14-11057		3.97	3.2	12.77		5.26	3.1	16.18		6.83	3.0	20.16
13-F	TE14-11057	MH13-132	265,790	4.24	3.2	13.49	275,917	5.54	3.1	16.90	275,917	7.10	2.9	20.84
13-G	MH13-132	MH13-122A	122,964	4.36	3.2	13.82	160,919	5.70	3.0	17.31	160,919	7.26	2.9	21.24
13-A	MH13-122A	MH13-105A	351,739				367,673				367,673			
13-B	MH13-122A	MH13-105A	49,458				66,873				66,873			
13-C	MH13-122A	MH13-105A	639,164				730,012				730,012			
13-D	MH13-122A	MH13-105A	708,753				726,821				726,821			
13-E	MH13-122A	MH13-105A	188,234	6.30	3.0	18.83	196,939	7.78	2.9	22.52	196,939	9.35	2.8	26.28
13-H	MH13-105A	PS 13	468,068	6.76	3.0	20.00	1,353,883	9.14	2.8	25.77	1,353,883	10.71	2.8	29.44
PS 13				6.76	3.0	20.00		9.14	2.8	25.77		10.71	2.8	29.44
	PS 13	MH10-145		6.76	3.0	20.00		9.14	2.8	25.77		10.71	2.8	29.44
10-A	MH10-145	MH10-121	932,249	7.70	2.9	22.30	1,149,110	10.29	2.8	28.47	1,149,110	11.86	2.7	32.08
10-B	MH10-121	MH10-201	412,216	8.11	2.9	23.30	461,286	10.75	2.7	29.54	461,286	12.32	2.7	33.13
10-C	MH10-220	MH10-214	325,867	0.33	4.0	1.30	964,209	0.96	4.0	3.86	964,209	0.96	4.0	3.86
10-D	MH10-214	MH10-201	392,316	0.72	4.0	2.87	554,722	1.52	3.7	5.69	554,722	1.52	3.7	5.69
	MH10-201	MH10-115		8.83	2.8	25.03		12.27	2.7	33.02		13.84	2.6	36.54
10-E	MH10-115	MH10-104A	173,558	9.00	2.8	25.44	185,986	12.45	2.7	33.44	185,986	14.02	2.6	36.95
10-F	MH10-305	MH10-104A	188,221	0.19	4.0	0.75	190,971	0.19	4.0	0.76	190,971	0.19	4.0	0.76
	MH10-104A	MH10-102A		9.19	2.8	25.89		12.64	2.7	33.87		14.21	2.6	37.38
10-G	MH10-102A	MH10-101	11,479	9.20	2.8	25.91	17,319	12.66	2.7	33.91	17,319	14.23	2.6	37.42
10-H	MH10-101	PS 10	579,684	9.78	2.8	27.28	599,396	13.26	2.7	35.26	599,396	14.83	2.6	38.74
PS 10				9.78	2.8	27.28		13.26	2.7	35.26		14.83	2.6	38.74
	PS 10	MH07-955		9.78	2.8	27.28		13.26	2.7	35.26		14.83	2.6	38.74

Table 8A-3
Peak Hourly Flows for PS18 and PS7 - Alternative 3 (Madison Design Curve)

Pumping Station Sub Basin	From	To	2010 U.F. Sub-Basin Flow (gpd)	2010 U.F. Cumulative Flow (MGD)	2010 U.F. Peak Factor	2010 U.F. Cumulative Peak Flow (MGD)	2030 U.F. Sub-Basin Flow (gpd)	2030 U.F. Cumulative Flow (MGD)	2030 U.F. Peak Factor	2030 U.F. Cumulative Peak Flow (MGD)	2060 Sub-Basin Flow (gpd)	2060 Cumulative Flow (MGD)	2060 Peak Factor	2060 Cumulative Peak Flow (MGD)
7-A	MH07-955	MH07-932	871,342	10.65	2.8	29.32	980,335	14.24	2.6	37.44	980,335	15.81	2.6	40.88
7-C	MH07-734	MH07-426	693,680	0.69	4.0	2.77	1,989,624	1.99	3.6	7.14	3,771,162	3.77	3.2	12.23
7-B	MH07-437	MH07-426	550,457	0.55	4.0	2.20	1,018,340	1.02	4.0	4.06	1,566,335	1.57	3.7	5.84
7-D	MH07-426	MH07-415	157,183	1.40	3.8	5.31	357,817	3.37	3.3	11.11	357,817	5.70	3.0	17.31
7-E	MH07-415	MH07-932	85,825	1.49	3.8	5.59	116,336	3.48	3.3	11.44	116,336	5.81	3.0	17.60
7-F	MH07-932	MH07-215	213,427	12.35	2.7	33.21	226,623	17.95	2.5	45.50	226,623	21.85	2.5	53.68
				12.35	2.7	33.21		17.95	2.5	45.50		21.85	2.5	53.68
	PS 18	WWTP		12.35	2.7	33.21		17.95	2.5	45.50		21.85	2.5	53.68
PS 18 - Alternative 3				12.35	2.7	33.21		17.95	2.5	45.50		21.85	2.5	53.68
	PS 18	WWTP		12.35				17.95				21.85		
Excess flow to PS7						20.86				27.55				31.84
7-J	MH07-249	MH07-228	518,417	0.52	4.0	2.07	1,368,622	1.37	3.8	5.21	1,734,576	1.73	3.7	6.36
9-A	MH09-108	MH09-104	647,586	0.65	4.0	2.59	918,416	0.92	4.0	3.67	1,380,367	1.38	3.8	5.25
9-B	MH09-104	PS 9	317,105	0.96	4.0	3.86	364,702	1.28	3.8	4.93	364,702	1.75	3.7	6.39
PS 9				0.96	4.0	3.86		1.28	3.8	4.93		1.75	3.7	6.39
	PS 9	MH07-517		0.96	4.0	3.86		1.28	3.8	4.93		1.75	3.7	6.39
7-G	MH07-517	MH07-512	10,080	0.97	4.0	3.90	25,880	1.31	3.8	5.02	25,880	1.77	3.7	6.47
7-H	MH07-618	MH07-512	77,097	0.08	4.0	0.31	141,857	0.14	4.0	0.57	141,857	0.14	4.0	0.57
7-I	MH07-512	MH07-228	56,267	1.11	3.9	4.36	141,304	1.59	3.7	5.92	141,304	2.05	3.6	7.33
	MH07-228	MH07-224		1.63	3.7	6.02		2.96	3.4	9.98		3.79	3.2	12.28
7-K	MH07-224	MH07-218	121,062	1.75	3.7	6.40	156,277	3.12	3.3	10.42	156,277	3.94	3.2	12.70
7-L	MH07-823	MH07-218	94,512	0.09	4.0	0.38	104,614	0.10	4.0	0.42	104,614	0.10	4.0	0.42
	MH07-218	MH07-215		1.84	3.6	6.69		3.22	3.3	10.71		4.05	3.2	12.99
	MH07-215	MH07-211		1.84	3.6	27.55		3.22	3.3	38.26		4.05	3.2	44.82
7-M	MH07-211	PS 7	305,045	2.15	3.5	28.47	350,317	3.57	3.3	39.23	350,317	4.40	3.2	45.76
6-A	MH06-209	MH06-108A	180,399	0.18	4.0	0.72	178,257	0.18	4.0	0.71	196,459	0.20	4.0	0.79
6-B	MH06-122	MH06-108A	156,634	0.16	4.0	0.63	201,410	0.20	4.0	0.81	209,378	0.21	4.0	0.84
6-C	MH06-108A	PS 6	36,339	0.37	4.0	1.49	35,643	0.42	4.0	1.66	44,024	0.45	4.0	1.80
6-D	NA	PS 6	1,235,750	1.24	3.9	4.78	1,321,888	1.32	3.8	5.06	1,540,062	1.54	3.7	5.75
PS 6				1.61	3.7	5.97		1.74	3.7	6.37		1.99	3.6	7.14
	PS 6	MH07-129		1.61	3.7	5.97		1.74	3.7	6.37		1.99	3.6	7.14
7-N	MH07-129	PS 7	675,724	2.28	3.5	8.02	682,620	2.42	3.5	8.42	682,620	2.67	3.4	9.15
				4.43	3.2	14.01		5.99	3.0	18.06		7.07	2.9	20.77
	PS 7	WWTP		4.43	3.2	14.01		5.99	3.0	18.06		7.07	2.9	20.77
PS 7 - Alternative 3				4.43	3.2	34.87		5.99	3.0	45.61		7.07	2.9	52.60
	PS 7	WWTP		4.43	3.2	34.87		5.99	3.0	45.61		7.07	2.9	52.60

Table 9A-1

Peak Hourly Flows for PS18 and PS7 - Alternative 1 (Modified Madison Design Curve)

			2010 U.F.				2030 U.F.				2060			
Pumping Station Sub Basin	From	To	Sub-Basin Flow (gpd)	Cumulative Flow (MGD)	Peak Factor	Cumulative Peak Flow (MGD)	Sub-Basin Flow (gpd)	Cumulative Flow (MGD)	Peak Factor	Cumulative Peak Flow (MGD)	Sub-Basin Flow (gpd)	Cumulative Flow (MGD)	Peak Factor	Cumulative Peak Flow (MGD)
14-A	MH 14-209	MH14-196	498,879	0.50	4.0	2.00	589,606	0.59	4.0	2.36	772,585	0.77	4.0	3.09
14-B	MH14-196	MH14-193	249,667	0.75	4.0	2.99	312,984	0.90	4.0	3.61	588,677	1.36	3.8	5.19
14-C	MH14-193	MH14-182	62,225	0.81	4.0	3.24	97,850	1.00	4.0	4.00	97,850	1.46	3.8	5.50
14-D	MH14-182	MH14-171	49,884	0.86	4.0	3.44	95,650	1.10	3.9	4.32	95,650	1.55	3.7	5.80
14-E	MH14-171	MH14-166	38,588	0.90	4.0	3.60	38,534	1.13	3.9	4.45	38,534	1.59	3.7	5.92
14-F	MH14-166	MH14-162	198,077	1.10	3.9	4.33	278,829	1.41	3.8	5.35	440,112	2.03	3.6	7.27
14-G	MH14-162	MH14-156	47,461	1.14	3.9	4.48	116,120	1.53	3.7	5.72	116,120	2.15	3.5	7.62
14-H	MH14-156	MH14-143	241,874	1.39	3.8	5.27	257,963	1.79	3.6	6.52	257,963	2.41	3.5	8.38
14-I	MH14-143	MH14-134	64,346	1.45	3.8	5.47	101,606	1.89	3.6	6.83	132,023	2.54	3.5	8.77
14-J	MH 14-416	MH14-134	308,576	0.31	4.0	1.23	519,368	0.52	4.0	2.08	624,919	0.62	4.0	2.50
14-K	MH14-134	MH14-102	53,627	1.81	3.6	6.60	66,727	2.48	3.5	8.58	66,727	3.23	3.3	10.74
14-Q	MH14-362	MH14-358	356,101	0.36	4.0	1.42	395,964	0.40	4.0	1.58	450,369	0.45	4.0	1.80
14-L	MH14-359	MH14-358	621,271	0.62	4.0	2.49	811,364	0.81	4.0	3.25	1,074,825	1.07	4.0	4.25
	MH14-358	MH14-356		0.98	4.0	3.91		1.21	3.9	4.69		1.53	3.7	5.71
14-M	MH14-356	MH14-323	429,812	1.41	3.8	5.33	747,196	1.95	3.6	7.03	1,094,496	2.62	3.4	9.00
14-N	MH14-323	MH14-315	153,514	1.56	3.7	5.82	204,977	2.16	3.5	7.65	261,387	2.88	3.4	9.75
14-O	MH14-315	MH14-102	194,823	1.76	3.7	6.42	214,995	2.37	3.5	8.28	305,002	3.19	3.3	10.61
	MH14-102	MH14-101		3.57	3.3	11.68		4.85	3.1	15.12		6.42	3.0	19.14
14-P	MH14-101	PS 14	400,678	3.97	3.2	12.77	409,746	5.26	3.1	16.18	409,746	6.83	3.0	20.16
PS 14				3.97	3.2	12.77		5.26	3.1	16.18		6.83	3.0	20.16
	PS 14	TE14-11057		3.97	3.2	12.77		5.26	3.1	16.18		6.83	3.0	20.16
13-F	TE14-11057	MH13-132	265,790	4.24	3.2	13.49	275,917	5.54	3.1	16.90	275,917	7.10	3.0	21.31
13-G	MH13-132	MH13-122A	122,964	4.36	3.2	13.82	160,919	5.70	3.0	17.31	160,919	7.26	3.0	21.79
13-A	MH13-122A	MH13-105A	351,739				367,673				367,673			
13-B	MH13-122A	MH13-105A	49,458				66,873				66,873			
13-C	MH13-122A	MH13-105A	639,164				730,012				730,012			
13-D	MH13-122A	MH13-105A	708,753				726,821				726,821			
13-E	MH13-122A	MH13-105A	188,234	6.30	3.0	18.83	196,939	7.78	3.0	23.35	196,939	9.35	3.0	28.06
13-H	MH13-105A	PS 13	468,068	6.76	3.0	20.00	1,353,883	9.14	3.0	27.42	1,353,883	10.71	3.0	32.12
PS 13				6.76	3.0	20.00		9.14	3.0	27.42		10.71	3.0	32.12
	PS 13	MH10-145		6.76	3.0	20.00		9.14	3.0	27.42		10.71	3.0	32.12
10-A	MH10-145	MH10-121	932,249	7.70	3.0	23.09	1,149,110	10.29	3.0	30.86	1,149,110	11.86	3.0	35.57
10-B	MH10-121	MH10-201	412,216	8.11	3.0	24.32	461,286	10.75	3.0	32.25	461,286	12.32	3.0	36.95
10-C	MH10-220	MH10-214	325,867	0.33	4.0	1.30	964,209	0.96	4.0	3.86	964,209	0.96	4.0	3.86
10-D	MH10-214	MH10-201	392,316	0.72	4.0	2.87	554,722	1.52	3.7	5.69	554,722	1.52	3.7	5.69
	MH10-201	MH10-115		8.83	3.0	26.48		12.27	3.0	36.80		13.84	3.0	41.51
10-E	MH10-115	MH10-104A	173,558	9.00	3.0	27.00	185,986	12.45	3.0	37.36	185,986	14.02	3.0	42.06
10-F	MH10-305	MH10-104A	188,221	0.19	4.0	0.75	190,971	0.19	4.0	0.76	190,971	0.19	4.0	0.76
	MH10-104A	MH10-102A		9.19	3.0	27.56		12.64	3.0	37.93		14.21	3.0	42.64
10-G	MH10-102A	MH10-101	11,479	9.20	3.0	27.60	17,319	12.66	3.0	37.99	17,319	14.23	3.0	42.69
10-H	MH10-101	PS 10	579,684	9.78	3.0	29.34	599,396	13.26	3.0	39.78	599,396	14.83	3.0	44.49
PS 10				9.78	3.0	3.4		13.26	3.0	39.78		14.83	3.0	44.49
	PS 10	MH07-955		9.78	3.0	29.34		13.26	3.0	39.78		14.83	3.0	44.49

