Madison Metropolitan Sewerage District
1610 Moorland Road • Madison, Wisconsin 53713

50-Year Master Plan Report

MMSD No. 8425001

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ABBRIVIATIONS

ADF	Average Daily Flow
A/O	Anaerobic/Aerobic
BOD_5	5-day Biochemical Oxygen Demand
CARPC	The Capital Area Regional Planning Commission
CFA	Conveyance Facilities Analysis
CTFM	Cross Town Force Main
CTH	County Highway
DAF	Dissolved Air Floatation
DMF	Daily Maximum Flow
DOA	Department of Administration
EBPR	Enhanced Biological Phosphorus Removal
ERW	Exceptional Resource Water
ES	Enforcement Standard
GBT	Gravity Belt Thickener
I/I	Inflow/Infiltration
MDC	Madison Design Curve
MGE	Madison Gas and Electric
MMSD	Madison Metropolitan Sewerage District
NH_4-N	Ammonium Nitrogen
NSVI	Nine Springs Valley Interceptor
NSWTP	Nine Springs Wastewater Treatment Plant
PAL	Preventive Action Limit
PHF	Peak Hourly Flow
PS	Pumping Station
RAS	Return Activated Sludge
SCB	Sludge Control Building
TAC	Technical Advisory Committee
TAZ	Traffic Analysis Zones
TKN	Total Kjeldahl Nitrogen
TM	Technical Memorandum
TMDL	Total Maximum Daily Loads
TP	Total Phosphorus
TPAD	Temperature Phased Anaerobic Digestion
TSS	Total Suspended Solids
UCT	University of Cape Town Process
UV	Ultraviolet
WAS	Waste Activated Sludge
W.C.	Water Column
WLA	Waste Load Allocation
WQBEL	Water Quality-Based Effluent Limit
WQC	Water Quality Criteria
WWTP	Wastewater Treatment Plant

1. EXECUTIVE SUMMARY

1.1. Introduction

The Madison Metropolitan Sewerage District (MMSD) is a municipal corporation created for the purpose of collecting and treating wastewater from the Madison metropolitan area. MMSD provides service to 43 municipal customers, including cities, villages, town utility districts and town sanitary districts in the area. MMSD's service area encompasses 177 square miles and serves a current population of approximately 330,000 people. MMSD owns and operates a regional wastewater conveyance system and the Nine Springs Wastewater Treatment Plant (NSWTP). On an average day, MMSD treats 41 million gallons of wastewater at the NSWTP. Each municipality within MMSD owns and operates its own wastewater collection system which feeds into MMSD's conveyance system.

The Madison area is one of the fastest growing areas in Wisconsin. An expanding population and potential limitations in the collection system and at the NSWTP have prompted MMSD to develop a 50-year master plan. An overarching objective of the planning process was to continue the District's practice of providing exceptional service at a reasonable cost to its customers while striking an appropriate balance between environmental, social, and economic impacts.

The team of Malcolm Pirnie Inc. and Strand & Associates was retained by MMSD to develop a 50-Year Master Plan for its wastewater conveyance and treatment systems. The Capital Area Regional Planning Commission (CARPC) provided population and wastewater flow forecasts and analyzed impacts on the conveyance system capacity. A technical advisory committee (TAC) consisting of professionals in various water resource management areas was also formed to guide this planning effort.

1.2. Planning Goals

The purpose of the 50-Year Master Plan is to provide MMSD with a general guidance tool for providing service over the next 50 year planning period. Key areas evaluated as part of the master planning process include:

- Population growth and resulting impacts
- Collection, conveyance and treatment capacity/condition
- Centralized vs. decentralized treatment
- Mitigation of inter-basin water transfers
- Effluent reuse
- Regulatory drivers

Detailed information regarding each of the above areas is presented in a series of nine technical memoranda that are attached to this report. A 14 member TAC provided input throughout the master planning process on the above areas and other relevant issues. Public input was solicited through two rounds of outreach efforts targeting customer communities and other interested parties.

The resulting Master Plan includes:

- Capital improvement projects that are already in MMSD's capital plan.
- Near-term alternatives to address increased capacity needs prior to 2030, with consideration given to existing and potential regulatory requirements, mitigation of inter-basin water transfers (watershed balancing) and effluent reuse.
- Long-term alternatives to be implemented between 2030 and 2060.
- Signposts/key factors that should be monitored to assist with future decision making (e.g. technology improvements or new regulatory initiatives).

More detailed facility plans will need to be developed as necessary at intervals of five to ten years. These facility plans should review the Master Plan and related population, flow and load projections, and re-evaluate the importance of planning variables. The detailed facility plans may continue with the general direction identified in the Master Plan, or may make modifications if warranted. The Master Plan will be a dynamic document and will be reviewed and updated with each facility plan. This is consistent with the way MMSD used its previous 50 year Master Plan.

1.3. Master Planning Alternative Development

In addition to capacity and condition related projects already included in MMSD's capital improvement plan, several master planning alternatives were developed to address centralized versus decentralized wastewater treatment, inter-basin water transfer, and effluent reuse. These master planning alternatives are classified into two groups: near-term alternatives which could be implemented between 2010 and 2030 and long-term alternatives which could be implemented between 2030 and 2060.

1.3.1. Near-Term Master Planning Alternatives

The following two near-term master planning alternatives have been developed. Implementation of either of these alternatives between 2010 and 2030 will address the wastewater treatment and conveyance system capacity needs in a portion of MMSD's service area, namely service in the Sugar River basin:

- **Alternative MP-1 – Westside Conveyance System Expansion**: This alternative would expand the existing conveyance system and continue the current model of centralized treatment at the NSWTP. This alternative includes four variations for pumping treated effluent from the NSWTP to different locations in the Sugar River basin.

- **Alternative MP-2 – Sugar River WWTP**: This alternative includes construction of a new high quality effluent treatment plant in the Sugar River watershed to treat wastewater generated in the PS 17 service area. Effluent from this plant would be discharged to the Sugar River. The current discharge point on Badger Mill Creek for 3.6 mgd of effluent from the NSWTP would remain. This alternative includes two variations to discharge treated effluent at different locations on the Sugar River. If either of these variations is implemented, none of the conveyance system capacity expansion projects included in alternative MP-1 would be necessary.

1.3.2. Long-Term Master Planning Alternatives

Long-term alternatives are those planning alternatives that cannot be implemented soon enough to provide relief in the conveyance system; however, they remain potentially viable options beyond the year 2030 for mitigating inter-basin transfers of water, or providing high quality effluent for reuse options. The following two alternatives have the potential to be implemented after 2030.

- **Alternative MP-3 – Centralized High Quality Effluent Treatment & Distribution:** This alternative includes construction of facilities at the NSWTP that would produce a high quality effluent for use in various applications including, stream flow augmentation, infiltration, industrial reuse, or turf irrigation. It also includes a pumping station and effluent force main to convey the effluent from the NSWTP to a point of use near PS 13.
- **Alternative MP-4 – Decentralized High Quality Effluent Treatment Facilities**:

  This alternative includes construction of facilities northeast of the Dane County Regional Airport. The new treatment plant would receive wastewater flows tributary to PS13 or both PS13 and PS14. Effluent from this facility could be used for stream flow augmentation to Starkweather Creek, wetland restoration at Cherokee Marsh, groundwater infiltration, industrial reuse water or turf irrigation.

1.4. Conclusions and Recommendation

All near-term and long-term planning alternatives were evaluated and ranked using the 10 ranking criteria developed during this planning effort. Conclusions and recommendations resulting from the Master Planning process include the following:

- Sufficient capacity exists at the NSWTP through 2030, provided there are no significant regulatory changes that would require a higher level of treatment.
- Sufficient space exists at the NSWTP for future expansion to serve the anticipated treatment needs through 2060.
- Alternative MP-1A, which reflects a continuation of the District's current service model with no provision for additional effluent conveyance capacity to the Sugar

River basin, achieves the lowest cost for providing wastewater conveyance and treatment service in MMSD's westside service area. However, this alternative will not mitigate future inter-basin transfers of water between the Sugar River basin and the Yahara River basin.

- Alternatives MP-1B, MP-1C and MP-1D, reflect a continuation of the District's current service model, but include pumping up to an additional 4.3 mgd of treated effluent to the Sugar River watershed to address the inter-basin transfer issue. The additional total life cycle costs to implement any of these alternatives would be $34 million assuming the current discharge limits to Badger Mill Creek and Badfish Creek stay unchanged.
- If mitigation of the inter-basin flow imbalance between the Sugar River basin and the Yahara River basin is determined to be necessary, satellite facilities in the Sugar River Basin may be favorable from both economic and non-economic standpoints to address west side conveyance capacity issues. More detailed cost and non-economic comparisons between alternatives with centralized treatment and alternatives with satellite treatment will need to be conducted since their life cycle costs and social and environmental benefits are closely ranked.
- Watershed balancing should be an important planning variable for future projects. Multiple planning alternatives could be implemented to mitigate inter-basin water transfers.
- Effluent reuse options should be evaluated during future facilities planning efforts, but will require partnerships to implement. Partnerships could potentially include other municipalities, water utilities, or public/private partnerships.
- Effluent discharge to Badfish Creek should continue, but the quantity could be impacted by watershed balancing and/or effluent reuse projects that decrease the amount of water that would otherwise have been discharged to Badfish Creek.
- Due to the long planning horizon, specific effluent reuse projects cannot be clearly defined for long term alternatives. Preliminary evaluation shows that the most cost effective approach to providing effluent for reuse options is to continue to treat wastewater centrally and construct an effluent delivery system(s).
- Reduction of inflow/infiltration (I/I) in the existing conveyance system is an important element for the areas that experience high groundwater during wet weather conditions. Effective I/I reduction could delay the need for major capital improvement projects required to expand the capacities of the conveyance system and treatment facilities. Therefore, programs to reduce I/I are recommended for all planning alternatives.
- Future service alternatives such as satellite plants in the upper Yahara River basin that would discharge to the Madison lakes and regional service options involving Sun Prairie and Stoughton are not further evaluated in the Master Plan. At this time, the strict regulatory constraints, high construction and operation costs, lack of proven technology, and potential strong public resistance make these service

alternatives less favorable than the services provided under the current operating model. However, these alternatives may become more viable in the future with changes in the political environment, water resource demand, or improvements in wastewater treatment technologies.

- Signposts such as technology improvements, regulatory trends, population growth rate, population shift, and changes in water use should be closely monitored during the planning period to allow MMSD to make appropriate adjustments to the Master Plan. The Master Plan will be a dynamic document and should be reviewed and updated periodically to reflect the impact of these types of key factors.

2. INTRODUCTION

2.1. Background and Objectives

The Madison Metropolitan Sewerage District (MMSD) is a municipal corporation created for the purpose of collecting and treating wastewater from the Madison metropolitan area. MMSD provides service to 43 municipal customers, including cities, villages, town utility districts and town sanitary districts in the area. MMSD's service area encompasses an area of 177 square miles and serves a current population of approximately 330,000 people. MMSD owns 94 miles of gravity interceptor sewers, 29 miles of wastewater force mains, 17 regional wastewater pumping stations, and the Nine Springs Wastewater Treatment Plant (NSWTP). Each municipality within MMSD owns and operates their own sewer collection system which ultimately feeds into MMSD's conveyance system.

The Madison area is one of the fastest growing areas in Wisconsin. An expanding population and potential limitations in the conveyance system and at the existing treatment plant have prompted MMSD to develop a 50-year master plan. An overall objective of the planning process was to continue the District's practice of providing exceptional service at a reasonable cost to its customers while striking an appropriate balance between environmental, social, and economic impacts. The team of Malcolm Pirnie Inc. and Strand & Associates was retained by MMSD to develop a 50-Year Master Plan for its wastewater conveyance and treatment systems. The Capital Area Regional Planning Commission (CARPC) provided population and wastewater flow forecasts and analyzed impacts on the conveyance system capacity. A technical advisory committee (TAC) consisting of professionals in various water resource management areas was also formed to guide this planning effort. Members of the TAC and their affiliations include:

- Ken Bradbury, Wisconsin Geologic & Natural History Survey
- Kevin Conners, Dane County Land and Water Resources Department
- Greg Fries, City of Madison Storm Water Utility
- John Hausbeck, Department of Health and Family Services
- Ken Johnson, Department of Natural Resources
- Sue Jones, Dane County Lakes and Watershed Commission
- Dick Lathrop, University of Wisconsin/Department of Natural Resources
- John Magnuson, UW-Madison Limnology Department
- Kamran Mesbah, Capital Area Regional Planning Commission
- Larry Nelson, City of Madison Engineering/Water Utility
- Daniel Noguera, UW-Madison Civil and Environmental Engineering Dept
- Ken Potter, UW-Madison Civil and Environmental Engineering Dept
- Bill Sonzogni, State Laboratory of Hygiene

The master planning project includes the following tasks:

Task 1 – Project Kickoff Meeting and Gather Initial Relevant Data
The project was started with a kickoff meeting at MMSD. The consultants used the meeting to introduce the team and to briefly review the project. The consultants collected relevant information and data for the development of the Master Plan.