Table 9A-1

Peak Hourly Flows for PS18 and PS7 - Alternative 1 (Modified Madison Design Curve)

			2010 U.F.				2030 U.F.				2060			
Pumping Station Sub Basin	From	To	Sub-Basin Flow (gpd)	Cumulative Flow (MGD)	Peak Factor	Cumulative Peak Flow (MGD)	Sub-Basin Flow (gpd)	Cumulative Flow (MGD)	Peak Factor	Cumulative Peak Flow (MGD)	Sub-Basin Flow (gpd)	Cumulative Flow (MGD)	Peak Factor	Cumulative Peak Flow (MGD)
7-A	MH07-955	MH07-932	871,342	10.65	3.0	31.95	980,335	14.24	3.0	42.73	980,335	15.81	3.0	47.43
7-C	MH07-734	MH07-426	693,680	0.69	4.0	2.77	1,989,624	1.99	3.6	7.14	3,771,162	3.77	3.2	12.23
7-B	MH07-437	MH07-426	550,457	0.55	4.0	2.20	1,018,340	1.02	4.0	4.06	1,566,335	1.57	3.7	5.84
7-D	MH07-426	MH07-415	157,183	1.40	3.8	5.31	357,817	3.37	3.3	11.11	357,817	5.70	3.0	17.31
7-E	MH07-415	MH07-932	85,825	1.49	3.8	5.59	116,336	3.48	3.3	11.44	116,336	5.81	3.0	17.60
7-F	MH07-932	MH07-215	213,427	12.35	3.0	37.05	226,623	17.95	3.0	53.85	226,623	21.85	3.0	65.54
PS 18 - Alternative 1				12.35	3.0	37.05		17.95	3.0	53.85		21.85	3.0	65.54
	PS 18	WWTP		12.35	3.0	37.05		17.95	3.0	53.85		21.85	3.0	65.54
7-J	MH07-249	MH07-228	518,417	0.52	4.0	2.07	1,368,622	1.37	3.8	5.21	1,734,576	1.73	3.7	6.36
9-A	MH09-108	MH09-104	647,586	0.65	4.0	2.59	918,416	0.92	4.0	3.67	1,380,367	1.38	3.8	5.25
9-B	MH09-104	PS 9	317,105	0.96	4.0	3.86	364,702	1.28	3.8	4.93	364,702	1.75	3.7	6.39
PS 9				0.96	4.0	3.86		1.28	3.8	4.93		1.75	3.7	6.39
	PS 9	MH07-517		0.96	4.0	3.86		1.28	3.8	4.93		1.75	3.7	6.39
7-G	MH07-517	MH07-512	10,080	0.97	4.0	3.90	25,880	1.31	3.8	5.02	25,880	1.77	3.7	6.47
7-H	MH07-618	MH07-512	77,097	0.08	4.0	0.31	141,857	0.14	4.0	0.57	141,857	0.14	4.0	0.57
7-I	MH07-512	MH07-228	56,267	1.11	3.9	4.36	141,304	1.59	3.7	5.92	141,304	2.05	3.6	7.33
	MH07-228	MH07-224		1.63	3.7	6.02		2.96	3.4	9.98		3.79	3.2	12.28
7-K	MH07-224	MH07-218	121,062	1.75	3.7	6.40	156,277	3.12	3.3	10.42	156,277	3.94	3.2	12.70
7-L	MH07-823	MH07-218	94,512	0.09	4.0	0.38	104,614	0.10	4.0	0.42	104,614	0.10	4.0	0.42
	MH07-218	MH07-215		1.84	3.6	6.69		3.22	3.3	10.71		4.05	3.2	12.99
	MH07-215	MH07-211		1.84	3.6	6.69		3.22	3.3	10.71		4.05	3.2	12.99
7-M	MH07-211	PS 7	305,045	2.15	3.5	7.61	350,317	3.57	3.3	11.68	350,317	4.40	3.2	13.93
6-A	MH06-209	MH06-108A	180,399	0.18	4.0	0.72	178,257	0.18	4.0	0.71	196,459	0.20	4.0	0.79
6-B	MH06-122	MH06-108A	156,634	0.16	4.0	0.63	201,410	0.20	4.0	0.81	209,378	0.21	4.0	0.84
6-C	MH06-108A	PS 6	36,339	0.37	4.0	1.49	35,643	0.42	4.0	1.66	44,024	0.45	4.0	1.80
6-D	NA	PS 6	1,235,750	1.24	3.9	4.78	1,321,888	1.32	3.8	5.06	1,540,062	1.54	3.7	5.75
PS 6				1.61	3.7	5.97		1.74	3.7	6.37		1.99	3.6	7.14
	PS 6	MH07-129		1.61	3.7	5.97		1.74	3.7	6.37		1.99	3.6	7.14
7-N	MH07-129	PS 7	675,724	2.28	3.5	8.02	682,620	2.42	3.5	8.42	682,620	2.67	3.4	9.15
PS 7 - Alternative 1				4.43	3.2	14.01		5.99	3.0	18.06		7.07	3.0	21.22
	PS 7	WWTP		4.43	3.2	14.01		5.99	3.0	18.06		7.07	3.0	21.22

Table 9A-2

Peak Hourly Flows for PS18 and PS7 - Alternative 2 (Modified Madison Design Curve)

Pumping Station Sub Basin	From	To	2010 U.F. Sub-Basin Flow (gpd)	2010 U.F. Cumulative Flow (MGD)	2010 U.F. Peak Factor	2010 U.F. Cumulative Peak Flow (MGD)	2030 U.F. Sub-Basin Flow (gpd)	2030 U.F. Cumulative Flow (MGD)	2030 U.F. Peak Factor	2030 U.F. Cumulative Peak Flow (MGD)	2060 Sub-Basin Flow (gpd)	2060 Cumulative Flow (MGD)	2060 Peak Factor	2060 Cumulative Peak Flow (MGD)
14-A	MH 14-209	MH14-196	498,879	0.50	4.0	2.00	589,606	0.59	4.0	2.36	772,585	0.77	4.0	3.09
14-B	MH14-196	MH14-193	249,667	0.75	4.0	2.99	312,984	0.90	4.0	3.61	588,677	1.36	3.8	5.19
14-C	MH14-193	MH14-182	62,225	0.81	4.0	3.24	97,850	1.00	4.0	4.00	97,850	1.46	3.8	5.50
14-D	MH14-182	MH14-171	49,884	0.86	4.0	3.44	95,650	1.10	3.9	4.32	95,650	1.55	3.7	5.80
14-E	MH14-171	MH14-166	38,588	0.90	4.0	3.60	38,534	1.13	3.9	4.45	38,534	1.59	3.7	5.92
14-F	MH14-166	MH14-162	198,077	1.10	3.9	4.33	278,829	1.41	3.8	5.35	440,112	2.03	3.6	7.27
14-G	MH14-162	MH14-156	47,461	1.14	3.9	4.48	116,120	1.53	3.7	5.72	116,120	2.15	3.5	7.62
14-H	MH14-156	MH14-143	241,874	1.39	3.8	5.27	257,963	1.79	3.6	6.52	257,963	2.41	3.5	8.38
14-I	MH14-143	MH14-134	64,346	1.45	3.8	5.47	101,606	1.89	3.6	6.83	132,023	2.54	3.5	8.77
14-J	MH 14-416	MH14-134	308,576	0.31	4.0	1.23	519,368	0.52	4.0	2.08	624,919	0.62	4.0	2.50
14-K	MH14-134	MH14-102	53,627	1.81	3.6	6.60	66,727	2.48	3.5	8.58	66,727	3.23	3.3	10.74
14-Q	MH14-362	MH14-358	356,101	0.36	4.0	1.42	395,964	0.40	4.0	1.58	450,369	0.45	4.0	1.80
14-L	MH14-359	MH14-358	621,271	0.62	4.0	2.49	811,364	0.81	4.0	3.25	1,074,825	1.07	4.0	4.25
	MH14-358	MH14-356		0.98	4.0	3.91		1.21	3.9	4.69		1.53	3.7	5.71
14-M	MH14-356	MH14-323	429,812	1.41	3.8	5.33	747,196	1.95	3.6	7.03	1,094,496	2.62	3.4	9.00
14-N	MH14-323	MH14-315	153,514	1.56	3.7	5.82	204,977	2.16	3.5	7.65	261,387	2.88	3.4	9.75
14-O	MH14-315	MH14-102	194,823	1.76	3.7	6.42	214,995	2.37	3.5	8.28	305,002	3.19	3.3	10.61
	MH14-102	MH14-101		3.57	3.3	11.68		4.85	3.1	15.12		6.42	3.0	19.14
14-P	MH14-101	PS 14	400,678	3.97	3.2	12.77	409,746	5.26	3.1	16.18	409,746	6.83	3.0	20.16
PS 14				3.97	3.2	12.77		5.26	3.1	16.18		6.83	3.0	20.16
	PS 14	TE14-11057		3.97	3.2	12.77		5.26	3.1	16.18		6.83	3.0	20.16
13-F	TE14-11057	MH13-132	265,790	4.24	3.2	13.49	275,917	5.54	3.1	16.90	275,917	7.10	3.0	21.31
13-G	MH13-132	MH13-122A	122,964	4.36	3.2	13.82	160,919	5.70	3.0	17.31	160,919	7.26	3.0	21.79
13-A	MH13-122A	MH13-105A	351,739				367,673				367,673			
13-B	MH13-122A	MH13-105A	49,458				66,873				66,873			
13-C	MH13-122A	MH13-105A	639,164				730,012				730,012			
13-D	MH13-122A	MH13-105A	708,753				726,821				726,821			
13-E	MH13-122A	MH13-105A	188,234	6.30	3.0	18.83	196,939	7.78	3.0	23.35	196,939	9.35	3.0	28.06
13-H	MH13-105A	PS 13	468,068	6.76	3.0	20.00	1,353,883	9.14	3.0	27.42	1,353,883	10.71	3.0	32.12
PS 13				6.76	3.0	20.00		9.14	3.0	27.42		10.71	3.0	32.12
	PS 13	MH10-145		6.76	3.0	20.00		9.14	3.0	27.42		10.71	3.0	32.12
10-A	MH10-145	MH10-121	932,249	7.70	3.0	23.09	1,149,110	10.29	3.0	30.86	1,149,110	11.86	3.0	35.57
10-B	MH10-121	MH10-201	412,216	8.11	3.0	24.32	461,286	10.75	3.0	32.25	461,286	12.32	3.0	36.95
10-C	MH10-220	MH10-214	325,867	0.33	4.0	1.30	964,209	0.96	4.0	3.86	964,209	0.96	4.0	3.86
10-D	MH10-214	MH10-201	392,316	0.72	4.0	2.87	554,722	1.52	3.7	5.69	554,722	1.52	3.7	5.69
	MH10-201	MH10-115		8.83	3.0	26.48		12.27	3.0	36.80		13.84	3.0	41.51
10-E	MH10-115	MH10-104A	173,558	9.00	3.0	27.00	185,986	12.45	3.0	37.36	185,986	14.02	3.0	42.06
10-F	MH10-305	MH10-104A	188,221	0.19	4.0	0.75	190,971	0.19	4.0	0.76	190,971	0.19	4.0	0.76
	MH10-104A	MH10-102A		9.19	3.0	27.56		12.64	3.0	37.93		14.21	3.0	42.64
10-G	MH10-102A	MH10-101	11,479	9.20	3.0	27.60	17,319	12.66	3.0	37.99	17,319	14.23	3.0	42.69
10-H	MH10-101	PS 10	579,684	9.78	3.0	29.34	599,396	13.26	3.0	39.78	599,396	14.83	3.0	44.49
PS 10				9.78	3.0	29.34		13.26	3.0	39.78		14.83	3.0	44.49
	PS 10	MH07-955		9.78	3.0	29.34		13.26	3.0	39.78		14.83	3.0	44.49

Table 9A-2
Peak Hourly Flows for PS18 and PS7 - Alternative 2 (Modified Madison Design Curve)

Pumping Station Sub Basin	From	To	2010 U.F.				2030 U.F.				2060			
			Sub-Basin Flow (gpd)	Cumulative Flow (MGD)	Peak Factor	Cumulative Peak Flow (MGD)	Sub-Basin Flow (gpd)	Cumulative Flow (MGD)	Peak Factor	Cumulative Peak Flow (MGD)	Sub-Basin Flow (gpd)	Cumulative Flow (MGD)	Peak Factor	Cumulative Peak Flow (MGD)
7-A	MH07-955	MH07-932	871,342	10.65	3.0	31.95	980,335	14.24	3.0	42.73	980,335	15.81	3.0	47.43
7-C	MH07-734	MH07-426	693,680	0.69	4.0	2.77	1,989,624	1.99	3.6	7.14	3,771,162	3.77	3.2	12.23
7-B	MH07-437	MH07-426	550,457	0.55	4.0	2.20	1,018,340	1.02	4.0	4.06	1,566,335	1.57	3.7	5.84
7-D	MH07-426	MH07-415	157,183	1.40	3.8	5.31	357,817	3.37	3.3	11.11	357,817	5.70	3.0	17.31
7-E	MH07-415	MH07-932	85,825	1.49	3.8	5.59	116,336	3.48	3.3	11.44	116,336	5.81	3.0	17.60
7-F	MH07-932	MH07-215	213,427	12.35	3.0	37.05	226,623	17.95	3.0	53.85	226,623	21.85	3.0	65.54
				12.35	3.0	37.05		17.95	3.0	53.85		21.85	3.0	65.54
	PS 18	WWTP		12.35	3.0	37.05		17.95	3.0	53.85		21.85	3.0	65.54
PS 18 - Alternative 2				12.35	3.0	37.05		17.95	2.5	44.00		21.85	2.0	44.00
	PS 18	WWTP		12.35	3.0	37.05		17.95	2.5	44.00		21.85	2.0	44.00
Excess peak flow to PS7						0.00				9.85				21.54
7-J	MH07-249	MH07-228	518,417	0.52	4.0	2.07	1,368,622	1.37	3.8	5.21	1,734,576	1.73	3.7	6.36
9-A	MH09-108	MH09-104	647,586	0.65	4.0	2.59	918,416	0.92	4.0	3.67	1,380,367	1.38	3.8	5.25
9-B	MH09-104	PS 9	317,105	0.96	4.0	3.86	364,702	1.28	3.8	4.93	364,702	1.75	3.7	6.39
PS 9				0.96	4.0	3.86		1.28	3.8	4.93		1.75	3.7	6.39
	PS 9	MH07-517		0.96	4.0	3.86		1.28	3.8	4.93		1.75	3.7	6.39
7-G	MH07-517	MH07-512	10,080	0.97	4.0	3.90	25,880	1.31	3.8	5.02	25,880	1.77	3.7	6.47
7-H	MH07-618	MH07-512	77,097	0.08	4.0	0.31	141,857	0.14	4.0	0.57	141,857	0.14	4.0	0.57
7-I	MH07-512	MH07-228	56,267	1.11	3.9	4.36	141,304	1.59	3.7	5.92	141,304	2.05	3.6	7.33
	MH07-228	MH07-224		1.63	3.7	6.02		2.96	3.4	9.98		3.79	3.2	12.28
7-K	MH07-224	MH07-218	121,062	1.75	3.7	6.40	156,277	3.12	3.3	10.42	156,277	3.94	3.2	12.70
7-L	MH07-823	MH07-218	94,512	0.09	4.0	0.38	104,614	0.10	4.0	0.42	104,614	0.10	4.0	0.42
	MH07-218	MH07-215		1.84	3.6	6.69		3.22	3.3	10.71		4.05	3.2	12.99
	MH07-215	MH07-211		1.84	3.6	6.69		3.22	3.3	20.56		4.05	3.2	34.53
7-M	MH07-211	PS 7	305,045	2.15	3.5	7.61	350,317	3.57	3.3	21.54	350,317	4.40	3.2	35.47
6-A	MH06-209	MH06-108A	180,399	0.18	4.0	0.72	178,257	0.18	4.0	0.71	196,459	0.20	4.0	0.79
6-B	MH06-122	MH06-108A	156,634	0.16	4.0	0.63	201,410	0.20	4.0	0.81	209,378	0.21	4.0	0.84
6-C	MH06-108A	PS 6	36,339	0.37	4.0	1.49	35,643	0.42	4.0	1.66	44,024	0.45	4.0	1.80
6-D	NA	PS 6	1,235,750	1.24	3.9	4.78	1,321,888	1.32	3.8	5.06	1,540,062	1.54	3.7	5.75
PS 6				1.61	3.7	5.97		1.74	3.7	6.37		1.99	3.6	7.14
	PS 6	MH07-129		1.61	3.7	5.97		1.74	3.7	6.37		1.99	3.6	7.14
7-N	MH07-129	PS 7	675,724	2.28	3.5	8.02	682,620	2.42	3.5	8.42	682,620	2.67	3.4	9.15
				4.43	3.2	14.01		5.99	3.0	27.91		7.07	3.0	42.76
	PS 7	WWTP		4.43	3.2	14.01		5.99	3.0	27.91		7.07	3.0	42.76
PS 7 - Alternative 2				4.43	3.2	14.01		5.99	4.7	27.91		7.07	6.0	42.76
	PS 7	WWTP		4.43	3.2	14.01		5.99	4.7	27.91		7.07	6.0	42.76

Table 9A-3

Peak Hourly Flows for PS18 and PS7 - Alternative 3 **(Modified Madison Design Curve)**

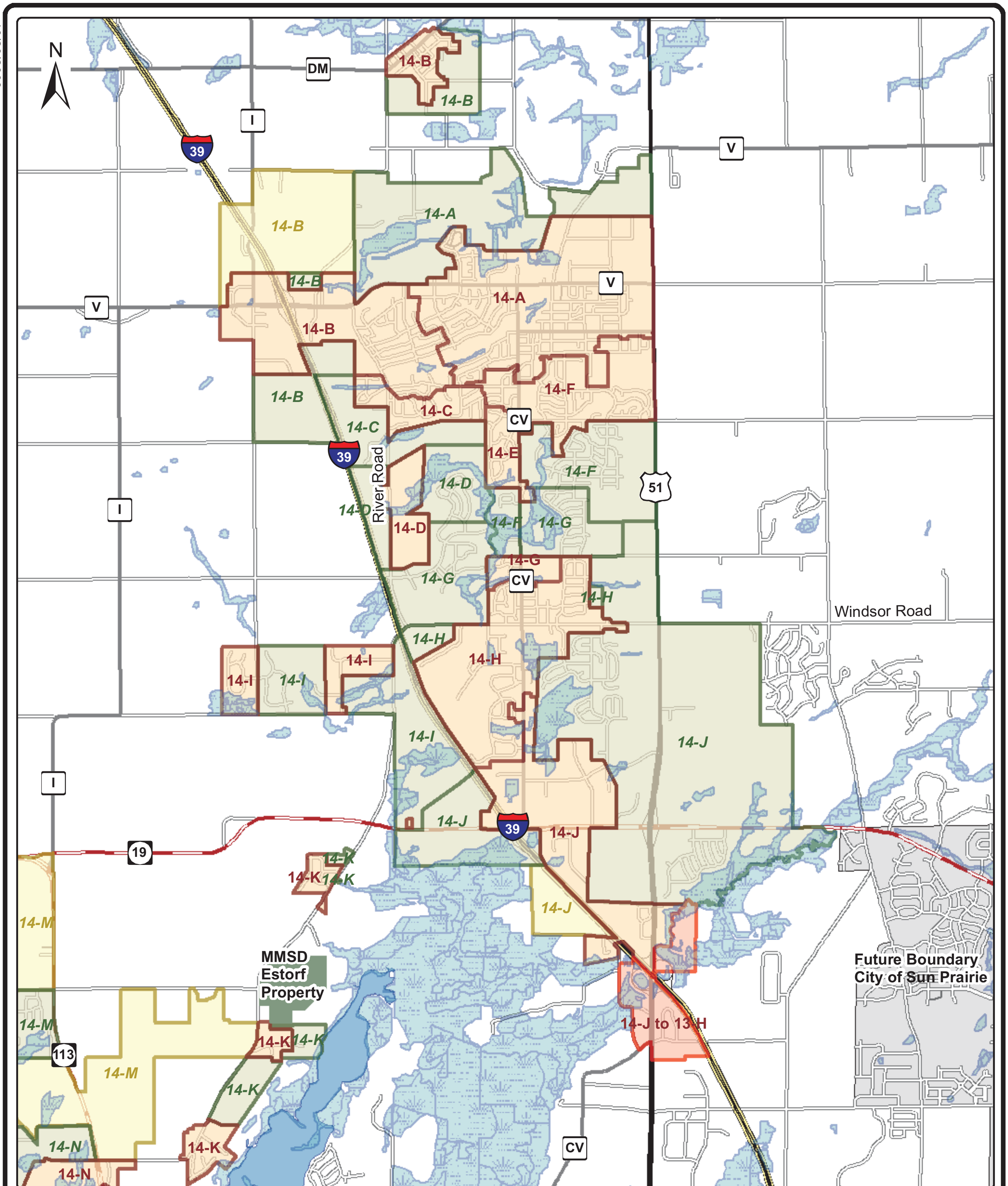
			2010 U.F.				2030 U.F.				2060			
Pumping Station Sub Basin	From	To	Sub-Basin Flow (gpd)	Cumulative Flow (MGD)	Peak Factor	Cumulative Peak Flow (MGD)	Sub-Basin Flow (gpd)	Cumulative Flow (MGD)	Peak Factor	Cumulative Peak Flow (MGD)	Sub-Basin Flow (gpd)	Cumulative Flow (MGD)	Peak Factor	Cumulative Peak Flow (MGD)
14-A	MH 14-209	MH14-196	498,879	0.50	4.0	2.00	589,606	0.59	4.0	2.36	772,585	0.77	4.0	3.09
14-B	MH14-196	MH14-193	249,667	0.75	4.0	2.99	312,984	0.90	4.0	3.61	588,677	1.36	3.8	5.19
14-C	MH14-193	MH14-182	62,225	0.81	4.0	3.24	97,850	1.00	4.0	4.00	97,850	1.46	3.8	5.50
14-D	MH14-182	MH14-171	49,884	0.86	4.0	3.44	95,650	1.10	3.9	4.32	95,650	1.55	3.7	5.80
14-E	MH14-171	MH14-166	38,588	0.90	4.0	3.60	38,534	1.13	3.9	4.45	38,534	1.59	3.7	5.92
14-F	MH14-166	MH14-162	198,077	1.10	3.9	4.33	278,829	1.41	3.8	5.35	440,112	2.03	3.6	7.27
14-G	MH14-162	MH14-156	47,461	1.14	3.9	4.48	116,120	1.53	3.7	5.72	116,120	2.15	3.5	7.62
14-H	MH14-156	MH14-143	241,874	1.39	3.8	5.27	257,963	1.79	3.6	6.52	257,963	2.41	3.5	8.38
14-I	MH14-143	MH14-134	64,346	1.45	3.8	5.47	101,606	1.89	3.6	6.83	132,023	2.54	3.5	8.77
14-J	MH 14-416	MH14-134	308,576	0.31	4.0	1.23	519,368	0.52	4.0	2.08	624,919	0.62	4.0	2.50
14-K	MH14-134	MH14-102	53,627	1.81	3.6	6.60	66,727	2.48	3.5	8.58	66,727	3.23	3.3	10.74
14-Q	MH14-362	MH14-358	356,101	0.36	4.0	1.42	395,964	0.40	4.0	1.58	450,369	0.45	4.0	1.80
14-L	MH14-359	MH14-358	621,271	0.62	4.0	2.49	811,364	0.81	4.0	3.25	1,074,825	1.07	4.0	4.25
	MH14-358	MH14-356		0.98	4.0	3.91		1.21	3.9	4.69		1.53	3.7	5.71
14-M	MH14-356	MH14-323	429,812	1.41	3.8	5.33	747,196	1.95	3.6	7.03	1,094,496	2.62	3.4	9.00
14-N	MH14-323	MH14-315	153,514	1.56	3.7	5.82	204,977	2.16	3.5	7.65	261,387	2.88	3.4	9.75
14-O	MH14-315	MH14-102	194,823	1.76	3.7	6.42	214,995	2.37	3.5	8.28	305,002	3.19	3.3	10.61
	MH14-102	MH14-101		3.57	3.3	11.68		4.85	3.1	15.12		6.42	3.0	19.14
14-P	MH14-101	PS 14	400,678	3.97	3.2	12.77	409,746	5.26	3.1	16.18	409,746	6.83	3.0	20.16
PS 14				3.97	3.2	12.77		5.26	3.1	16.18		6.83	3.0	20.16
	PS 14	TE14-11057		3.97	3.2	12.77		5.26	3.1	16.18		6.83	3.0	20.16
13-F	TE14-11057	MH13-132	265,790	4.24	3.2	13.49	275,917	5.54	3.1	16.90	275,917	7.10	3.0	21.31
13-G	MH13-132	MH13-122A	122,964	4.36	3.2	13.82	160,919	5.70	3.0	17.31	160,919	7.26	3.0	21.79
13-A	MH13-122A	MH13-105A	351,739				367,673				367,673			
13-B	MH13-122A	MH13-105A	49,458				66,873				66,873			
13-C	MH13-122A	MH13-105A	639,164				730,012				730,012			
13-D	MH13-122A	MH13-105A	708,753				726,821				726,821			
13-E	MH13-122A	MH13-105A	188,234	6.30	3.0	18.83	196,939	7.78	3.0	23.35	196,939	9.35	3.0	28.06
13-H	MH13-105A	PS 13	468,068	6.76	3.0	20.00	1,353,883	9.14	3.0	27.42	1,353,883	10.71	3.0	32.12
PS 13				6.76	3.0	20.00		9.14	3.0	27.42		10.71	3.0	32.12
	PS 13	MH10-145		6.76	3.0	20.00		9.14	3.0	27.42		10.71	3.0	32.12
10-A	MH10-145	MH10-121	932,249	7.70	3.0	23.09	1,149,110	10.29	3.0	30.86	1,149,110	11.86	3.0	35.57
10-B	MH10-121	MH10-201	412,216	8.11	3.0	24.32	461,286	10.75	3.0	32.25	461,286	12.32	3.0	36.95
10-C	MH10-220	MH10-214	325,867	0.33	4.0	1.30	964,209	0.96	4.0	3.2	964,209	0.96	4.0	3.86
10-D	MH10-214	MH10-201	392,316	0.72	4.0	2.87	554,722	1.52	3.7	5.69	554,722	1.52	3.7	5.69
	MH10-201	MH10-115		8.83	3.0	26.48		12.27	3.0	36.80		13.84	3.0	41.51
10-E	MH10-115	MH10-104A	173,558	9.00	3.0	27.00	185,986	12.45	3.0	37.36	185,986	14.02	3.0	42.06
10-F	MH10-305	MH10-104A	188,221	0.19	4.0	0.75	190,971	0.19	4.0	0.76	190,971	0.19	4.0	0.76
	MH10-104A	MH10-102A		9.19	3.0	27.56		12.64	3.0	37.93		14.21	3.0	42.64
10-G	MH10-102A	MH10-101	11,479	9.20	3.0	27.60	17,319	12.66	3.0	37.99	17,319	14.23	3.0	42.69
10-H	MH10-101	PS 10	579,684	9.78	3.0	29.34	599,396	13.26	3.0	39.78	599,396	14.83	3.0	44.49
PS 10				9.78	3.0	29.34		13.26	3.0	39.78		14.83	3.0	44.49
	PS 10	MH07-955		9.78	3.0	29.34		13.26	3.0	39.78		14.83	3.0	44.49