Task 2 – Project Scoping and Visioning Workshop
After the kickoff meeting and preliminary data gathering, two workshops were conducted between the Technical Advisory Committee (TAC), MMSD staff and the consultants. The purpose of these workshops was to review and verify the overall goals, objectives and tasks of the master plan; brainstorm on planning concepts and future scenarios; and establish the mechanisms used to manage and execute the project.

Task 3 – Detailed Review of Current Plans and Wastewater Data
The information collected under Task 1 was reviewed in more detail to provide the framework and baseline conditions for the master planning effort. Previous population projections and collection system evaluations by the Capital Area Regional Planning Commission (CARPC) staff, the Dane County groundwater model, and other relevant information were also reviewed.

Task 4 – Evaluate Existing Facilities
The existing facilities were evaluated using the data collected in previous tasks. As part of the effort to define baseline conditions, a detailed evaluation of the NSWTP and the MMSD conveyance system (interceptor sewers, pump stations, and force mains) was conducted. The evaluation included a review of main sewer capacities, historical and projected capacities of pump stations, and identification of bottlenecks in the collection system.

Task 5 – Estimate Growth and Future Conditions
Using the information gathered in Task 1 and information developed by CARPC, this task looked at the expected population changes in the Madison region over the next 50 years, and how these changes would impact MMSD’s wastewater conveyance and treatment requirements.

Task 6 – Define Planning Variables
This task established the planning criteria and variables that were used in subsequent tasks. Multiple meetings and workshops were held with the TAC, MMSD staff and consultants to assist with this effort.

Task 7 – Scenario Planning Workshop
Two scenario planning workshops were conducted with the TAC, MMSD staff and consultants to review planning variables to assure that all potentially significant applicable factors were considered, that relevant historical perspectives were considered, and that critical uncertainties were identified.

Task 8 – Develop and Evaluate Planning Alternatives
Based on the initial planning, definition of planning variables, and the team's scenario planning work, a range of planning alternatives were developed that would enable MMSD to address future service area changes such as population growth and distribution, water quality and availability, regulatory requirements, and public expectations and preferences. These planning alternatives involved changes and improvements to the NSWTP and the conveyance system as well as treatment and discharge alternatives.

Task 9 – Develop Rating Criteria
Under this task, the planning alternative ranking and evaluation criteria were developed. Weighted scores were assigned to each ranking criterion according to their level of importance as determined collectively by the TAC, MMSD staff and the consultant team.

Task 10 – Rank Options and Develop Pros and Cons
Under this task, all planning alternatives were ranked and evaluated using the ranking criteria developed in Task 9. Advantages and disadvantages of each planning alternatives were discussed, and recommendations were made regarding implementation of near-term and long-term solutions by MMSD.

Task 11 – Develop Draft Report
Most of the previously described tasks have Technical Memos associated with them. Information in these Technical Memos formed the basis for developing a comprehensive draft report. All work tasks, evaluations, and recommendations were summarized in the draft report.

Task 12 – Develop Final Report
Based upon the feedback gathered on the draft report, the Final Report was prepared and delivered to MMSD.

Task 13 – Public Involvement
Efforts to engage the public and key stakeholders were conducted in two phases. The first phase was conducted early in the planning effort and included the development and distribution of an educational "fact sheet" and questionnaire, compilation of the questionnaire responses, and presentations of the master planning process and elements to numerous audiences.

The second phase of the public involvement effort involved District staff presenting preliminary findings, conclusions and recommendations in various public forums, including eighteen meetings of various public bodies where the presentation was included as part of their regular meeting, and an open house held at MMSD's offices.

2.2. Plan Basis

Information from the following documents, written prior to the Master Plan, was reviewed and considered during the planning process:

1. MMSD Facility Plan Upgrade, Volume 1-4, October, 1994
2. MMSD Nine Springs WWTP 9th Addition, Preliminary Design Report, June, 1995
3. Evaluation of Alternative Management Strategies – Dane County Regional Hydraulic Study, August, 1997
4. MMSD Facility Plan Report – Nine Springs WWTP 10th Addition, Volume 1-3, January, 2000
5. MMSD Vision, Goals and Strategies, 2nd Edition, March, 2003
6. MMSD Nine Springs WWTP 10th Addition, Preliminary Design Report, April, 2002
7. Madison MSD Collection System Facilities Plan, July, 2002
8. Dane County Water Quality Plan – Summary Plan, September, 2004
9. 2004 Modeling and Management Program – Dane County Regional Hydrologic Study, September, 2004
10. Seventy-Sixth Annual Report of the Commissioners of the Madison Metropolitan Sewerage District, 2005
11. MMSD Report on Sewerage and Sewage Treatment by Greeley & Hansen, January, 1961
12. Draft of Dane County Comprehensive Plan, 2007
13. City of Madison, Wisconsin Comprehensive Plan, 2006
14. MMSD WPDES permit issued in 2004.

2.3. Project Documentation

The completion of the master planning project tasks was documented through a series of technical memoranda (TMs) as summarized in Table 2-1. Table 2-1 also includes the issues discussed in each TM and the appendix where each TM can be found.

Table 2-1. Master Plan Technical Memoranda

TM No.	Document Title	Document Content	Master Plan Report Appendix
1	Review of Existing Treatment Facilities	This TM provides an evaluation of the existing flows and loadings to the NSWTP, unit capacities of the existing liquid treatment and solids disposal facilities, plant hydraulics, site considerations, electrical distribution systems, and operation and maintenance facilities.	A

TM No.	Document Title	Document Content	Master Plan Report Appendix
2	Flow and Loading Projections	This TM documents projected flows and loadings for the 50-year planning period. The memorandum presents information regarding the plant influent flow and loading projections, and internal loadings which will result from the projected influent flows and loadings. The projected internal flows and loadings were compared with the rated unit process capacities determined in TM-1. The comparison results provide information to be used for identifying the system needs for the planning period.	B
3	Conveyance Facilities Analysis (CFA)	This TM reviews the existing MMSD conveyance infrastructure with regard to age and condition of the infrastructure asset, and the ability to meet projected capacity requirements for the planning years assuming all wastewater will continue to be treated at the NSWTP.	C
4	Planning Variables	This TM documents a workshop held with the TAC and key MMSD staff to identify and discuss major planning variables that will govern or impact MMSD's available options for continuing to provide high quality services over the 50-year master planning period.	D
5	Regulatory Review and Analyses	This TM reviews existing and foreseeable future regulatory issues potentially affecting MMSD's planning and operations in the next 50 years.	E
6	Scenario Planning Workshops	This TM documents the two scenario planning workshops held with the Technical Advisory Committee (TAC) and key MMSD staff to identify and discuss possible future scenarios that may occur during the 50-year planning period and their implications to MMSD's available options for continuing to provide high quality services over the planning period.	F
7	Development of Planning Alternatives	This TM develops projects and groups them into potential planning alternatives that provide different approaches to meet the needs of the MMSD during the next 50 years.	G
8	Planning Alternative Ranking Criteria	This TM identifies the applicable ranking criteria to be used for master planning alternative ranking and determines appropriate level of importance for all ranking criteria to be used in the planning alternative evaluation.	H
9	Planning Alternative Ranking and Evaluation	This TM refines the master planning alternatives developed by TM-7; determines the life cycle costs for the selected master planning alternatives; evaluates and ranks planning alternatives using the criteria and methods developed in TM-8; recommends the best near-term planning alternatives for implementation, evaluates and identifies long-term planning alternatives and provides general guidance for potential implementations.	I

3. FLOWS/LOADINGS

3.1. Current Flows and Loadings

Influent loadings to the plant consist of raw wastewater delivered from the MMSD service area via four force mains and septage holding tank, landfill leachate and other wastes that are trucked to the plant. Historical flows and loadings to the plant were analyzed by examining daily average plant records for the period of January 1996 through December 2007. The annual average plant flows, concentrations, and loadings are presented in Tables 3-1 and 3-2. Detailed current flow and loading analyses are provided in Appendix A, TM1 – Review of Existing Treatment Facilities.

Table 3-1. Historical Daily Average Raw Influent Flow Characteristics for 1996-2007

Year	Flow (mgd)	TSS (mg/L)	BOD_5 (mg/L)	TKN (mg/L)	TP (mg/L)
1996	38.18	203	209	30.3	6.64
1997	36.92	208	220	31.6	6.54
1998	41.12	205	208	30.9	6.35
1999	41.59	208	208	30.9	6.07
2000	42.10	229	218	31.8	6.07
2001	41.76	222	216	32.2	5.88
2002	40.14	248	224	33.6	6.07
2003	38.56	261	243	35.2	6.49
2004	41.93	251	231	33.9	6.21
2005	39.37	243	245	37.5	6.39
2006	41.22	229	245	38.2	6.29
2007	42.88	215	240	36.4	5.95
Average	40.69	226	225	33.5	6.25

Table 3-2. Historical Daily Average Raw Influent Loadings for 1996-2007

Year	TSS (lb/day)	BOD_5 (lb/day)	TKN (lb/day)	TP (lb/day)
1996	68,116	69,918	10,020	2,150
1997	65,162	69,954	9,967	2,036
1998	69,414	71,424	10,569	2,180
1999	70,843	71,481	10,741	2,109
2000	78,127	75,424	11,045	2,102

Year	TSS (lb/day)	BOD_5 (lb/day)	TKN (lb/day)	TP (lb/day)
2001	76,269	74,933	11,162	2,045
2002	81,509	75,107	11,204	2,039
2003	83,769	78,115	11,342	2,087
2004	86,915	80,860	11,915	2,186
2005	80,197	81,648	12,439	2,132
2006	78,214	83,722	13,185	2,165
2007	75,592	84,396	12,955	2,125
Average	76,712	76,796	11,462	2,111

In spite of the wet weather periods that occurred during 1996, 2000 and 2004, the historical raw wastewater flow to the plant appears to have been relatively stable over the period of record. The 5-day biochemical oxygen demand (BOD_5), total suspended solids (TSS), total Kjeldahl nitrogen (TKN) and ammonium nitrogen (NH_4-N) concentrations and loadings in the raw wastewater appear to have been steadily rising over the past 12 years. However, total phosphorus (TP) concentrations and loadings appear to have been relatively stable over the period of record. Detailed information on current flows is included in TM1- Review of Existing Treatment Facilities.

3.2. CARPC Population and Flow Projections

As a part of the master planning for MMSD's sewerage collection system, the CARPC developed population and raw wastewater flow projections for different planning years between Year 2010 and Year 2060. These flow projections were used in estimating future flows for the MMSD wastewater treatment facilities and conveyance system. Major planning period service area populations and total flows are presented in Table 3-3. Detailed projected flows based on service areas and pumping stations are included in Appendix C, TM3 – Conveyance Facilities Analysis (CFA).

Table 3-3. Population and Flow Projection Summary

Year	Population	Average Raw Wastewater Flow (mgd)
2030*	406,000	47
2030**	431,000	50
2060*	491,000	53
2060**	560,000	60

** Population and flow based upon the projections developed by CARPC using Wisconsin Department of Administration (DOA) data and represent the low estimates.*
***Population and flow based upon the projections developed by CARPC using Traffic Analysis Zones (TAZ) data and represent the high estimates.*

Wastewater Loading Projections

In the previous facilities planning studies by the MMSD, the following three methods have been used to project future loadings for BOD_5, TSS, TKN, and TP:

- **Method based on per capita loading factors and projected future population.** This method determined the current per capita loading factors by dividing current average loadings by the current population of the MMSD service area. The resulting per capita loading factors were then multiplied by projected future populations to project future loadings.
- **Method based on current waste-load strength and projected future raw wastewater flow rates.** This method determined the current average concentrations of the parameters of interest and then multiplied the concentrations by projected future flow rates to project future loadings.
- **Method based on historic loading trends.** This method plotted historical raw wastewater loadings and then projected future loadings based on linear regression of the historical loading trends.