Table 9A-3

Peak Hourly Flows for PS18 and PS7 - Alternative 3 **(Modified Madison Design Curve)**

Pumping Station Sub Basin	From	To	2010 U.F. Sub-Basin Flow (gpd)	2010 U.F. Cumulative Flow (MGD)	2010 U.F. Peak Factor	2010 U.F. Cumulative Peak Flow (MGD)	2030 U.F. Sub-Basin Flow (gpd)	2030 U.F. Cumulative Flow (MGD)	2030 U.F. Peak Factor	2030 U.F. Cumulative Peak Flow (MGD)	2060 Sub-Basin Flow (gpd)	2060 Cumulative Flow (MGD)	2060 Peak Factor	2060 Cumulative Peak Flow (MGD)
7-A	MH07-955	MH07-932	871,342	10.65	3.0	31.95	980,335	14.24	3.0	42.73	980,335	15.81	3.0	47.43
7-C	MH07-734	MH07-426	693,680	0.69	4.0	2.77	1,989,624	1.99	3.6	7.14	3,771,162	3.77	3.2	12.23
7-B	MH07-437	MH07-426	550,457	0.55	4.0	2.20	1,018,340	1.02	4.0	4.06	1,566,335	1.57	3.7	5.84
7-D	MH07-426	MH07-415	157,183	1.40	3.8	5.31	357,817	3.37	3.3	11.11	357,817	5.70	3.0	17.31
7-E	MH07-415	MH07-932	85,825	1.49	3.8	5.59	116,336	3.48	3.3	11.44	116,336	5.81	3.0	17.60
7-F	MH07-932	MH07-215	213,427	12.35	3.0	37.05	226,623	17.95	3.0	53.85	226,623	21.85	3.0	65.54
				12.35	3.0	37.05		17.95	3.0	53.85		21.85	3.0	65.54
	PS 18	WWTP		12.35	3.0	37.05		17.95	3.0	53.85		21.85	3.0	65.54
PS 18 - Alternative 3				12.35	3.0	37.05		17.95	3.0	53.85		21.85	3.0	65.54
	PS18	WWTP		12.35	3.0	37.05		17.95	3.0	53.85		21.85	3.0	65.54
Excess flow to PS7						24.70				35.90				43.70
7-J	MH07-249	MH07-228	518,417	0.52	4.0	2.07	1,368,622	1.37	3.8	5.21	1,734,576	1.73	3.7	6.36
9-A	MH09-108	MH09-104	647,586	0.65	4.0	2.59	918,416	0.92	4.0	3.67	1,380,367	1.38	3.8	5.25
9-B	MH09-104	PS 9	317,105	0.96	4.0	3.86	364,702	1.28	3.8	4.93	364,702	1.75	3.7	6.39
PS 9				0.96	4.0	3.86		1.28	3.8	4.93		1.75	3.7	6.39
	PS 9	MH07-517		0.96	4.0	3.86		1.28	3.8	4.93		1.75	3.7	6.39
7-G	MH07-517	MH07-512	10,080	0.97	4.0	3.90	25,880	1.31	3.8	5.02	25,880	1.77	3.7	6.47
7-H	MH07-618	MH07-512	77,097	0.08	4.0	0.31	141,857	0.14	4.0	0.57	141,857	0.14	4.0	0.57
7-I	MH07-512	MH07-228	56,267	1.11	3.9	4.36	141,304	1.59	3.7	5.92	141,304	2.05	3.6	7.33
	MH07-228	MH07-224		1.63	3.7	6.02		2.96	3.4	9.98		3.79	3.2	12.28
7-K	MH07-224	MH07-218	121,062	1.75	3.7	6.40	156,277	3.12	3.3	10.42	156,277	3.94	3.2	12.70
7-L	MH07-823	MH07-218	94,512	0.09	4.0	0.38	104,614	0.10	4.0	0.42	104,614	0.10	4.0	0.42
	MH07-218	MH07-215		1.84	3.6	6.69		3.22	3.3	10.71		4.05	3.2	12.99
	MH07-215	MH07-211		1.84	3.6	31.39		3.22	3.3	46.61		4.05	3.2	56.68
7-M	MH07-211	PS 7	305,045	2.15	3.5	32.31	350,317	3.57	3.3	47.59	350,317	4.40	3.2	57.62
6-A	MH06-209	MH06-108A	180,399	0.18	4.0	0.72	178,257	0.18	4.0	0.71	196,459	0.20	4.0	0.79
6-B	MH06-122	MH06-108A	156,634	0.16	4.0	0.63	201,410	0.20	4.0	0.81	209,378	0.21	4.0	0.84
6-C	MH06-108A	PS 6	36,339	0.37	4.0	1.49	35,643	0.42	4.0	1.66	44,024	0.45	4.0	1.80
6-D	NA	PS 6	1,235,750	1.24	3.9	4.78	1,321,888	1.32	3.8	5.06	1,540,062	1.54	3.7	5.75
PS 6				1.61	3.7	5.97		1.74	3.7	6.37		1.99	3.6	7.14
	PS 6	MH07-129		1.61	3.7	5.97		1.74	3.7	6.37		1.99	3.6	7.14
7-N	MH07-129	PS 7	675,724	2.28	3.5	8.02	682,620	2.42	3.5	8.42	682,620	2.67	3.4	9.15
				4.43	3.2	14.01		5.99	3.0	18.06		7.07	3.0	21.22
	PS 7	WWTP		4.43	3.2	14.01		5.99	3.0	18.06		7.07	3.0	21.22
PS 7 - Alternative 3				4.43	3.2	38.71		5.99	3.0	53.96		7.07	3.0	64.91
	PS7	WWTP		4.43	3.2	38.71		5.99	3.0	53.96		7.07	3.0	64.91

10/23/2008



Sub-Basins in 2000
Removed by 2030
Potential by 2030
Potential by 2060
DNR Wetlands