Based on previous facilities planning studies by the MMSD, linear projection based on historical loading trends has proven to be a reasonable and reliable method for predicting the BOD_5, TSS, and TKN loadings to the plant. However it is not appropriate in predicting TP loadings due to lack of long term influent TP monitoring data. Because of the stability of the influent TP concentration and unclear TP loading trends, the method based on waste-load strength was used to project TP loadings in the planning period. Detailed information is included in TM2- Flow and Loading Projections.

The projected wastewater loadings to the NSWTP at different planning years are presented in Table 3-4. Detailed plant loading projection analyses are provided in Appendix B, TM2 – Flow and Loading Projections.

Table 3-4. Total Plant Loading Projection

Parameter	Year 2030	Year 2060
BOD_5 (lbs/d)	122,000	173,000
TSS (lbs/d)	127,000	179,000
TKN (lbs/d)	20,000	28,000
TP (lbs/d)	2,700	3,200

The current and projected future flows and loadings were used to estimate the current capacity utilization rates of the existing conveyance system and treatment facilities and the additional future capacity needs at different planning years.

4. CONVEYANCE SYSTEM

4.1. Background

The existing MMSD conveyance infrastructure was evaluated with regard to condition, age, and the ability to meet projected capacity requirements for the year 2020, 2030, and 2060 assuming all wastewater will continue to be treated at the NSWTP. The evaluation provided a baseline for comparison of potential alternatives to treating wastewater at the NSWTP. Future flows used in the evaluation were based on the CARPC analysis of population and flows prepared for the MMSD 2008 Collection System Facilities Plans. Detailed analyses of the existing conveyance system are provided in Appendix C, TM3 – Conveyance Facility Analysis (CFA).

4.2. Existing Conveyance System

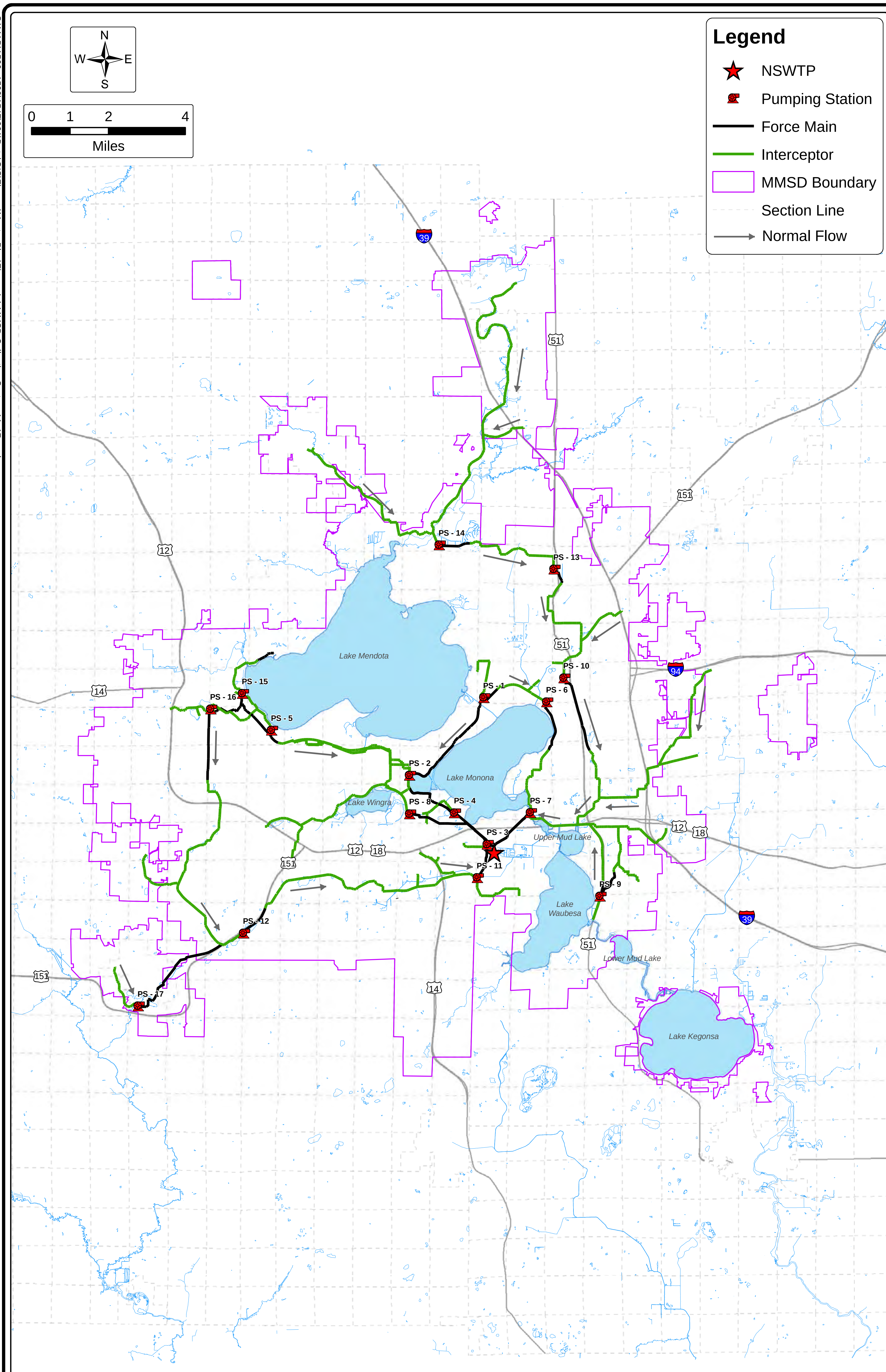
MMSD owns 94 miles of gravity interceptor sewers, 29 miles of wastewater force mains, and 17 regional wastewater pumping stations. Each municipality within MMSD owns and operates their own sewer collection system which ultimately feeds into MMSD's conveyance system. All 17 pumping stations convey wastewater to the NSWTP. All wastewater treated at the NSWTP is pumped to the treatment plant. The MMSD service area is divided into 2 service areas: an Eastside area and a Westside area. The Eastside area includes service areas for pumping stations 6, 7, 9, 10, 13 and 14. The Westside area includes service areas for pumping stations 1, 2, 3, 4, 5, 8, 11, 12, 15, 16 and 17).

An overall system schematic is presented in Figure 4-1. Detailed pumping station information is presented in Table 4-1.

Table 4-2 summarizes the current hydraulic capacities of pumping stations. Table 4-3 presents the 2008 Collection System Facilities Priority Ranking prepared by MMSD staff.

Table 4-4 summarizes MMSD's existing force mains. The comparison of the capacities of the force mains to the projected peak flows is based on the flows contained in the 2008 MMSD Collection System Evaluation prepared by the CARPC. Average daily flows for each of the pumping stations from 1996 through 2007 are summarized in Table 4-5.

Wastewater is conveyed to the pumping stations through interceptor sewers. Table 4-6 provides an overview of the adequacy of the capacity of interceptor sewers. The Madison Design Curve is used for estimating peak flows in the conveyance system, based on the average day flows. Refer to Section 1 of TM 3 in Appendix C for a detailed description of the Madison Design Curve.



S:\MAD\1500--1599\1547\001\Data\GIS\Figures\Master Plan\Figure 4-1 MMSD Collection System 11x17.mxd

FIGURE 4-1

STRAND ASSOCIATES, INC.® ENGINEERS

MALCOLM PIRNIE

EXISTING MMSD COLLECTION SYSTEM
NORMAL AND ALTERNATIVE OPERATING MODES

MMSD 50-YEAR MASTER PLAN

MMSD's 2002 Collection System Facilities Plan and annual reports provide a summary of televising done to review the condition of its existing system. Table 4-7 summarizes the past history of MMSD in reviewing system condition.

Contributory customers also have pumping stations included in their own collection systems. Table 4-8 summarizes contributory customer pumping stations. Collection systems owned by the contributory customers convey their wastewater to these pumping stations. The total length of collection sewers connected to MMSD's system was approximately 1,332 miles in 2008. MMSD maintains some of these pumping stations on a contract basis with its customers as noted in Table 4-8. Average daily flows from each of these contributory customers for the years 2000 through 2007 are summarized in Table 4-9.

Table 4-1. MMSD Regional Pumping Station Description

Pumping Station	Address	Description
PS 1	104 North First Street, Madison	This station receives flow only from its gravity drainage service area. Wastewater is pumped to PS 2 via the Cross Town Force Main (CTFM) or to PS 6.
PS 2	833 West Washington Ave, Madison (Brittingham Park)	This station receives flow from its gravity drainage service area plus a portion of the flow discharge by PS 1.
PS 3	Nine Springs, Madison	This station receives flow only from its gravity drainage service area. The station discharges to a force main shared by PS 2, PS 3, and PS 4.
PS 4	522 John Nolen Drive, Madison	This station receives flow only from its gravity drainage service area. The station discharges to a force main shared by PS 2, PS 3, and PS 4.
PS 5	5221 Lake Mendota Drive, Madison (Spring Harbor)	This station receives flow on a routine basis from its gravity drainage service area but may receive flow bypassed around either PS 15 or PS 16 in alternate operating modes.
PS 6	402 Walter Street, Madison (Olbrich Park)	This is one of three stations that pump to PS 7. PS 1 is piped to discharge to PS 6. Under 2010 operations, a portion of PS 1 flows are discharged to PS 6 while the majority of flows are discharged to PS 2.
PS 7	6300 Metropolitan Lane, Madison	PS 7 is the MMSD's largest station and in many ways its most critical pumping station. This station discharges directly to the NSWTP.
PS 8	967 Plaenert Street, Madison	This station receives flow from its gravity drainage service area in addition to pumped flows from PS 5 and PS 15.
PS 9	4612 Larson Beach Road, Mc Farland	This is one of the three stations that discharge to PS 7. There are no contributory MMSD pumping stations upstream of PS 9.

Pumping Station	Address	Description
PS 10	110 Regas Road, Madison (Main Post Office)	This is one of three stations that pumps to PS 7. PS 14 and PS 13 also contribute flow to PS 10.
PS 11	4760 East Clayton Road, Madison	This station discharges directly to the NSWTP and is the fourth largest pumping station contributing directly to the NSWTP. It receives flow from PS 12.
PS 12	2739 Fitchrona Road, Madison	This station receives pumped flow routinely from PS 16 and PS 17 and as an alternate operating mode PS 15. Flows from this station are pumped to PS 11.
PS 13	3634 Amelia Earhart, Madison (Truax Field)	This station discharges to PS 10 and receives flow from PS 14.
PS 14	5000 School Road, Madison	This station discharges to PS 13. There are no contributory MMSD pumping stations upstream of PS 14.
PS 15	2115 Allen Boulevard, Madison (Marshall Park)	This station receives flow only from its gravity drainage service area. This station normally pumps to PS 8, but can also pump to PS 16.
PS 16	1301 Gammon Road, Madison	This station ordinarily only receives flows from its gravity drainage service area. Alternatively flows from PS 15 may be pumped to PS 16.
PS 17	407 Bruce Street, Verona	This station only receives flows from its gravity drainage service area.

Table 4-2. Existing Pumping Station Summary (2010)

Pumping Station	Location	Original Construction	Capacity (mgd)		Completed or Proposed Upgrade	Potential Flow Diverted to Other Treatment Plants[6]	Existing or Potential Internal Flow Diversion	Gravity Service Area (acres)		
			Maximum[4]	Firm[5]						
1	104 North First Street	1950			2004		PS 13 or PS 14 (Future)	2,824	2,824	2,824
	To Pumping Station 6		18.0	15.0			PS 6			
	To Pumping Station 2		20.3	20.3			PS 2			
2	833 W. Washington Ave (Brittingham Park)	1964	41.0	41.0	2004		PS8, PS 1	923	1,179	1,179
3	Nine Springs	1959	1.5	1.5				514	514	514
4	522 John Nolen Drive	1967	4.2	4.2				1,331	1,331	1,331
5	5221 Lake Mendota Drive (Spring Harbor)	1996	3.6	3.6			PS2	1,101	1,016	1,016
6	402 Walter Street	1950	24.2	24.2	2010		PS 10	2,604	2,784	2,784
7	6300 Metropolitan Lane	1950	45.0	39.0	1992	Stoughton Plant, Sun Prairie Plant, Mendota	PS 18 (Future)	9,265	19,221	26,032
8	967 Plaenert Drive	1964	34.1	34.0	2010		PS 2	8,160	7,904	7,904
9	4612 Larson Beach Road, McFarland	1962	4.5	4.5		Stoughton Plant		2,615	4,955	6,495
10	110 Regas Road	1965	42.2	42.2	2004	Mendota Plant	PS 6	5,374	7,404	7,404
11	4760 East Clayton Road	1966	31.2	25.5	Rehabilitation Scheduled for 2013-2015	Sugar River		7,345	10,014	12,964
12	2739 Fitchrona Road	1969	23.5	16.6	Rehabilitation Scheduled for 2013-2015	Sugar River		4,548	8,253	8,482
13	3634 Amelia Earhart Drive	1970	20.2	20.0	Firm Capacity 2008	Mendota Plant		5,041	9,349	9,349
14	5000 School Road	1971	15.6	15.0	Firm Capacity 2008	Mendota Plant		8,202	16,710	21,735
15	2115 Allen Blvd	1975	8.8	5.8			PS 16, PS 5	3,463	6,275	7,194
16	1301 Gammon Road	1982	18.7	18.7				3,647	4,994	5,221
17	407 Bruce Street, Verona	1996	4.6	4.6		Sugar River		1,902	9,166	10,027
								68,859	**113,893**	**132,455**

Notes:

1. Priority Ranking are as established in the MMSD Collection System Facilities Plan Update-2008 adjusted for completed or projects schedule for completion prior to 2010
2. Station Capacities are as indicated in the Collection System Facilities Plan Update-2008 except for those stations noted as having upgrades completed or underway. Capacities for those stations are then new capacities after the upgrade.
3. MMSD currently has 43 million gallons of storage for treated effluent in the equalization basins on-site. If the storage is full, flow will be routed to Nine Springs Creek.
4. Maximum capacity represents the maximum pumping capacity of the pumping station.
5. Firm capacity represents the maximum pumping capacity of the pumping station with the largest pump out of service.
6. The Stoughton and Sun Prairie plants are existing plants and the Mendota and Sugar River plants are potential plants identified by the Master Plan. Detailed discussions on flows that will be diverted to these plants in different alternatives are included in the Chapter 7 of this report.

Table 4-3. Collection System Facilities Plan Priority Ranking (2008)

PS	Adequacy, Condition of Mission Critical Category: Maximum Flow Capacity (5 Points)	Firm Flow Capacity (5 Points)	Power System Redundancy (5 Points)	Mechanical Condition (5 Points)	Building and Structural Condition (5 Points)	Electrical Condition (5 Points)	Total	Station Weighting Factor	Overall Rating	Ordinal Ranking
1	1	1	1	1.5	1	1	6.5	1.75	11.38	12
2	1	1	1	1.5	1	1	6.5	2.00	13.00	10
3	2	2	3	2	4	1	14	1.00	14.00	9
4	3	3	3	2	2	3	16	1.10	17.60	6
5	1	1	1	1	1	1	6	1.20	7.20	17
6	1	1	1	1	1	1	6	1.30	7.80	15
7	4	4	2.5	2	1	1.5	15	2.00	30.00	1
8	1	1	1	1	1	1	6	1.90	11.40	11
9	1	1	1	1	2	1	7	1.05	7.35	16
10	1	1	1	1.5	1	1	6.5	1.60	10.40	13
11	3	4	3	2	2	4	18	1.50	27.00	3
12	3	5	4	2	2	3.5	19.5	1.50	29.25	2
13	3	3	4	1	3	3.5	17.5	1.10	19.25	4
14	2	2	4	1	3	3.5	15.5	1.10	17.05	7
15	1	1	4	2	4	3	15	1.20	18.00	5
16	1	1	2	2	1	2	9	1.05	9.45	14
17	5	5	1	3	1	1	16	1.05	16.80	8

Notes:

1. Condition rating:
   - 1-Excellent
   - 2-Good
   - 3-Adequate
   - 4-Poor
   - 5-Very Poor
2. All ratings based on MMSD staff assessments
3. The Station Weighting Factors are based on MMSD staff assessments and range from 1 to 2.
4. The Station Weighting Factors reflect the relative criticality of each pump station in MMSD's conveyance system.

Table 4-4. Existing MMSD Force Mains

Pumping Station Force Main	Length (feet)	Diameter (inches)	Material	Year Installed	Nominal Force Main Capacity		Required Capacity 2030		Required Capacity 2060		Excess Available Capacity 2060 (mgd	Additional Required Capacity 2060 Based on 8 fps	Addition Required	
					8 fps Velocity (mgd)	Pressure (mgd)	Low (mgd)	High (mgd)	Low (mgd)	High (mgd)			High	Low
1	2,638	30	RCCP	1948	25.4		16.08	16.90	16.90	18.45	6.92			
Cross-Town FM	14,213	30	DIP	2002	25.4		16.08	16.90	16.90	18.45	6.92			
	998	20	PVC	1995	11.3		16.08	16.90	16.90	18.45		7.17		
	1,346	24	DIP	2000	16.2		16.08	16.90	16.90	18.45		2.21		
2	9,890	36	DIP	2001	36.5		27.06	29.53	29.53	33.69	2.85			
2 and 4	6,395	36	DIP	2001	36.5		29.36	31.88	31.88	36.12	0.42			
	364	36	DIP	2005	36.5		29.36	31.88	31.88	36.12	0.42			
2, 3, and 4	1,123	36	DIP	2001	36.5		30.10	32.68	32.68	36.90		0.36	2057	
3	5	8	CIP	1959	1.8		1.29	1.40	1.40	1.40	0.40			
	21	8	DIP	2000	1.8		1.29	1.40	1.40	1.40	0.40			
4	100	16	CIP	1959	7.2		3.93	4.09	4.09	4.29	2.93			
	53	16	DIP	2000	7.2		3.93	4.09	4.09	4.29	2.93			
5	28	16	DIP	1996	7.2		2.40	2.52	2.52	2.68	4.54			
	457	16	RCCP	1959	7.2		2.40	2.52	2.52	2.68	4.54			
5 and 15	1,742	24	RCCP	1959	16.2		7.47	8.54	8.54	9.52	6.72			
6	7,214	36	RCCP	1948	36.5		6.36	6.37	6.37	7.14	29.40			
7	6,996	36	RCCP	1948	36.5	27.5	22.95	29.93	29.93	36.15		8.65	2025	2042
	6,996	36	RCCP	1963	36.5	27.5	22.95	29.93	29.93	36.15		8.65	2025	2042
	1,332	48	RCCP	1963	65.0		45.90	59.86	59.86	72.30		7.35	2042	
	323	48	DIP	2005	65.0		45.90	59.86	59.86	72.30		7.35	2042	
8	13,174	42	RCCP	1964	49.7		24.27	26.17	26.17	28.02	21.71			
	194	36	RCCP	1964	36.5		24.27	26.17	26.17	28.02	8.52			
	334	42	DIP	2005	49.7		24.27	26.17	26.17	28.02	21.71			
9	4,329	20	DIP	1987	11.3		4.24	4.93	4.93	6.39	4.89			
	40	14	DIP	1987	5.5		4.24	4.93	4.93	6.39		0.86		
	2,197	10			2.8		4.24	4.93	4.93	6.39		3.57		
10	11,109	36	RCCP	1964	36.5		29.25	35.26	35.26	38.74		2.20	2040	
11	4,173	36	RCCP	1965	36.5		32.51	39.17	39.17	44.82		8.28	2025	2050
12	4,786	36	RCCP	1968	36.5		23.24	28.93	28.93	32.3	4.24			
13	1,927	36	RCCP	1969	36.5		21.56	25.77	25.77	29.44	7.10			
14	3,108	30	RCCP	1971	25.4		14.58	16.18	16.18	20.16	5.21			
	1,358	30	RCCP	1971	25.4		15.30	16.90	16.90	20.84	4.53			
15-8	1,360	24	DIP	1974	16.2		5.63	6.65	6.65	7.57	8.67			
	1,071	24	DIP	1974	16.2		5.63	6.65	6.65	7.57	8.67			
	4,837	20	RCCP	1959	11.3		5.63	6.65	6.65	7.57	3.71			
	18	24	RCCP	1959	16.2		5.63	6.65	6.65	7.57				
16	7,214	36	DIP	1979	36.5		8.53	10.24	10.24	10.55	25.99			
	2,965	30	DIP	1980	25.4		8.53	10.24	10.24	10.55	14.82			
17	13,357	16	DIP	1995	7.2		7.82	11.25	11.25	13.57		6.35	2015	2026
	3,071	20	DIP	1995	11.3		7.82	11.25	11.25	13.57		2.29	2025	2052
Total (feet)	142,947													

					Nominal Force Main Capacity		Required Capacity 2030		Required Capacity 2060				Addition Required	
Pumping Station Force Main	**Length (feet)**	**Diameter (inches)**	**Material**	**Year Installed**	**8 fps Velocity (mgd)**	**Pressure (mgd)**	**Low (mgd)**	**High (mgd)**	**Low (mgd)**	**High (mgd)**	**Excess Available Capacity 2060 (mgd**	**Additional Required Capacity 2060 Based on 8 fps**	**High**	**Low**
Total (Miles)	27.1													
Notes:														
1.	MMSD has 50-million gallons of treated storage at the NSWTP that is used if the effluent flows exceed 78.6 mgd. If flows extend for a period of time, discharge to Nine Springs Creek will occur via an overflow structure.													
2.	Estimates for velocities in Force Mains 2, 3, and 4 are based on all pumping stations pumping at firm capacity at the same time.													
3.	Material abbreviations: CIP – Cast Iron Pipe DIP – Ductile Iron Pipe PVC – Polyvinyl Chloride Pipe RCCP – Reinforced Concrete Cylinder Pipe													

Table 4-5. Existing Pumping Station Flows in MGD (1996-2006)

PS	Average	1996	1997	1998	1999	2000	2001	2002	2003	2004	2005	2006
1	5.91	7.01	6.62	6.60	6.84	6.39	6.49	5.82	5.09	3.99	5.01	5.21
2	5.29	3.97	3.51	4.27	4.15	4.45	4.20	5.07	5.00	5.83	9.05	8.68
3	0.36	0.34	0.34	0.34	0.35	0.31	0.32	0.48	0.43	0.44	0.32	0.32
4	1.08	1.07	1.03	1.26	1.22	0.92	1.10	1.07	1.05	1.14	0.97	1.02
5	0.64	0.90	0.60	0.51	0.56	0.70	0.66	0.58	0.60	0.71	0.65	0.56
6	6.45	8.25	8.19	8.23	8.32	7.73	7.54	7.12	6.38	5.07	2.51	1.63
7	18.46	19.50	18.74	19.36	20.05	20.15	20.15	19.15	18.01	18.58	14.52	14.80
8	8.16	8.10	8.02	8.21	8.33	8.77	8.50	8.05	7.65	7.86	7.52	7.52
9	0.81	0.82	0.79	0.80	0.81	0.81	0.82	0.79	0.77	0.83	0.80	0.85
10	9.44	9.59	9.79	9.95	10.53	10.76	9.84	9.50	8.26	9.37	8.21	8.00
11	7.70	7.12	7.32	6.67	7.44	7.49	7.87	7.88	7.89	8.49	8.24	8.26
12	4.67	5.03	4.71	4.16	4.70	4.31	4.48	4.51	4.56	4.86	4.98	5.05
13	5.13	4.94	5.18	4.98	5.29	5.06	4.84	4.84	4.82	5.30	5.19	5.94
14	3.33	3.05	2.84	2.90	3.23	3.33	3.49	3.44	3.66	3.66	3.40	3.62
15	1.23	0.99	1.12	1.23	1.29	1.30	1.33	1.26	1.26	1.29	1.19	1.24
16	1.67	2.25	2.05	1.29	1.45	1.51	1.50	1.55	1.59	1.61	1.73	1.81
17	0.69	0.63	0.61	0.66	0.66	0.68	0.71	0.72	0.72	0.77	0.73	0.73
Collection System												
Total Pumped Flow	78.79	75.46	81.43	81.42	85.20	84.66	83.84	81.84	77.74	79.81	67.50	67.73
Effluent Pumping												
Badfish Creek		41.70	40.81	42.06	41.23	41.50	40.40	38.38	36.85	40.29	37.48	38.63
Badger Mill Pumping		0.00	0.00	0.81	2.20	2.38	3.00	2.99	2.99	2.78	3.11	3.09
Total Effluent Pumping		41.70	40.81	42.87	43.43	43.88	43.40	41.37	39.84	43.07	40.59	41.72