PUMPING STATION 14 SUB-BASINS A-K

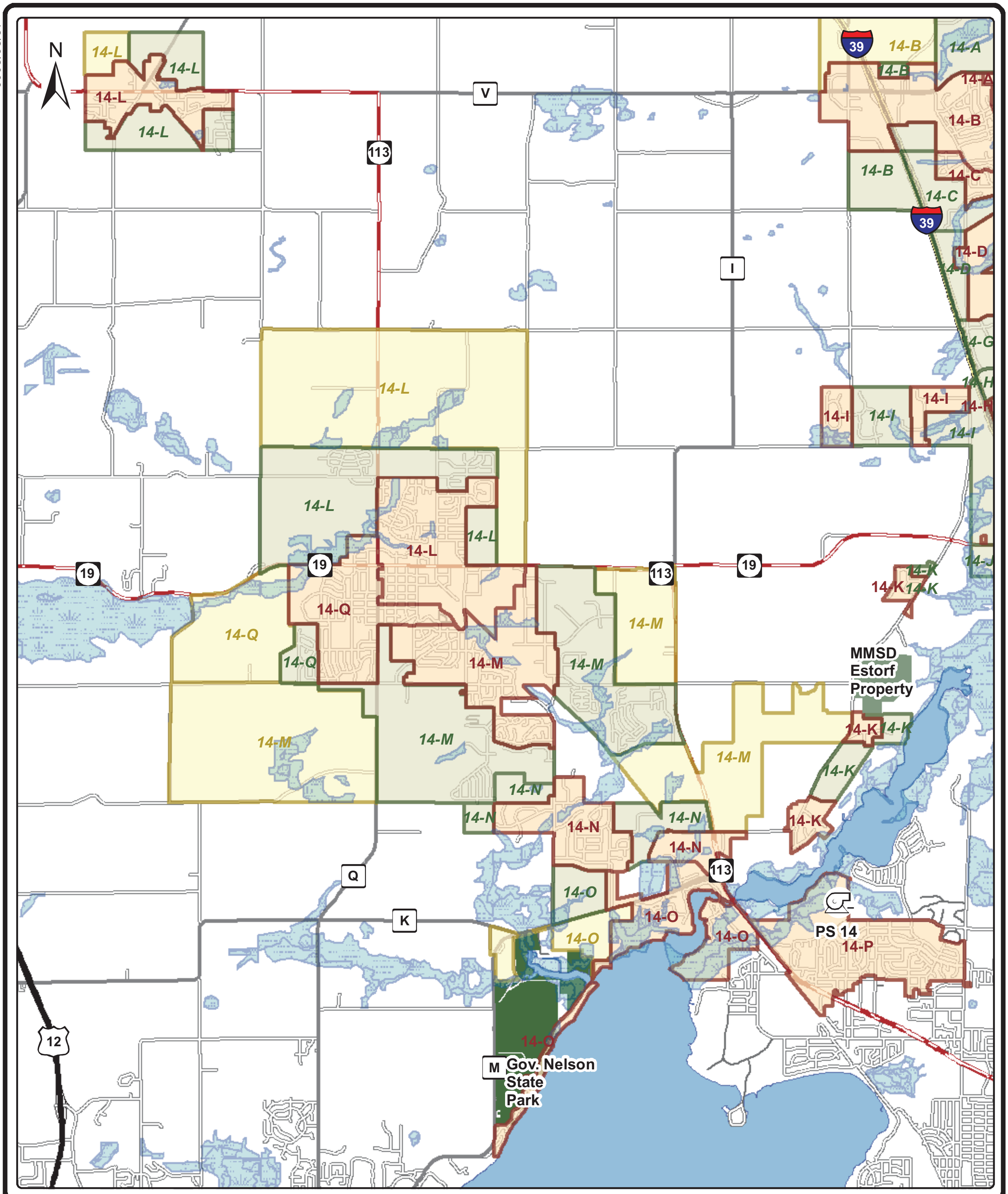
MADISON METROPOLITAN SEWERAGE DISTRICT
COLLECTION SYSTEM EVALUATION
2008

0 0.5 1 2 Miles

MMSD

Figure 14

12/26/2008



Sub-Basin 2000

Potential by 2030

Potential by 2060

DNR Wetlands

PUMPING STATION 14 SUB-BASINS L-Q

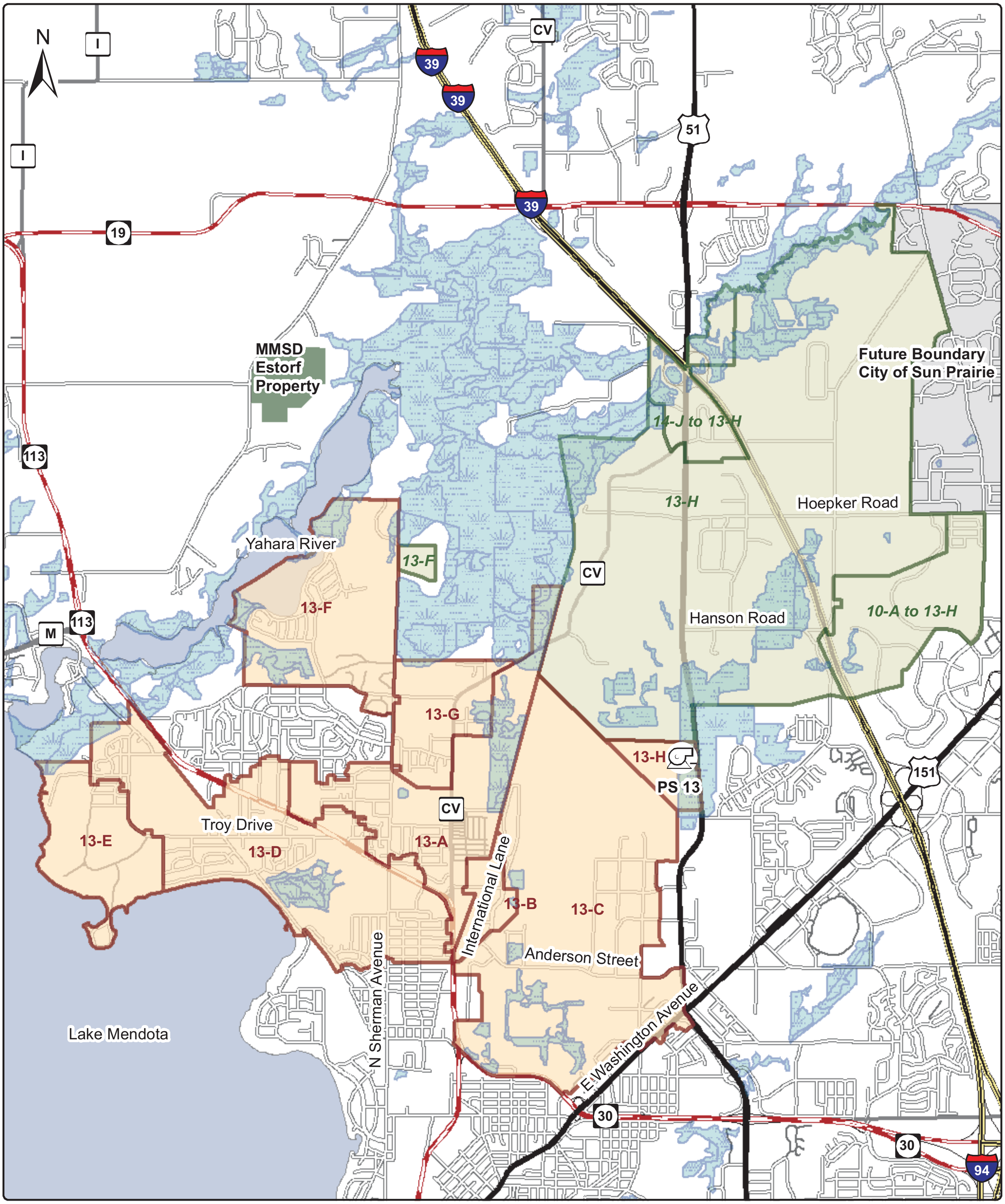
MADISON METROPOLITAN SEWERAGE DISTRICT
COLLECTION SYSTEM EVALUATION
2008

0 0.5 1 2 Miles

MMSD

Figure 14

10/23/2008



Sub-Basins in 2000
Potential by 2030
Potential by 2060
DNR Wetlands

PUMPING STATION 13 SUB-BASINS

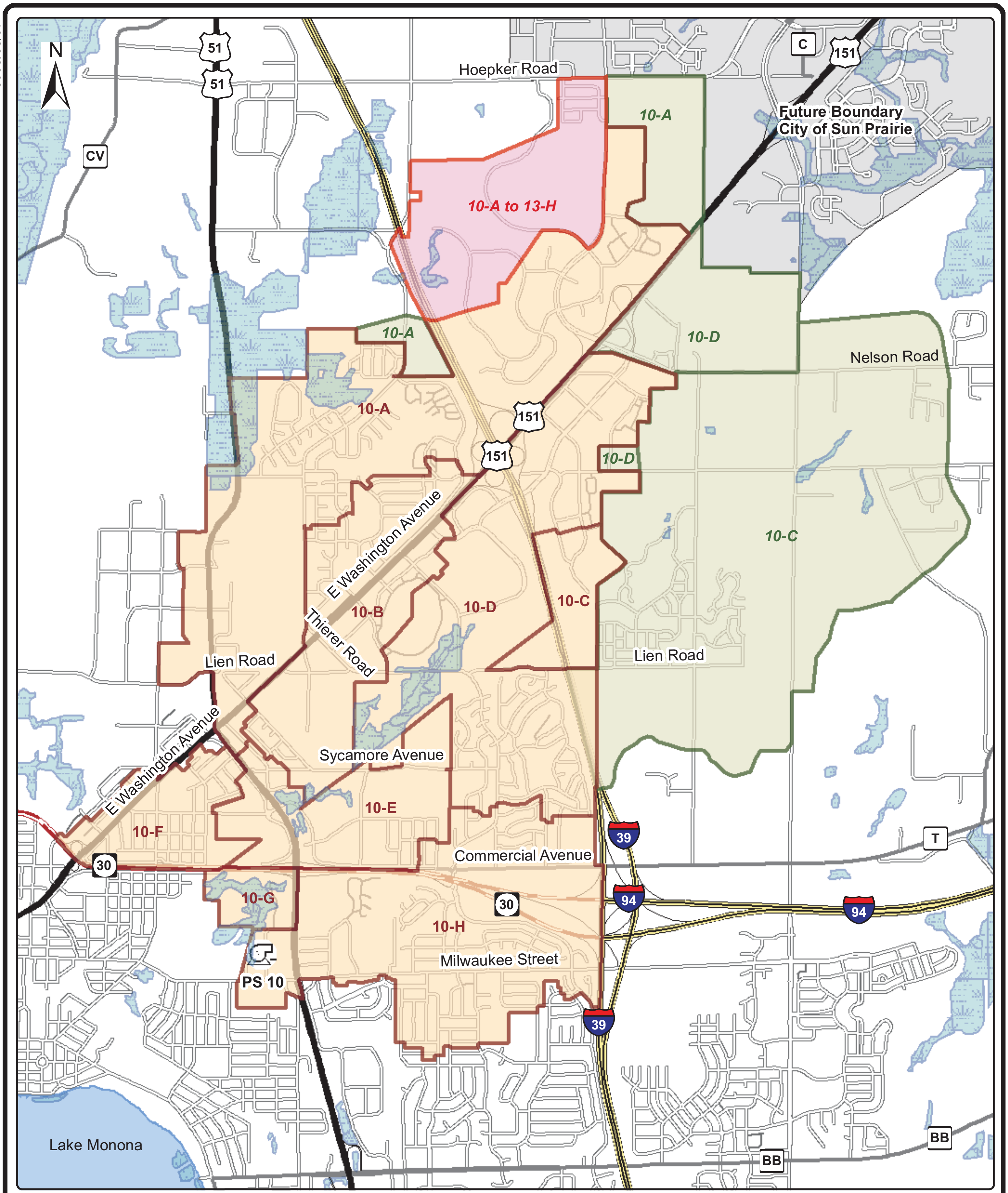
MADISON METROPOLITAN SEWERAGE DISTRICT
COLLECTION SYSTEM EVALUATION
2008

0 0.5 1 2 Miles

MMSD

Figure 15

10/23/2008



Sub-Basins in 2000
Removed by 2030
Potential by 2030
Potential by 2060
DNR Wetlands