Table 4-6. Gravity Conveyance Facilities

Pumping Station Service Area	Total Gravity Sewer (miles)	Interceptors Reaching Capacity (miles)			
		2010	2010-2020	2020-2030	2030-2060
1	2.64	0.00	0.00	0.00	0.00
2	2.46	0.41	0.00	0.00	0.00
3	1.02	0.72	0.00	0.00	0.00
4	1.58	0.00	0.00	0.00	0.00
5	3.00	0.00	0.00	0.00	0.00
6	1.91	0.00	0.00	0.00	0.00
7	19.41	2.07	3.32	3.95	2.62
8	13.91	1.68	0.71	0.83	0.75
9	0.64	0.00	0.00	0.00	0.63
10	5.71	1.74	2.08	0.00	0.00
11	10.16	0.00	1.48	3.81	0.75
12	7.87	0.00	0.67	0.00	0.00
13	3.05	0.00	0.03	0.33	0.86
14	15.85	0.00	0.88	2.61	7.96
15	1.71	0.00	0.00	0.04	0.44
16	1.61	0.00	0.00	0.53	0.19
17	1.48	0.00	0.00	0.00	0.00
Total	94.02	6.62	9.17	12.09	14.20
		7.0%	9.8%	12.9%	15.1%

Table 4-7. Televising History

Interceptor	Miles Televised (year)								
	2000	2001	2002	2003	2004	2005	2006	2007	2008
SWI-Northerly Leg	1.07								
SWI-Southerly Leg	1.01								
SWI-Main Leg	3.82								
NEI-PS 14 to Airport	1.72								
NEI-PS 10 to SEI	2.46								
FEI and Cottage Grove Extension		3.40							
SEI		1.78							
EI-PS 1 to PS 6		1.48							
EI-PS 6 to PS 7		2.16							
Rimrock Interceptor			0.72						
EI/East Monona Extension			0.41						
SI and Baird Street Extension			1.40						
WI Spring Street Extension			0.78						
NEI-Waunakee Extension			4.20						
NSVI				6.50					
NSVI-Waubesa Extension				1.80					
NSVI-Hwy 14 Extension				1.77					
SEI				2.24					
West Interceptor-Gammon Extension				2.84					
NSVI-Mineral Point Extension					6.23				
NSVI-Midtown Extension					1.57				
NEI-Deforest Extension					9.16				
NEI-Highway 19 Extension					1.19				
SEI-Blooming Grove Extension					2.73				
SEI-Sigglekow Extension					1.01				
SEI-McFarland Relief					1.08				
SI/Baird Street Extension						0.30			
SI/Lakeside Extension						1.10			
FEI/Door Creek Extension						3.37			
NEI/ P.S. 13 to P.S. 10						4.14			

Interceptor	Miles Televised (year)								
	2000	2001	2002	2003	2004	2005	2006	2007	2008
WI/PS 15 to PS 5						0.64			
WI/Randall Relief						11.45			
NEI PS 14 to PS 13							2.98		
EI							4.62		
FEI/Cottage Grove Extension							3.40		
WI/Spring Street Relief							0.87		
WI/Midvale Relief							0.50		
NEI PS 14 to PS 13								2.98	
EI								4.62	
FEI/Cottage Grove Extension								3.40	
WI/Spring Street Relief								0.87	
WI/Midvale Relief								0.50	
Northeast Interceptor/Waunakee Extension									4.93
Southwest Interceptor									3.39
West Interceptor									4.68

Table 4-8. Pumping stations Tributary to MMSD (2006)

Owner	Number Maintained by Owner	Number Maintained by MMSD	Tributary Sewer Length (miles)
Cities			
Fitchburg			53
Madison		29	752
Middleton	8		77
Monona	7		38
Verona		1	50
Villages			
Cottage Grove	4		32
Dane	1		11
DeForest	1		39
Maple Bluff		3	7
McFarland	4		30
Shorewood Hills	1		13
Waunakee	2		60
Townships			
Blooming Grove			NDA
Burke			NDA
Madison		3	NDA
Verona			NDA
Others			
UW Campus			
Pumping Stations	6		
Grinder Pumps	4		
UW Arboretum	1		
Dane County Landfill	1		
Dane County Vilas Zoo	1		
Dane County Lake Farm Park		1	
Districts			
Blooming Grove SD 2	1		3
Blooming Grove SD 10			0
Burke Utility District 1			1
Burke Utility District 2			NDA
Burke Utility District 6			NDA
Token Creek SD	1		4
Town of Dunn SD 1		4	5
Town of Dunn SD 3		3	7

Owner	Number Maintained by Owner	Number Maintained by MMSD	Tributary Sewer Length (miles)
Town of Dunn SD 4			6
Kegonsa SD			20
Pumping Stations	5		
Grinder Pumps	354		
Middleton SD 5			1
Pleasant Springs SD 1			33
Pumping Stations	9		
Grinder Pumps	55		
Verona Utility District 1			3
Vienna Utility District 1	1		3
Vienna Utility District 2	1		3
Westport Utility District 1			60
Pumping Stations	10		
Grinder Pumps	1		
Westport Utility District 2			NDA
Westport Utility District 3			NDA
Westport Utility District 4			NDA
Cherokee Golf and Tennis			NDA
Windsor SD 1	3		17
Windsor SD 3			NDA
Illinois Seed Foundation			NDA
Hidden Springs SD			NDA
Lake Windsor SD			2
Morrisonville SD	1		2
Oak Springs SD			2
Total	**483**	**44**	**NDA**

Note: NDA means no data available.

Table 4-9. Contributing Existing Flows (2000-2008)

	Average Daily Flows (mgd)								
	Year								
	2000	2001	2002	2003	2004	2005	2006	2007	2008
Cities									
Fitchburg	1.44	1.48	1.56	1.60	1.90	1.68	1.95	1.88	1.96
Madison	29.65	28.82	27.02	25.77	28.40	26.45	26.90	28.81	31.65
Middleton	1.91	1.99	1.97	1.93	1.82	1.70	1.67	1.76	1.94
Monona	0.91	0.93	1.00	0.88	0.88	0.90	0.85	0.95	1.04
Verona	0.67	0.71	0.72	0.73	0.76	0.73	0.74	0.82	0.91
Villages									
Cottage Grove	0.34	0.34	0.34	0.34	0.56	0.58	0.63	0.68	0.76
Dane	0.05	0.05	0.05	0.05	0.05	0.06	0.06	0.06	0.06
DeForest	0.62	0.69	0.72	0.74	0.70	0.62	0.64	0.77	1.05
Maple Bluff	0.17	0.15	0.14	0.14	0.17	0.16	0.17	0.19	0.26
McFarland	0.56	0.55	0.55	0.50	0.56	0.55	0.62	0.64	0.70
Shorewood Hills	0.17	0.18	0.18	0.19	0.19	0.18	0.20	0.19	0.19
Waunakee	1.16	1.34	1.34	1.23	1.26	1.24	1.37	1.53	1.72
Townships and Districts									
Town of Blooming Grove	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01
Town of Burke	0.01	0.01	0.01	0.01	0.01	0.01	0.01		
Town of Madison	0.90	0.90	0.94	0.95	0.94	0.88	0.87	0.86	0.93
Town of Verona	0.01	0.01	0.01	0.01	0.01	0.01		0.01	
Blooming Grove SD 2	0.21	0.15	0.13	0.09	0.16	0.17	0.11	0.14	0.23
Blooming Grove SD 10	0.02	0.02	0.02	0.02	0.02	0.02	0.02	0.02	
Burke Utility District 1	0.01	0.01	0.15	0.01	0.02	0.02	0.01	0.02	0.00
Burke Utility District 2	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01
Burke Utility District 6	0.01	0.01	0.01	0.01	0.01	0.01	0.00	0.01	
Token Creek SD	0.04	0.05	0.05	0.05	0.06	0.05	0.05	0.07	0.12
Town of Dunn SD 1	0.11	0.11	0.11	0.11	0.17	0.15	0.14	0.17	0.25
Town of Dunn SD 3	0.08	0.08	0.07	0.08	0.08	0.07	0.07	0.07	0.07
Town of Dunn SD 4	0.03	0.03	0.03	0.02	0.02	0.03	0.02	0.07	0.04

	Average Daily Flows (mgd)								
	Year								
	2000	**2001**	**2002**	**2003**	**2004**	**2005**	**2006**	**2007**	**2008**
Kegonsa SD	0.03	0.13	0.13	0.13	0.15	0.18	0.15	0.03	0.17
Middleton SD 5	0.02	0.02	0.02	0.02	0.02	0.02	0.02	0.02	0.02
Pleasant Springs SD 1	0.09	0.10	0.10	0.10	0.10	0.10	0.09	0.08	0.06
Verona Utility District 1	0.02	0.02	0.02	0.02	0.02	0.02	0.02	0.05	0.02
Vienna Utility District 1	0.05	0.04	0.04	0.05	0.05	0.02	0.04	0.06	0.05
Vienna Utility District 2	0.01	0.01	0.01	0.01	0.01	0.02	0.03	0.03	0.04
Westport Utility District 1	0.09	0.01	0.17	0.18	0.18	0.17	0.19	0.2	0.16
Westport Utility District 2	0.26	0.11	0.30	0.34	0.40	0.32	0.37	0.38	0.43
Westport Utility District 3	0.01	0.27	0.02	0.02	0.01	0.01	0.01	0.02	0.02
Westport Utility District 4	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01
Cherokee Golf and Tennis	0.01	0.01	0.01	0.01	0.01	0.01	0.00	0.01	0.01
Windsor SD 1	0.19	0.20	0.21	0.21	0.19	0.20	0.19	0.225	0.28
Windsor SD 3	0.01	0.01	0.01	0.01	0.01	0.01	0.00	0.01	0.01
Illinois Seed Foundation	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01
Hidden Springs SD	0.01	0.01	0.01	0.01	0.01	0.01	0.00	0.01	0.04
Lake Windsor SD	0.03	0.04	0.04	0.03	0.04	0.03	0.03	0.04	0.06
Morrisonville SD	0.05	0.08	0.05	0.04	0.06	0.05	0.05	0.07	0.08
Oak Springs SD	0.04	0.04	0.04	0.03	0.03	0.03	0.03	0.04	0.04
Infiltration into District Interceptors	1.99	1.99	1.99	1.94	1.94	1.94	1.87	1.87	1.84
Total Flow at Nine Springs	**42.01**	**41.73**	**40.32**	**38.64**	**42.01**	**39.44**	**40.21**	**42.89**	**47.25**

4.3. Current Conveyance Deficiencies for Centralized Treatment

Without the construction of satellite plants to potentially divert flow away from existing pumping stations, the pumping station system capacity needs at different planning years are shown in Table 4-10.

Table 4-10. Pumping Station Capacity Expansion Needs

Planning Years	Description
Year 2000-2010	PS 7 Firm Capacity (High Flow Projections) PS 11 Firm Capacity (High Flow Projections) PS 12 Firm Capacity PS 17 Firm and Maximum Capacity (High Flow Projections)
Year 2010-2020	PS 7 Firm Capacity PS 7 Maximum Capacity (High Flow Projections) PS 11 Firm Capacity PS 11 Maximum Capacity (High Flow Projections) PS 12 Firm Capacity PS 12 Maximum Capacity (High Flow Projections) PS 13 Firm Capacity PS 13 Maximum Capacity (High Flow Projections) PS 17 Firm and Maximum Capacity
Year 2020-2030	PS 7 Firm and Maximum Capacity PS 11 Firm and Maximum Capacity PS 12 Firm Capacity PS 12 Maximum Capacity (High Flow Projections) PS 13 Firm and Maximum Capacity PS 14 Firm Capacity (High Flow Projections) PS 15 Firm and Maximum Capacity (High Flow Projections) PS 17 Firm and Maximum Capacity
Year 2030-2060	PS 4 Firm and Maximum Capacity (High Flow Projections) PS 7 Firm and Maximum Capacity PS 9 Firm and Maximum Capacity PS 11 Firm and Maximum Capacity PS 12 Firm and Maximum Capacity PS 13 Firm and Maximum Capacity PS 14 Firm and Maximum Capacity PS 15 Firm Capacity PS 17 Firm and Maximum Capacity

A review of the capacity of the existing MMSD force mains (FM) shows that the following force mains have future flow velocities at peak hourly flow in excess of the nominal maximum velocity target of 8 feet per second and will likely need to be expanded in the time span shown:

- Cross-Town FM (CTFM) (20-inch PVC and 24-inch DIP Only - 2010)
- PSs 2, 3 and 4 FM (2057 - High Flow Projections Only)
- PS 7 FM (2025-2042)
- PS 10 FM (2040 - High Flow Projections Only)

- PS 11 FM (2025-2049)
- PS 17 FM – 16 inch portion (2015-2026)
- PS 17 FM – 20 inch portion (2030-2060)

Table 4-11 presents the approximate percentage of the MMSD total interceptor length that will reach capacity and require expansion during the 50-year planning period. The detailed interceptor sewer capacity needs at different planning periods are summarized in TM3 – Conveyance Facilities Analysis (CFA).

Table 4-11. Interceptor Capacity Expansion Needs

Planning Years	Percentage of the Total Interceptor Length
Year 2010-2020	11.2%
Year 2020-2030	11.5%
Year 2030-2060	15.4%

4.4. Existing Facility Condition Assessment

Figure 4-2 shows the age distribution of the MMSD force mains. The general condition of the force mains is good since they tend to operate in a full pipe condition. In early 2009 the PS 6 force main was inspected as a part of an emergency repair and found to be in very good condition despite its being placed in service in 1950. The assessment of condition is based on MMSD inspections of force mains when they are not in service. The percentage of the length of force mains reaching an age of 75 years during the master planning period is as follows:

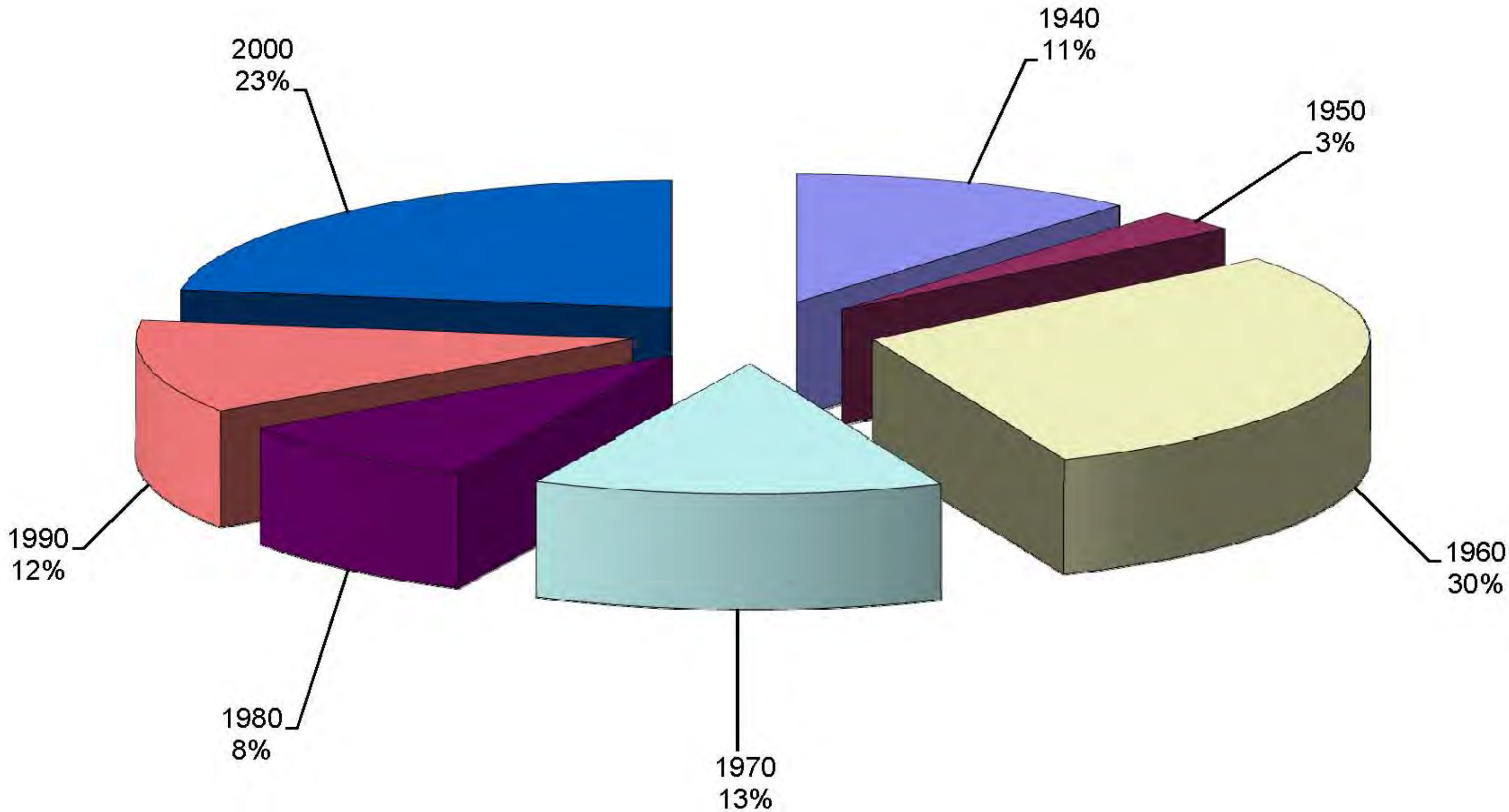
- By 2015 – 11 percent
- 2016 to 2025 – 3 percent
- 2026 to 2035 – 30 percent
- 2036 to 2045 – 13 percent
- 2046 to 2055 – 8 percent

The force mains for PS 7 and PS 11 may need relief or replacement because of capacity considerations or as a result of limited pressure capabilities by 2020. These replacements will not be required if satellite plants can reduce the flows to PS 7 and PS 11, or, if PS 18 is constructed which would reduce the capacity concerns for the PS 7 force main. PS 18 is a planned future raw sewerage pump station in the City of Monona that would operate in parallel with PS 7 and include a force main to the NSWTP.

The general condition (2008) of MMSD's interceptors is very good based on the relatively small list of interceptor segments identified as needing repair in TM 3. MMSD has already repaired most of these sewers, or has included their repair in the capital budget. MMSD is in the process of modifying its interceptor inspection program to target higher priority interceptors for more frequent review.

Figure 4-2. Madison Metropolitan Sewerage District Force Mains

(Percentage of the Total Length of MMSD Force Mains Constructed in the Decade Beginning with the Year Shown)



The current conveyance system capacity utilization rates and future capacity needs at different planning years determined in this chapter were used for planning alternatives development in the later phase of the master planning.

Budgeted Capital Improvement Projects

MMSD had prepared a list of capital improvement projects that are scheduled for implementation between 2009 and 2018 to address foreseeable system capacity needs and to improve existing facility conditions. MMSD's capital improvement plan is attached as Appendix K. A summary of the budgeted conveyance system capital improvement projects is presented in Table 4-12.

Table 4-12. Summary of Conveyance System Capital Improvement Projects (2009-2018)

Project	Year of Implementation
West Interceptor Extension Replacement in Middleton	2009
NEI - Truax Extension Rehabilitation at the Airport	2009
Far East Int - Door Creek Extension at Gaston Road	2009
PS 7 Back-up Power at Bridge Road in Monona	2009
West Interceptor - Upstream of PS 5 near Lake Mendota	2009-2011
West Interceptor Replacement at Old University Avenue	2009-2011
NEI - Relief Upstream of PS 10 near Hwy 51	2009-2011
South Interceptor - Baird Street Replacement	2009-2011
Far East Int - Cottage Grove Extension Lining	2009-2011
Pumping Stations 6 & 8 Rehabilitation	2009-2011
Lower Badger Mill Creek Interceptor Project north from Verona	2009-2015
East Monona Interceptor at Fair Oaks u/s of Starkweather Creek	2010-2012
NSVI - Morse Pond Extension near UW Golf Course	2010-2012
P.S. No. 18 Force Main Construction from Monona to NSWTP	2010-2013
P.S. No. 18 Construction on Broadway in Monona	2010-2014
NEI - Far East Int. to Southeast Int. Junction	2012-2014
PS7 - Improvements (in conjunction with PS18 construction)	2013-2015
Pumping Stations 11 & 12 Rehabilitation	2013-2016
Pumping Station 17 Upgrade (Completed in conjunction with LBMCI or SRTP)	2013-2014
Pumping Station 15 Rehabilitation	2014-2018
Pumping Stations 13 & 14 Rehabilitation	2015-2018
Southwest Interceptor - Haywood Extension Rehab or Replacement	2016-2018
Pumping Stations 3, 4, & 9 Revisions	2017-2018

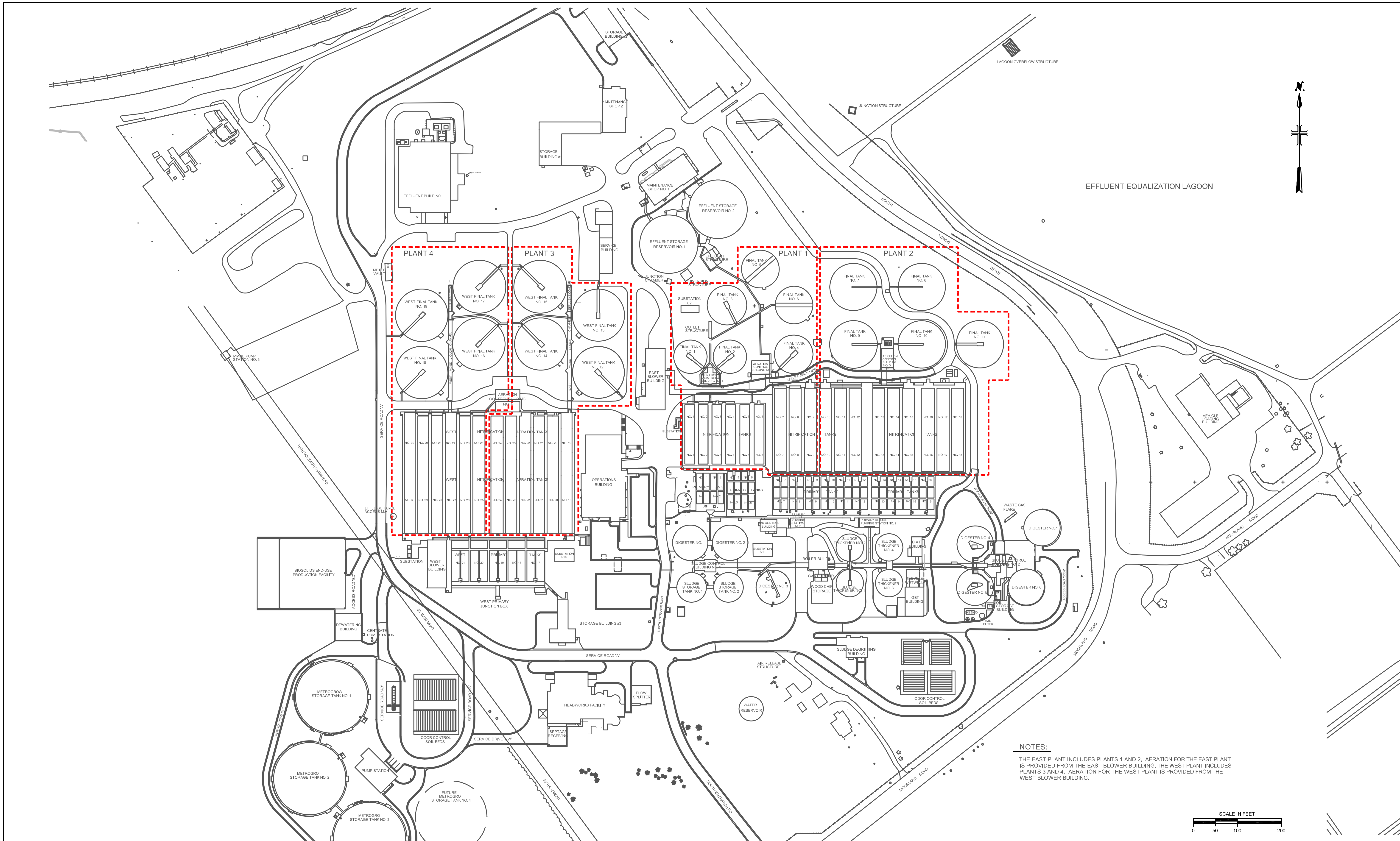
5. TREATMENT FACILITIES

5.1. Background

MMSD owns and operates the NSWTP, which treats wastewater collected from the greater Madison metropolitan area. Figure 5-1 shows the general layout of the treatment and support facilities at the plant. Figure 5-2 shows the MMSD property in the vicinity of the NSWTP.

5.2. Nine Springs Wastewater Treatment Plant

The NSWTP receives wastewater being pumped to the plant and pumps treated effluent to Badfish Creek in the lower Rock River basin and Badger Mill Creek in the Sugar River basin via two effluent force mains. Both discharge outfalls are regulated by a WPDES Permit issued by the WDNR in 2009, which will expire in 2014. The major discharge limits are summarized in Table 5-1. The biosolids are land applied to agricultural land after anaerobic digestion. The plant has a rated average flow capacity of 57 mgd and a peak flow capacity of 140 mgd. Detailed evaluations of the existing treatment processes at the NSWTP are provided in Appendix A, TM1 – Review of Existing Treatment Facilities.