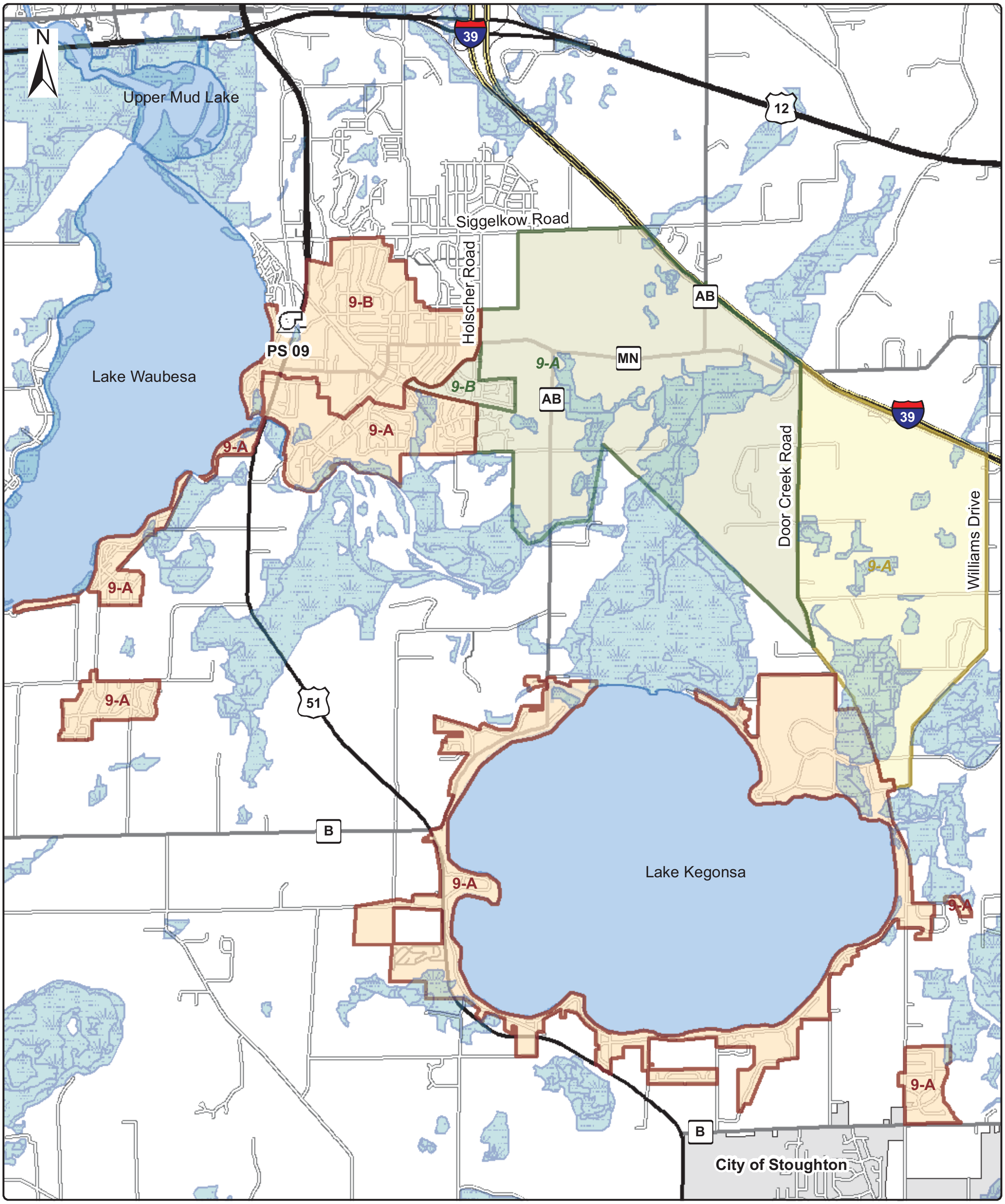
PUMPING STATION 10 SUB-BASINS

MADISON METROPOLITAN SEWERAGE DISTRICT
COLLECTION SYSTEM EVALUATION
2008

0 0.45 0.9 1.8 Miles

Figure 16

11/03/2008



Sub-Basins in 2000
Potential by 2030
Potential by 2060
DNR Wetlands

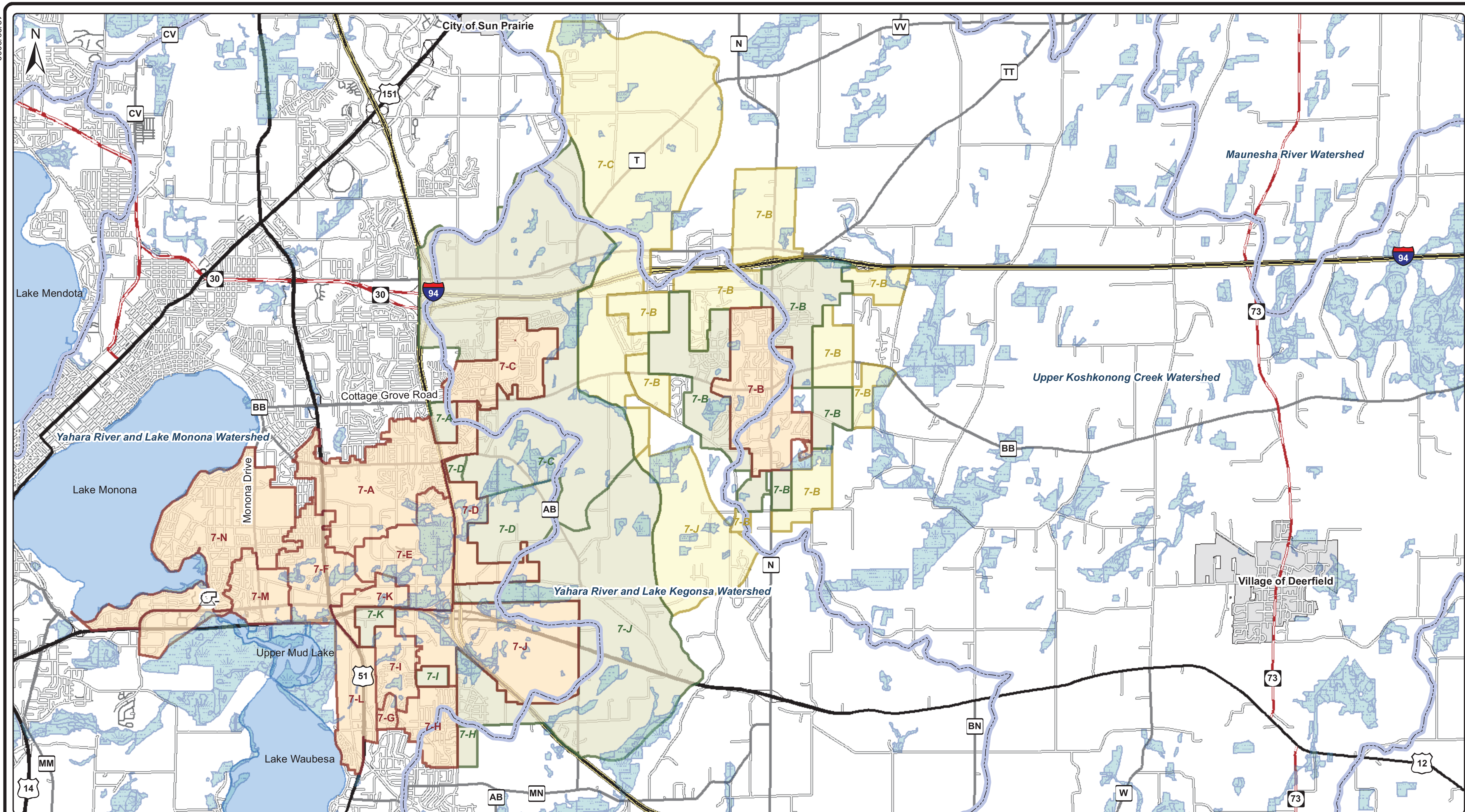
PUMPING STATION 9 SUB-BASINS

MADISON METROPOLITAN SEWERAGE DISTRICT
COLLECTION SYSTEM EVALUATION
2008

0 0.5 1 2 Miles

Figure 17

12/29/2008



DNR Wetlands
Sub-Basins 2000
Potential by 2030
Potential by 2060

PUMPING STATION 7 SUB-BASINS

MADISON METROPOLITAN SEWERAGE DISTRICT
COLLECTION SYSTEM EVALUATION
2008

0 1.5 3 6 Miles

Figure 18

Appendix A10
District Response to June (2008) High Flow Events

MADISON METROPOLITAN SEWERAGE DISTRICT

1610 Moorland Road
Madison, WI 53713-3398
Telephone (608) 222-1201
Fax (608) 222-2703

Jon W. Schellpfeffer
Chief Engineer & Director

Protecting Public Health and the Environment

Memo

To:	District Municipal Customers
From:	Jon W. Schellpfeffer, Chief Engineer and Director JWS
Subject:	**June 2008 High Flow Events**
Date:	July 17, 2008

The Madison Metropolitan Sewerage District is recovering from the extraordinary precipitation events and subsequent flooding that took place in early June. We trust things are also getting back to normal in your community. The District measured its all-time high peak flow rate of 100,000 gallons per minute and its all-time high one-day wastewater volume of 106 million gallons on June 9. The average daily flow received at the Nine Springs Wastewater Treatment Plant in June was 62 million gallons per day (mgd). Daily flows are still above 50 mgd. Typical volumes during periods of normal weather are about 40 mgd.

The high flows resulted from rain water that leaked into basements and rain water that flooded streets and low areas and leaked into manholes and through defects in sewer lines. The excess flow from these sources overwhelmed the District's conveyance system for a period of about 30 hours on June 8 and 9. Even though all pumping equipment was available and in operation during this entire event, many homes and businesses experienced basement back-ups from the sewer system. The District employed portable pumping equipment at two pumping stations to remove water from the sewer system and prevent further basement backups. Even so, the sewer system overflowed from manholes at several points. System overflows and the portable pump discharges ultimately reached nearby wetlands, streams, and the lakes.

Over the past fifteen years the District has experienced six or more significant storm events that have led to high flows in the sewer system. Many scientists now expect that storms of higher intensity and longer duration will be experienced more frequently in the future. To address this possibility and to lessen the likelihood of future events overwhelming the sewer system, the Madison Metropolitan Sewerage District is taking the following actions:

1. The District is reviewing its design standards for sizing interceptor sewers and pumping stations. The District currently provides an allowance for high flows in these facilities that varies from peak flow capacities 4.0 times greater than the

metrogro

average daily flows for facilities with average day design flows of one million gallons per day to peak flow capacities 2.5 times greater than the average daily flows for facilities with average day design flows of 20 million gallons per day. The review will include data from the storm events of the past fifteen years. If higher peaking factors are judged to be necessary, the schedule for construction of replacement interceptor sewers and pumping stations will need to be accelerated, and new and replaced facilities will be larger. This will lessen the likelihood of back-ups and overflows and will result in higher costs for service.

2. The District is reviewing its design standards for materials used in constructing interceptor sewers, including manholes, to assure that rain waters are less likely to leak into these facilities during heavy rains and floods. We encourage you to do the same. During our plan review process, we will pay particular attention to details in sewer extension plans that will be constructed in areas more prone to flooding.
3. The District is reviewing flow data and inspecting its existing interceptor sewers to identify and repair defects that allowed excessive rain water leakage into the District's system. We encourage you to make similar inspections and repairs in your local system.
4. The District is reviewing flow data from each municipality collected during the recent high flow events. We will attempt to identify likely areas in community sewer systems that experienced excessive leakage during the recent high flow events. The District will work with these communities to address these areas.
5. The District will make greater efforts to educate the public in the area of water conservation and how to prevent rain water from leaking into basements. Water conservation and reduced inflow will have positive impacts in both dry and wet weather.

Although the events of June were unprecedented, this will not be the last challenge of this type that we will face. The District is taking action now to better insure that future events will have less impact on the public and the environment.

Memo

To: File

From: Jon W. Schellpfeffer, Chief Engineer and Director

Subject: Responses to June 2008 High Flow Event

Date: June 3, 2009

The purpose of the memo is to review the various actions the District has taken since the high flow events of June 2008. At that time the District developed a list of actions it would take in response to that event, and those actions are the section headings used in this memo.

Review of Design Standards for Sizing Interceptors and Pump Stations.

Historically the District has used a peak flow factor based on information in the 1961 Greeley and Hansen Report on Sewerage and Sewage Treatment. The peak hourly flow (PHF) factor developed in that report is applied to average daily flows (ADFs) in the range of 1 mgd to 20 mgd as follows: PHF= $(ADF)^{0.842}$ x 4. For ADFs less than 1 mgd, the peaking factor is 4. For ADFs greater than 20 mgd, the peaking factor is 2.5. It appears that this factor may not provide sufficient capacity for all District interceptors and pump stations. For those interceptors and pump stations located closer to the lakes, it appears that the peak flow factor may need to be increased by about 1.0 to account for the higher peak flows experienced recently.

More analyses need to be done to determine more precisely in which areas of the District's service area this more conservative factor should be applied. These additional analyses will be included in the Collection System Facilities Plan and completed by the end of this year. Since application of a more conservative peaking factor would, in all likelihood, accelerate the need to increase the conveyance system capacity beyond what is practical or doable, initial efforts to reduce high flows will concentrate on hardening the interceptor system to prevent excessive levels of inflow and infiltration. The appropriate peaking factor as determined by a thorough analysis of available data will be applied to all future conveyance system projects.

Review of Design Standards for Interceptor and Manhole Materials.

During interceptor design, additional attention will be directed to pipe gasket details, connections between pipes and manholes, connections between manhole sections, and sealing between manhole decks and castings. All manhole castings will be provided with gasketed manhole covers and will be installed with chimney seals. These standards have been in use on new interceptors built in areas that could experience flooding, but will now be used for all new interceptor sewers. Where flooding is currently possible and whenever practical (for example, during pavement replacements), older manhole castings and chimneys will be replaced with new gasketed castings and chimney seals. This work will be performed by the District's Sewer Maintenance crew. Those manholes potentially subject to flooding should be upgraded by the end of this year; however, this will be an on-going effort as new information becomes available through regular inspections, anecdotally, or from information provided by others.

Review of Flow Data and Inspection of Existing Interceptors.

Review of flow data from the June 2008 high flow event identified a number of record high flows in the conveyance system. As mentioned earlier, those portions of the conveyance system in lower-lying areas around the lakes appeared to experience the highest of the high flows. The interceptors in these areas were the first inspected after the event to locate possible inflow sources.

Immediately after the June 2008 high flow events, the District's Sewer Maintenance crew began inspecting manholes in areas that had been flooded. One major source of inflow was discovered and repaired upstream of Pumping Station 12. Several manholes appeared to have been under water in other areas. Castings on those were raised, and those that were not equipped with gasketed covers and chimney seals were rebuilt using castings with gasketed covers and chimney seals. System inspection and the follow-up work to harden the District's system against inflow and infiltration is an on-going and routine part of the Sewer Maintenance crew's work. Larger more costly, but less urgent, projects are typically included in the District's capital budget for repair, rehabilitation, or replacement.

Review of Communities Flow Data.

As with the District's conveyance system, those communities located in low-lying areas and near the lakes experienced the highest flows. The District sent a memo to all of its municipal customers following the event. The memo summarized the record high flows and the actions the District planned to take in response to this and other recent high flow events. The memo also encouraged the communities to undertake an inspection and repair program for their local sewer systems. The District also has provided individual reports to the City of Madison, Town of Dunn Sanitary Districts 1 and 3, and Town of Windsor Lake Windsor Sanitary District. At this time, the District has not directed any community to take any specific action related to high flows.

Greater Efforts at Public Education.

In April of this year the District ran a series of four radio spots related to efforts home owners could take to prevent rain water from entering their basements (and then likely the sanitary sewer system). The District had budgeted $20,000 for this effort and has spent about $14,000 so far. Based on anecdotal feedback and increased hits on the District's website, it appears that we were successful in reaching a fairly large audience with this information. We will need to undertake follow-up public education efforts in this area.

Appendix A11
Public Participation

MADISON METROPOLITAN SEWERAGE DISTRICT

NOTICE OF PUBLIC HEARING
February 22, 2012

COLLECTION SYSTEM FACILITIES PLAN UPDATE

The Madison Metropolitan Sewerage District will hold a Public Hearing on **Wednesday, February 22, 2012 at 6:30 p.m. at the Nine Springs Wastewater Treatment Plant**, 1610 Moorland Road, Madison, WI, 53713. The hearing will be held in the Commission Room of the Operations Building, which is handicap accessible. MMSD staff will be present to answer questions and receive comments prior to a short presentation at 7:00 p.m.

The purpose of the hearing is to receive public input regarding submission of MMSD's Collection System Facilities Plan Update to the Wisconsin Department of Natural Resources. The Plan provides recommendations for improvements to the District's collection system facilities through the Year 2030. The Plan is available for public inspection at the Nine Springs Wastewater Treatment Plant on weekdays from 7:00 a.m – 4:00 p.m. It will also be made available for viewing at the District's website (www.madsewer.org).

Anyone interested is invited to attend this meeting. If you wish to comment but cannot be present at the public hearing, please submit a written statement by 3:00 p.m., Monday, February 20, 2012, to Mr. D. Michael Mucha, Madison Metropolitan Sewerage District, 1610 Moorland Road, Madison, WI 53713.

Dated this 7th day of February 2012.

D. Michael Mucha
Chief Engineer & Director, MMSD

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MADISON METROPOLITAN SEWERAGE DISTRICT
NOTICE OF PUBLIC HEARING
February 22, 2012
COLLECTION SYSTEM FACILITIES PLAN UPDATE

The Madison Metropolitan Sewerage District will hold a Public Hearing on **Wednesday, February 22, 2012 at 6:30 p.m. at the Nine Springs Wastewater Treatment Plant,** 1610 Moorland Road, Madison, WI, 53713. The hearing will be held in the Commission Room of the Operations Building, which is handicap accessible. MMSD staff will be present to answer questions and receive comments prior to a short presentation at 7:00 p.m. The purpose of the hearing is to receive public input regarding submission of MMSD's Collection System Facilities Plan Update to the Wisconsin Department of Natural Resources. The Plan provides recommendations for improvements to the District's collection system facilities through the Year 2030. The Plan is available for public inspection at the Nine Springs Wastewater Treatment Plant on weekdays from 7:00 a.m – 4:00 p.m. It will also be made available for viewing at the District's website (www.madsewer.org).

Anyone interested is invited to attend this meeting. If you wish to comment but cannot be present at the public hearing, please submit a written statement by 3:00 p.m., Monday, February 20, 2012, to Mr. D. Michael Mucha, Madison Metropolitan Sewerage District, 1610 Moorland Road, Madison, WI 53713.

Dated this 7th day of February 2012.

D. Michael Mucha
Chief Engineer & Director, MMSD

PUB. WSJ: February 10, 2012
#1879986 WNAXLP

(Signed) Sharon Scallon

(Title) Principal Clerk

Subscribed and sworn to before me on

Feb 10, 2012

Eileen M. Morgan

Notary Public, Dane County, Wisconsin

My Commission expires May 24th, 2013

MADISON METROPOLITAN SEWERAGE DISTRICT

1610 Moorland Road
Madison, WI 53713-3398
Telephone (608) 222-1201
Fax (608) 222-2703

D. Michael Mucha, P.E.
Chief Engineer & Director

Protecting Public Health and the Environment

February 7, 2012

Mr. Paul Woodard
City of Fitchburg Public Works Department
5520 Lacy Road
Fitchburg, WI 53711

RE: Notice of Public Hearing - MMSD Collection System Facilities Plan Update

Dear Mr. Woodard:

The Madison Metropolitan Sewerage District will hold a Public Hearing on **Wednesday, February 22, 2012 at 6:30 p.m. at the Nine Springs Wastewater Treatment Plant** regarding the above referenced facility plan. The hearing will be held in the Commission Room of the Operations Building, which is handicap accessible. MMSD staff will be present to answer questions and receive comments prior to a short presentation at 7:00 p.m.

The purpose of the hearing is to receive public input regarding submission of MMSD's Collection System Facilities Plan Update to the Wisconsin Department of Natural Resources. The Plan provides recommendations for improvements to the District's collection system facilities through the Year 2030. The Plan is available for public inspection at the Nine Springs Wastewater Treatment Plant on weekdays from 7:00 a.m – 4:00 p.m. It will also be made available for viewing at the District's website (www.madsewer.org).

Anyone interested is invited to attend this meeting. If you wish to comment but cannot be present at the public hearing, please submit a written statement by 3:00 p.m., Monday, February 20, 2012, to my attention at the following address:

Madison Metropolitan Sewerage District
1610 Moorland Road
Madison, WI 53713

Please feel free to contact Todd Gebert, of my staff, at 608-222-1201 (ext 235) with any questions regarding the facility plan and/or public hearing.

Sincerely,

D. Michael Mucha

D. Michael Mucha
Chief Engineer & Director

metrogro

Mailing List for Notice of Public Hearing
MMSD *Collection System Facilities Plan Update*
Wednesday, February 22, 2012

No.	Name	Representing	Address
1	Paul Woodard	City of Fitchburg	5520 Lacy Road, Fitchburg, WI 53711
2	Rob Phillips	City of Madison	210 Martin Luther King Jr. Blvd, Room 115, Madison, WI 53703
3	Shawn Stauske	City of Middleton	7426 Hubbard Avenue, Middleton, WI 53562
4	Dan Stephany	City of Monona	5211 Schluter Road, Monona, WI 53716
5	Ron Rieder	City of Verona	410 Investment Court, Verona, WI 53593
6	Mike Wolf	Town of Blooming Grove	1880 South Stoughton Road, Madison, WI 53716
7	Terri Winans	Waunona Sanitary District No. 2	3325 Thurber Avenue, Madison, WI 53714
8	Brenda Ayers	Town of Burke	5365 Reiner Road, Madison, WI 53718
9	Dan Paltz	Town of Dunn Sanitary District #1	3022 Waubesa Avenue, Madison, WI 53711
10	Tammy Rayfield	Town of Dunn Sanitary District #3	2879 Exchange Street, McFarland, WI 53558
11	John Ong	Town of Dunn Sanitary District #4	4725 Nora Lane, Madison, WI 53711
12	Michael Sherry	Town of Dunn - Kegona Sanitary District No. 2	P.O. Box 486, Stoughton, WI 53589
13	Rick Rose	Town of Madison	2120 Fish Hatchery Road, Madison, WI 53713
14	David Shaw	Town of Middleton Sanitary District No. 5	7555 W. Old Sauk Road, Verona, WI 53593
15	Gary Teigen	Town of Pleasant Springs Sanitary District No. 1	2083 Williams Drive, Stoughton, WI 53589
16	Rose Johnson	Town of Verona Utility District No. 1	335 N. Nine Mound Road, Verona, WI 53593
17	Shawn Haney	Town of Vienna	7161 County Highway I, DeForest, WI 53532
18	Bob Anderson	Town of Westport	5387 Mary Lake Road, Waunakee, WI 53597
19	Jeff Bartosiak	Town of Windsor	P.O. Box 473, Windsor, WI 53598
20	Victor Schneider	Lake Windsor Sanitary District	P.O. Box 411, Windsor, WI 53598
21	Kitty Repas	Morrisonville Sanitary District #1	P.O. Box 200, Morrisonville, WI 53571
22	Peter Byfield	Oak Springs Sanitary District	4534 South Hill Court, DeForest, WI 53532
23	Jim Hessling	Village of Cottage Grove	221 East Cottage Grove Road, Cottage Grove, WI 53527
24	Rebecca Simpson	Village of Dane	P.O. Box 168, Dane, WI 53529
25	Deane Baker	Village of DeForest	205 DeForest Street, DeForest, WI 53532
26	Tom Schroeder	Village of Maple Bluff	18 Oxford Place, Madison, WI 53704
27	Allan Coville	Village of McFarland	5115 Terminal Drive, P.O. Box 110, McFarland, WI 53558
28	Denny Lybeck	Village of Shorewood Hills	810 Shorewood Boulevard, Madison, WI 53705
29	Kevin Even	Village of Waunakee	500 W. Main Street, Waunakee, WI 53597

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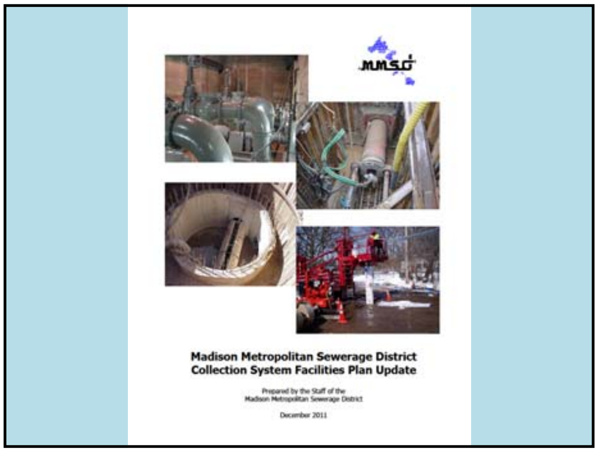
Collection System Facilities Plan Update

PUBLIC HEARING ATTENDANCE

Collection System Facilities Plan Update
Madison Metropolitan Sewerage District
Wednesday, February 22, 2012, 6:30 p.m.

NAME	REPRESENTING	ADDRESS
TODD GEBERT	MMSD	1610 MOORLAND RD. MADISON, WI
MIKE SIMON	MMSD	"

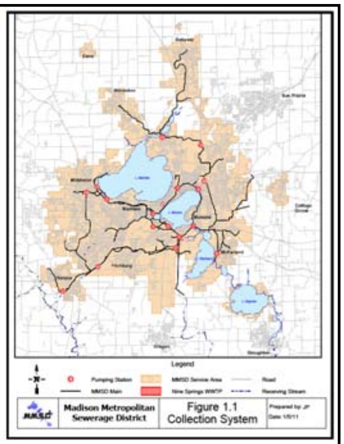


Presentation Outline

- **Overview of MMSD Collection System**
- **Purpose of Facility Plan**
- **Assessment Methodologies**
  - Capacity
  - Condition
- **Plan Initiatives and Recommendations**
- **Rate Impacts**
- **Questions**

MMSD Collection System Statistics

- 37 customers served
- Service area = 180.7 square miles
- Service population ~342,000
- Miles of gravity sewer = 94
- Number of pumping stations = 17
- Miles of force main = 29



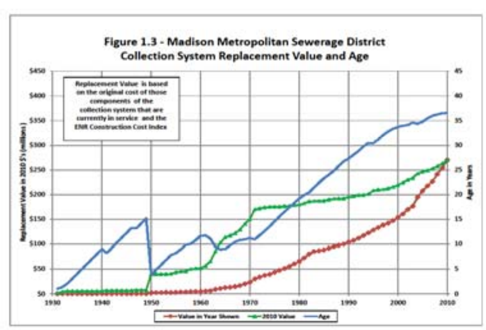
What is a Collection System Facilities Plan?