NOTES:

THE EAST PLANT INCLUDES PLANTS 1 AND 2. AERATION FOR THE EAST PLANT IS PROVIDED FROM THE EAST BLOWER BUILDING. THE WEST PLANT INCLUDES PLANTS 3 AND 4. AERATION FOR THE WEST PLANT IS PROVIDED FROM THE WEST BLOWER BUILDING.

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Figure 5-1
NSWTP SITE PLAN - 2009

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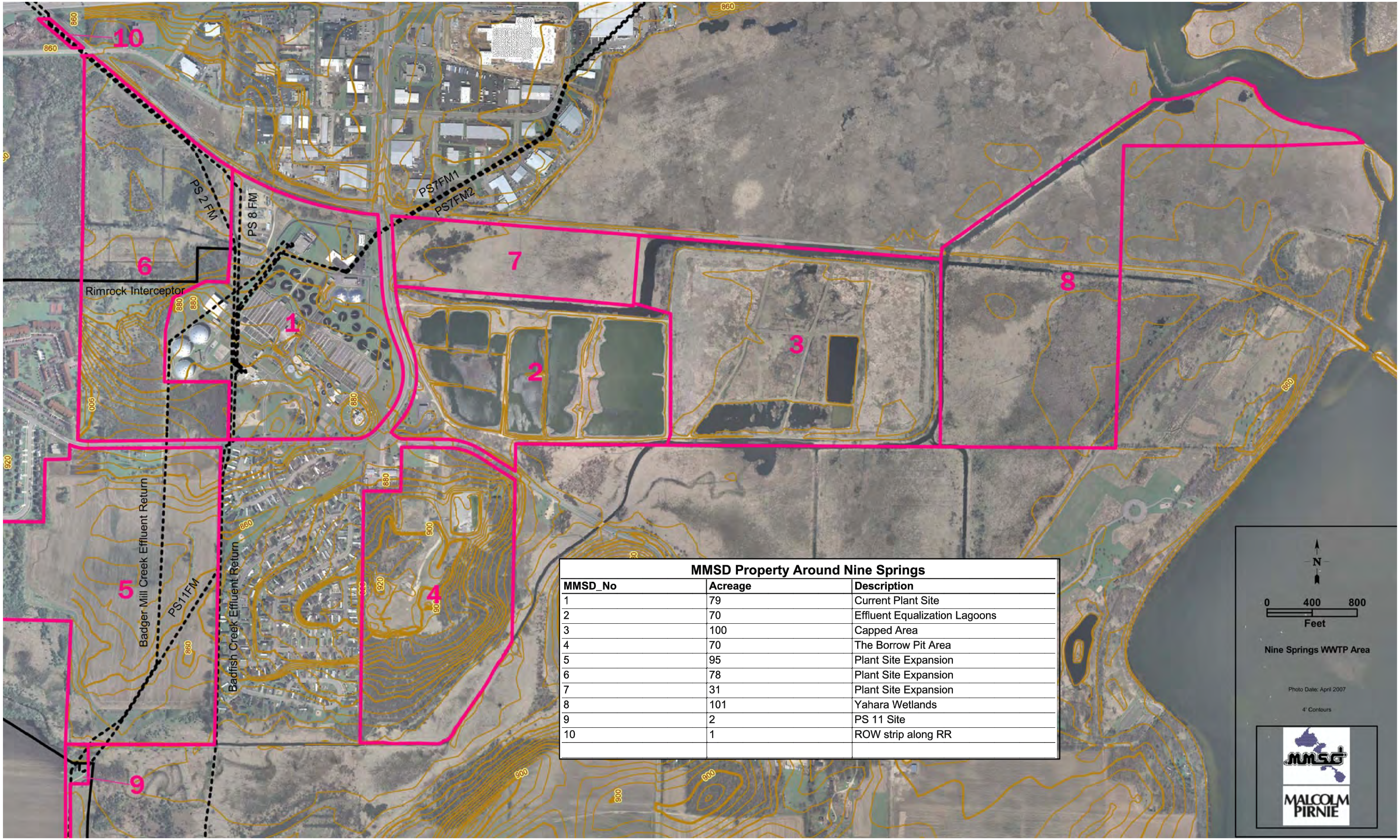

MMSD Property Around Nine Springs

MMSD_No	Acreage	Description
1	79	Current Plant Site
2	70	Effluent Equalization Lagoons
3	100	Capped Area
4	70	The Borrow Pit Area
5	95	Plant Site Expansion
6	78	Plant Site Expansion
7	31	Plant Site Expansion
8	101	Yahara Wetlands
9	2	PS 11 Site
10	1	ROW strip along RR

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Figure 5-2
MMSD PROPERTY MAP

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Table 5-1. Summary of Current WPDES Permit for the Nine Springs WWTP (April 1, 2009 - March 31, 2014)

Effluent Characteristics	Units	Monthly Average	Weekly Average	Daily Minimum	Daily Maximum	Geometric Mean
Badfish Creek Outfall						
BOD_5, Total	mg/L	19	20			
BOD_5, Total*	lb/day[1]	7,923	8,340			
TSS	mg/L	20	23			
TSS*	lb/day[1]	8,340	9,591			
DO	mg/L			5.0		
pH				6.0	9.0	
Phosphorus, Total	mg/L	1.5				
Fecal Coliform (April 15 – October 15)	#/100 ML					400
NH_4-N (May – September)	mg/L	1.8	4.4		17	
NH_4-N (October – April)	mg/L	4.1	10		17	
Badger Mill Creek Outfall						
BOD_5, Total (November – April)	mg/L		16			
BOD_5, Total (May – October)	mg/L		7.0			
TSS (November – April)	mg/L	16				
TSS (May – October)	mg/L	10				
DO	mg/L			5.0		
pH				6.0	9.0	
Phosphorus, Total	mg/L	1.5				
Fecal Coliform (April 15 – October 15)	#/100 ML					400
NH_4-N (October – April)	mg/L	3.8	8.7		11	
NH_4-N (May – September)	mg/L	1.1	2.6		11	
Chloride	mg/L		400			

** All loadings are calculated based on the nominal design average flow of 50 mgd.*

5.2.1. Liquid Treatment Facilities

The liquid treatment facilities at the NSWTP include preliminary treatment, primary clarification, nitrifying activated sludge treatment incorporating biological phosphorus removal, ultraviolet (UV) disinfection, excess flow storage and effluent pumping. Figure 5-3 shows the schematic of the liquid treatment process at the NSWTP. The liquid treatment facilities, including the primary clarifiers, aeration basins and secondary clarifiers, are divided into two complexes; the East Complex and the West Complex. The East Complex includes Plant 1 and Plant 2, and the West Complex includes Plant 3 and Plant 4. The treatment facilities included in each plant are shown in Table 5-2.

Table 5-2. NSWTP Sub-Plant Descriptions

Treatment Facility	East Complex		West Complex	
	Plant 1	Plant 2	Plant 3	Plant 4
Primary Clarifier	No. 1-2 No. 5-6 No. 7-16		No. 17-21	
Aeration Basin	No. 1-6	No. 7-9 No. 10-18	No. 19-24	No. 25-30
Secondary Clarifier	No. 1-6	No. 7-11	No. 12-15	No. 16-19

The detailed descriptions of liquid treatment facilities are presented as follows:

- **Headworks**
  The existing headworks include influent flow measurement, fine screening, grit removal by vortex grit basins, and a weir flow splitting structure distributing flows to the East and West Complexes. It also includes screenings and grit processing equipment, the plant water system, and the septage receiving facility. Wastewater enters the headworks facility via influent force mains. Flow is measured by Venturi flowmeters on each force main before proceeding through fine screens. After screening, the flow continues to vortex grit basins. Screenings are conveyed by a sluice trough to screenings processing units. Grit from the vortex grit basins is pumped to grit processing units. Processed screenings and grit are conveyed to roll-off containers by a reversible belt conveyor.

  Flow exiting the grit basins enters the flow splitting structure and is distributed to the East and West Complexes through weir troughs with manual stop plates.

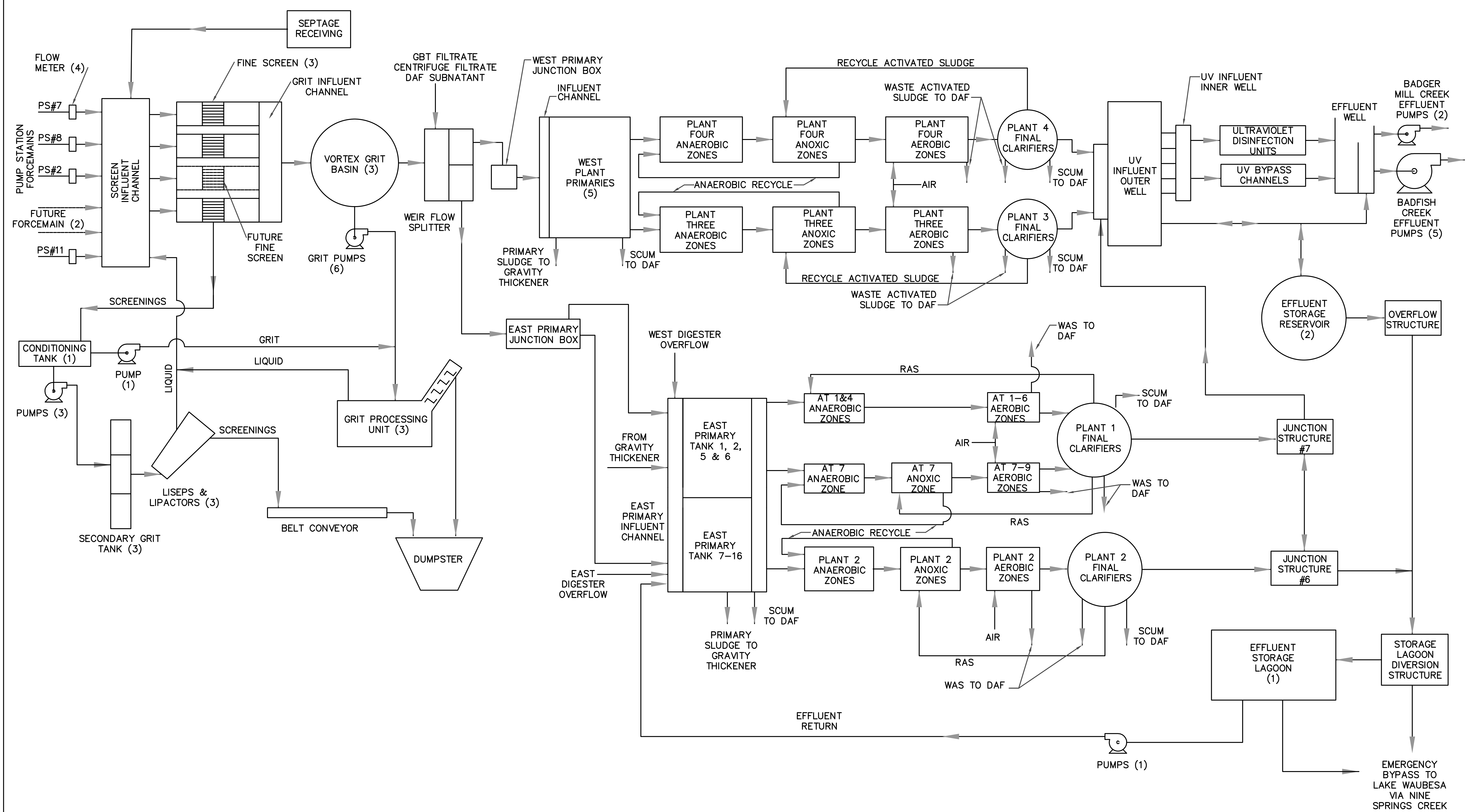

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Figure 5-3
LIQUID PROCESS SCHEMATIC

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- **Flow Splitter**
  The existing flow splitter was constructed during the plant's Tenth Addition. The structure splits screened and degritted plant flow between the East and West Complexes using fixed weir flow splitting structures.

- **Primary Settling Facilities**
  There are 14 primary clarifiers in the East Complex and 5 primary clarifiers in the West Complex. All clarifiers are rectangular units with chain and flight sludge removal mechanisms. Settled primary sludge is pumped to gravity thickeners for thickening before being digested.