- MMSD's Facilities Plan **provides an assessment of existing collection system assets and identifies required system improvements to meet customer demands and future growth.**
- **Major collection system assets include:**
  - Pumping Stations
  - Intercepting sewers and manholes
  - Raw wastewater forcemains

Uses of Collection System Facilities Plan

1. **Satisfy WDNR Facility Planning requirements and approval of projects**
2. **Development of Capacity, Management, Operation and Maintenance (CMOM) program**
3. **Provide basis for planning and budgeting of capital improvement projects**

System Value and Age



Status of Recommended Projects from 2002 CSFP

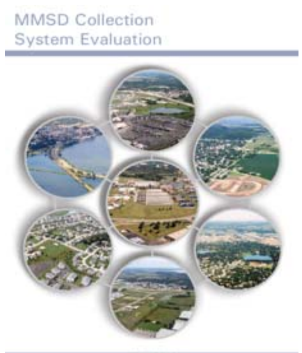
- 48 of 52 projects have been completed to date
- Remaining projects:

Project	Status	Projected Completion
New PS 18	Facility planning starting in 2011	2015
PS 18 – New forcemain	Facility planning starting in 2011	2015
PS 10 – I/I study	Pending	-
PS 14 – I/I Study	Recommended per CSFP Update	2012-2013

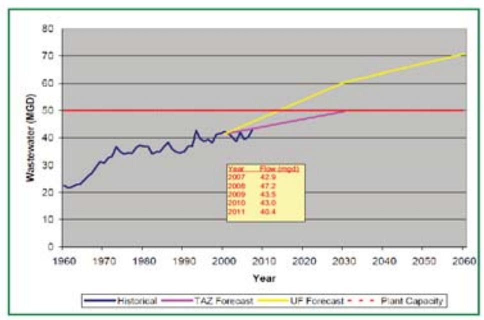
Major Focus Areas of Facilities Plan

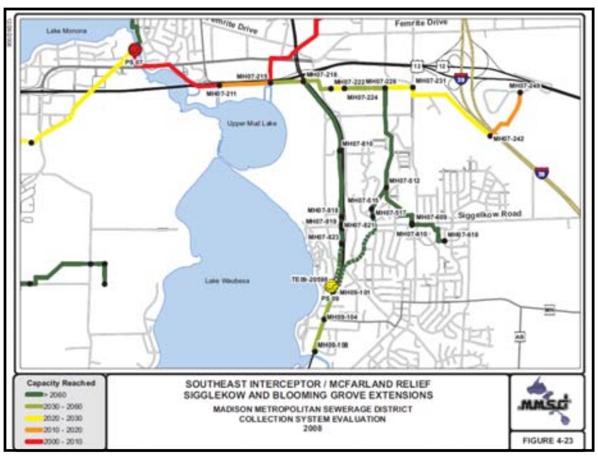
1. Asset management and CMOM
2. **System capacity and projected flows**
3. **Condition and needs assessment of major assets**
4. Special projects and diversions
5. Collection system maintenance
6. Addressing I/I issues and high flows
7. **Recommended projects and initiatives**

CARPC's MMSD Collection System Evaluation

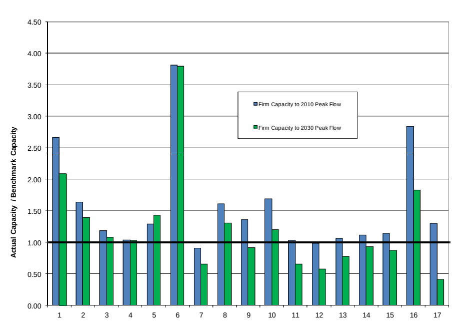


CARPC's Projected Wastewater Forecast





Pumping Station Capacity Analysis



Interceptor and Forcemain Capacity Analysis

Pumping Station Service Area	Total Gravity Interceptor Mileage in Service Area (miles)	Total Force Main Mileage in Service Area (miles)	Mileage Predicted to Reach Benchmark Capacity By 2020				Mileage Predicted to Reach Benchmark Capacity By 2030			
			Gravity Interceptors		Force Mains		Gravity Interceptors		Force Mains	
			(miles)	(%)	(miles)	(%)	(miles)	(%)	(miles)	(%)
PS1	1.71	3.67	0.00	0%	0.45	12%	0.00	0%	0.45	12%
PS2	2.73	3.29	0.41	15%	0.00	0%	0.41	15%	0.00	0%
PS3	0.72	0.005	0.72	100%	0.00	0%	0.72	100%	0.00	0%
PS4	1.55	0.03	0.00	0%	0.00	0%	0.00	0%	0.00	0%
PS5	3.00	0.42	0.00	0%	0.00	0%	0.00	0%	0.00	0%
PS6	1.91	1.37	0.00	0%	0.00	0%	0.00	0%	0.00	0%
PS7	19.76	2.96	4.44	22%	0.00	0%	8.39	42%	1.33	45%
PS8	14.64	2.60	2.39	16%	0.00	0%	3.22	22%	0.00	0%
PS9	0.63	1.24	0.00	0%	0.01	1%	0.05	9%	0.01	1%
PS10	6.59	2.10	2.07	31%	0.00	0%	2.07	31%	0.00	0%
PS11	10.04	0.79	1.21	12%	0.00	0%	5.29	53%	0.79	100%
PS12	7.86	0.91	0.67	8%	0.00	0%	0.67	8%	0.00	0%
PS13	2.96	0.49	0.00	0%	0.00	0%	0.36	12%	0.00	0%
PS14	15.84	0.85	0.88	6%	0.00	0%	3.49	22%	0.00	0%
PS15	1.97	2.80	0.00	0%	0.00	0%	0.04	2%	0.00	0%
PS16	1.63	1.93	0.00	0%	0.00	0%	0.53	32%	0.00	0%
PS17	2.52	3.11	0.00	0%	2.53	81%	0.00	0%	2.53	81%
Totals	96.06	28.57	12.80	13%	2.98	10%	25.25	26%	5.10	18%

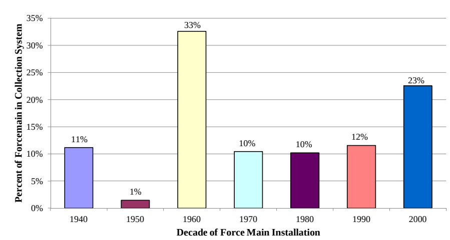
Condition Assessment - Pumping Stations

Facility	Adequacy/Condition of Mission Critical Category Likert Scale (1-5) - Category dependent (see text for explanation)						Total	Mean Weighting Factor (Sliding scale of 1 to 2)	Overall Rating	Ordinal Ranking (1 - 17)
	Peak Flow Capacity Qp (5 points)	Firm Flow Capacity Qf (5 points)	Power System Redundancy (5 points)	Mechanical Condition/ (5 points)	Structural Integrity (5 points)	Electrical Condition (5 points)				
PS NO. 1	1	1	1.5	1	1	1	6.5	1.75	11.38	**13**
PS NO. 2	1	1	1.5	1	1	1	6.5	1.95	12.68	**11**
PS NO. 3	2.5	1.5	3	1.5	4	1	13.5	1.00	13.50	**9**
PS NO. 4	3	2	3	1.5	2	3	14.5	1.15	16.68	**7**
PS NO. 5	1	1	1	1	1	1	6	1.20	7.20	**17**
PS NO. 6	1	1	1.5	1	1	1	6.5	1.30	8.45	**16**
PS NO. 7	3.5	3.5	2	2.5	1	2	14.5	2.00	29.00	**2**
PS NO. 8	1	1	1.5	1	1	1	6.5	1.85	12.03	**12**
PS NO. 9	2	2	1	1	2	1	9	1.10	9.90	**15**
PS NO. 10	1.5	1	1.5	1.5	1	1	7.5	1.70	12.75	**10**
PS NO. 11	3	3	3	3	2	4	18	1.70	30.60	**1**
PS NO. 12	2.5	4	4	2	2	3.5	18	1.50	27.00	**3**
PS NO. 13	3.5	3	4	1	3	3.5	18	1.30	23.40	**4**
PS NO. 14	2.5	2.5	4	1	3	3.5	16.5	1.15	18.98	**6**
PS NO. 15	1	2.5	4	2.5	4	3	17	1.25	21.25	**5**
PS NO. 16	1	1	2	2.5	2	1.5	10	1.10	11.00	**14**
PS NO. 17	3.5	3	1	4	1	1	13.5	1.15	15.53	**8**

Condition Assessment – Interceptor Sewers

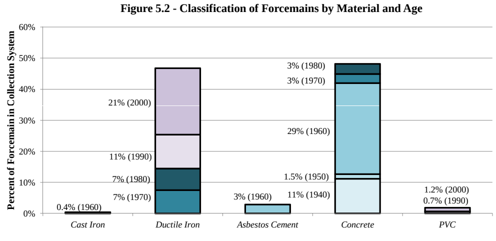
Condition Assessment – Forcemain Age

Figure 5.1 - Forcemain Age



Condition Assessment – Forcemain Material

Figure 5.2 - Classification of Forcemains by Material and Age



Collection System Initiatives

- Evaluate peaking factors for wet weather flows.
- Develop risk-based condition assessment tool to help identify and prioritize projects.
- Provide enhancements to District's televising program for sewer condition assessment

Figure 2 - Preliminary Route for Pumping Station 18 Forcemain

Rate Impacts

- Projects in Facility Plan to be funded from MMSD reserves and borrowed funds.
- Borrowed funds will average approximately $7M/yr for next 20 years.
- Annual service charge for average household in MMSD to increase by approximately $4/year (does not include increases associated with inflation in wages, materials, energy, etc).

Questions?

Appendix A12
Regulatory Approval

State of Wisconsin
DEPARTMENT OF NATURAL RESOURCES
101 S. Webster Street
Box 7921
Madison WI 53707-7921

Scott Walker, Governor
Cathy Stepp, Secretary
Telephone 608-266-2621
FAX 608-267-3579
TTY Access via relay - 711

WISCONSIN
DEPT. OF NATURAL RESOURCES

July 19, 2012

Project No. S-2012-0195
CWF No. 4010-39

Mr. Michael Mucha
Madison Metropolitan Sewerage District
1610 Moorland Road
Madison, WI 53713-3398

Subject: Approval For Sanitary Sewer Collection System Facilities Planning Report Update -- Madison Metropolitan Sewerage District

Dear Mr. Mucha:

The Department of Natural Resources has completed the review of the referenced wastewater facilities planning report update addressing proposed improvements for the sanitary sewer collection system at the Madison Metropolitan Sewerage District. The facilities planning report update is hereby approved. We concur with the report recommendations which preliminarily address various proposed sanitary sewer and sewage lift station rehabilitation / upgrade projects over approximately the next 20 years for the Madison Metropolitan Sewerage District.

If you believe that you have a right to challenge this decision, you should know that the Wisconsin statutes, administrative rules and case law establish time periods within which requests to review Department decisions must be filed.

To request a contested case hearing pursuant to section 227.42, Wis. Stats., you have 30 days after the decision is mailed, or otherwise served by the Department, to serve a petition for hearing on the Secretary of the Department of Natural Resources. All requests for contested case hearings must be made in accordance with section NR 2.05(5), Wis. Adm. Code, and served on the Secretary in accordance with section NR 2.03, Wis. Adm. Code. The filing of a request for a contested case hearing is not a prerequisite for judicial review and does not extend the time period for filing a petition for judicial review.

For judicial review of a decision pursuant to sections 227.52 and 227.53, Wis. Stats., you must file your petition with the appropriate circuit court and serve the petition on the Department within the prescribed time period. A petition for judicial review must name the Department of Natural Resources as the respondent.

Sincerely,

Thomas J. Mugan, P.E., Chief
Wastewater Section
Bureau of Water Quality

Stephen J. Smith, P.E.
Wastewater Section
Bureau of Water Quality

Cc: Mr. Todd Gebert – Madison Metropolitan Sewerage District (Madison, WI)
Amy Schmidt / Bernie Robertson -- Fitchburg Service Center
Maureen Hubeler -- CF/2 (CWF No. 4010-39)

CAPITAL AREA
REGIONAL PLANNING COMMISSION

210 Martin Luther King Jr. Blvd. Room 362 Madison, WI 53703 Phone: 608-266-41[illegible] Fax: 608-266-9117 www.CapitalAreaRPC.org info@CapitalAreaRPC.org

RECEIVED
JUN 20 2012
MADISON METROPOLITAN
SEWERAGE DISTRICT

June 18, 2012

Mr. Thomas Mugan
Wisconsin Department of Natural Resources
Central Office
Madison, WI 53703

RE: Collection System Facilities Plan Update, 2011
Madison Metropolitan Sewerage District (MMSD),
1610 Moorland Road, Madison, WI 53713

Dear Mr. Mugan:

We have reviewed the Madison Metropolitan Sewerage District's Collection System Facilities Plan Update, 2011. The flow and loading estimates used as the basis for this facilities plan are consistent with the range of the 2035 population forecasts for the MMSD service area. These forecasts were based on official population projections and a detailed evaluation of sub-basins within the MMSD service area, and are consistent with the *Dane County Water Quality Plan* as revised and updated through June 8, 2012. The detailed evaluation has been documented in our MMSD Collection System Evaluation study report dated January, 2009.

Please don't hesitate to contact us if we can be of further assistance.

Sincerely,

Kamran Mesbah, Director
Environmental Resources Planning

Cc: Mr. Michael Mucha, Chief Engineer and Director, MMSD
Mr. Todd Gebert, Collection System Engineer, MMSD