- **Aeration Basins**
  Biological treatment of the primary effluent occurs in the aeration basins. There are 18 aeration basins in the East Complex and 12 in the West Complex. The aeration basins are configured such that each group of three aeration basins functions as one "folded" treatment unit. Thus, there are 6 treatment units in the East Complex and 4 treatment units in the West Complex. Aeration tank effluent proceeds into the secondary clarifiers for settling. The existing secondary treatment facility is an enhanced biological phosphorus removal (EBPR) system with two process configurations being utilized – The University of Cape Town (UCT) Variation process, which is utilized for the majority of the plant, and the anaerobic/aerobic (A/O) process.

  The UCT process consists of anaerobic, anoxic and aerobic zones. Influent wastewater enters the anaerobic zone and is combined with recycle from the anoxic zone. Mixed liquor then flows into the anoxic zone that is created by pumping return activated sludge (RAS) from the final clarifiers. The mixed liquor then proceeds into the aerobic zone for further treatment.

  The A/O process is utilized in 2 of the 3 treatment units of Plant 1. In the A/O process, the anoxic zone is eliminated and RAS is combined with the influent wastewater in the anaerobic zone. Following the anaerobic zone, the mixed liquor flows to the aerobic zone.

- **Secondary Clarification Facilities**
  Effluent from the aeration tanks flows to secondary clarifiers for settling. There are 11 secondary clarifiers in the East Complex and 8 in the West Complex. The effluent of the secondary clarifiers flows to UV disinfection facilities before being discharged. The RAS is pumped to aeration tanks while waste activated sludge (WAS) and scum are pumped to dissolved air floatation (DAF) thickeners for thickening before being digested.

- **UV Disinfection Facilities**
  UV disinfection facilities disinfect the effluent from the secondary clarifiers. The existing UV disinfection system is an open channel, low pressure mercury vapor type. There are a total of 7 channels, 5 of which are installed with UV disinfection equipment, one is reserved for future equipment, and the seventh channel is used as a by-pass channel when the UV system is out of service. Each of the five channels has two UV banks in series. Normally two to four channels are in service with one bank of lamps operational. During peak flow rates, additional UV channels are added to meet the flow demands. Channels are brought online and taken offline by automatically controlled motorized gates installed on the inlet of each channel.

- **Plant Effluent Pumping Facilities**
  The existing effluent pumping facilities were constructed during the plant's Seventh Addition. The plant effluent is pumped to Badfish Creek through a 54" force main of 5 miles and to Badger Mill Creek through a 20" force main of 10 miles. The Badfish Creek effluent pumps consist of five horizontal split case centrifugal pumps, each with an 800 hp, 880 rpm motor. Three pumps are outfitted with 25.94" diameter impellers and two are equipped with impellers trimmed to 24" to save energy when lower flow rates are practicable. The Badger Mill Creek effluent pumps include two centrifugal pumps, each with a 200 hp, variable speed motor. Each pump has a capacity of 2,000 gpm at a total dynamic head of 190 feet.

- **Effluent Storage Facilities**
  The plant has two effluent storage tanks and an effluent storage lagoon for plant effluent storage. The disinfected effluent beyond effluent pumping capacity and up to an estimated flow rate of 115 mgd overflows to effluent storage reservoirs. The effluent storage reservoirs, in turn, overflow to the effluent storage lagoon when their maximum storage capacities are reached. Flows in excess of 115 mgd (estimated) receive secondary treatment and are diverted to the effluent equalization facilities. This estimated flow rate is based on a flow split at the flow splitter of 45 percent to the east side of the plant and 55 percent to the west side of the plant. At a total flow of 115 mgd, the East Complex flow would be 52 mgd which is the flow rate from the east side final clarifiers at which bypassing of secondary effluent was observed previously. The effluent equalization facilities (storage lagoons) have a nominal volume of 50 MG. When this volume is exceeded, an overflow structure diverts additional flows to the ditch on the north side of the lagoons. Flow in the ditch discharges to Nine Springs Creek, which in turn discharges to Lake

Waubesa. Discharges to the effluent equalization facilities are pumped back to the secondary process when the plant peak flow subsides. Since the effluent storage lagoons are open to the atmosphere, effluent storage volume is reduced by 1.3 million gallons for each inch of precipitation.

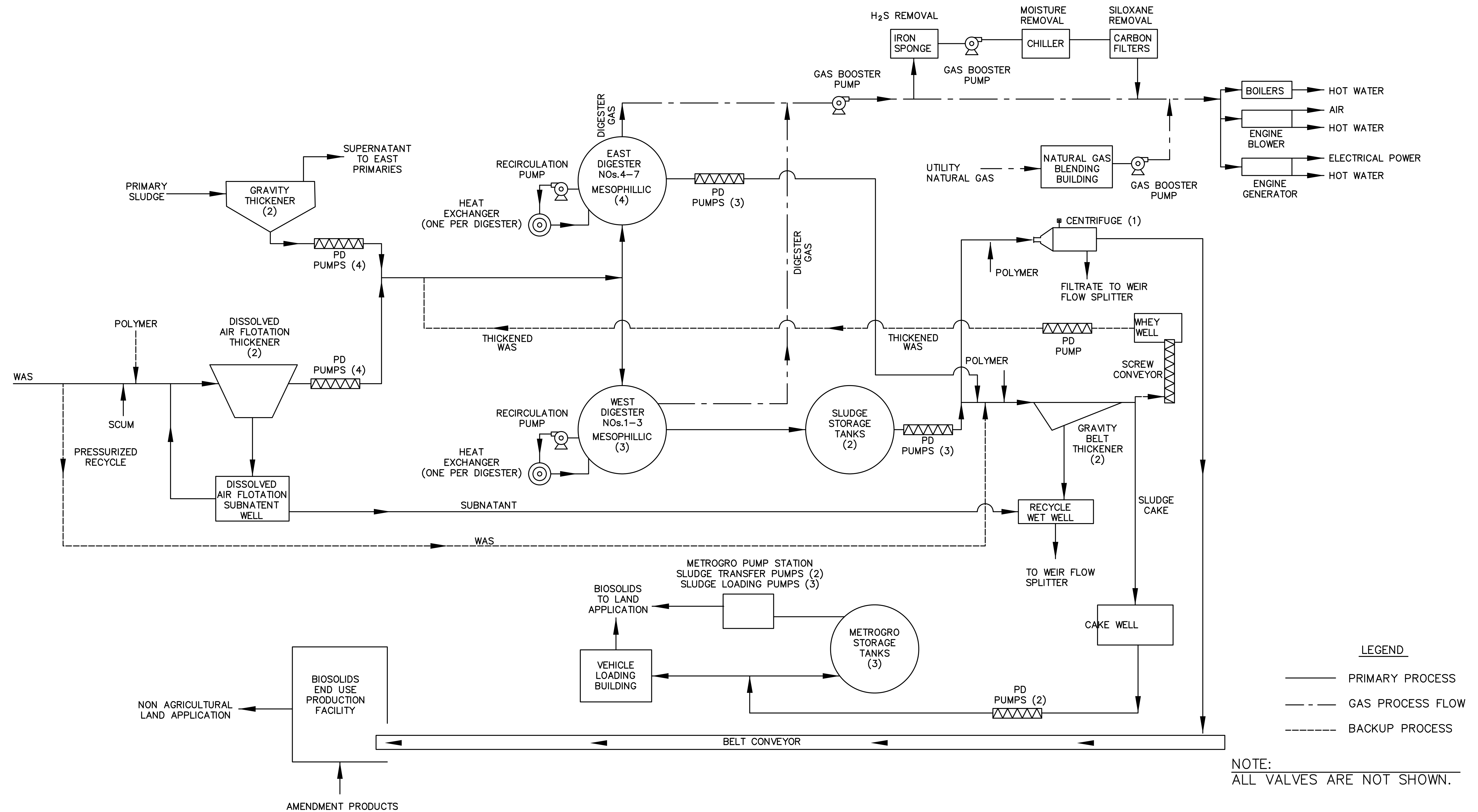
5.2.2. Biosolids Disposal Facilities

The biosolids production facilities at the NSWTP include primary sludge thickening by gravity thickeners, waste activated sludge thickening by DAF thickeners, anaerobic digestion, digested sludge thickening by gravity belt thickeners, digested sludge dewatering by centrifuge, and onsite biosolids storage. Figure 5-4 shows the schematic of the biosolids production processes at the NSWTP.

- **Gravity Thickeners**
  Primary sludge is pumped into the gravity thickeners for thickening. The thickened sludge pumps operate continuously, typically one per thickener, to convey the thickened sludge to the anaerobic digester feed header, where it combines with thickened WAS. The combined stream is fed to the digesters. Operators manually adjust pump speed to maintain appropriate sludge blanket levels in the thickeners and minimum sludge flows to the digesters. Supernatant from the gravity sludge thickeners flows by gravity to the East Complex primary clarifiers.

- **Dissolved Air Flotation Thickeners**
  Waste activated sludge is continuously pumped to the dissolved air flotation (DAF) thickeners by the WAS pumps. Primary and secondary scum is also periodically pumped to the flotation thickener by the scum pneumatic ejectors. The flotation thickener components such as recirculation pumps, the skimmers, and the air compressor system operate continuously. The thickened sludge is transported to the thickened sludge wet well, and periodically pumped out by the DAF thickened sludge pumps. The thickener subnatant flows by gravity to the DAF Building and then to the gravity belt thickener (GBT) recycle well, or alternately to the east primary clarifiers.

- **Anaerobic Digesters**
  Anaerobic digestion occurs in two locations, the West Complex and the East Complex. The West Complex consists of three digesters (Nos. 1, 2, and 3) and two Sludge Storage Tanks (Nos. 1 and 2). The East Complex consists of four digesters (Nos. 4, 5, 6, and 7). During the Tenth Addition Sludge Storage Tank 3 was converted to Digester 6 and Digester 7 was constructed.



NOTE:
ALL VALVES ARE NOT SHOWN.

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Figure 5-4
SOLIDS PROCESS SCHEMATIC

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A Temperature Phased Anaerobic Digestion (TPAD) system was implemented during the Tenth Addition. In this system Digesters 4, 5, and 6 were to be operated at thermophilic temperatures in a batch mode. Each of these digesters was to sequentially cycle through filling, holding, and withdrawing periods. The holding cycle time would vary between 12 to 24 hours depending on the thermophilic temperature being maintained in the digester to allow the biosolids to meet the EPA time/temperature requirements to attain Class A biosolids. The biosolids withdrawn from these digesters were to be sent to Digester 7 for a period of time to cool before they were pumped to Digesters 1, 2, and 3 for mesophilic anaerobic digestion.

Due to various operational issues, the digestion process is being reconfigured. Currently thickened primary sludge and thickened waste activated sludge are pumped continuously into one of the seven digesters on a rotating basis. It is expected that the digestion operation will be changed to an acid-gas phased digestion system. The thickened primary and waste activated sludges will be sent to one or two acid digesters having a very short detention time. Further digestion of this material will occur at thermophilic temperatures in the seven existing digesters and a new Digester 8.

- **Digester Gas Compression, Treatment and Storage**
  Digester gas is collected from digesters Nos. 4, 5, 6 and 7, and piped to the gas control rooms in Sludge Control Building (SCB) 2. The gas from each digester passes through a dedicated foam separator and is drawn to gas boosters in SCB 2. A portion of the gas is drawn into mixing compressors, is recycled and used to mix the contents of these four digesters. Digester gas from digesters Nos. 1, 2 and 3 is piped to the same header which is tied to the gas boosters in SCB 2. The gas storage in Sludge Storage Tanks 1 and 2 is also tied to this same header, and the storage tanks serve to control the pressure in the raw gas piping. When gas pressure reaches approximately 7.5 inches water column (w.c.), the gas holder covers will begin to rise and store gas. When the cover exceeds 75 percent full, the pressure will begin to rise until it reaches 9.2 inches w.c. When the covers are full, the waste gas is flared, or the gas will release around the sides of the floating covers.

  All gas produced is diverted to gas boosters that raise the gas pressure for transport to the gas treatment facilities outside of SCB 2. The boosters discharge through gas treatment which includes hydrogen sulfide removal using iron-impregnated wood chips, moisture removal using a condenser and chiller, and siloxane removal via carbon filtration. Gas from the treatment system is reheated to 80 degrees F and returned to the gas system at a pressure of 3 psi.

Treated gas is used as fuel for six hot water boilers (three in the Boiler Building and three in SCB 2) and three gas engines. Two gas engines in SCB 2 drive 475 KW induction generators. The third engine in the East Blower Building drives a positive displacement air blower. Surplus gas is flared through a waste gas flare, located near SCB 2.

When digester gas pressure and storage volume is low, a natural gas blending system is started and blended gas is used to supplement the digester gas. This gas is generally used to fuel the three boilers in the Boiler Building and the blower engine.

Natural gas is also available for direct use by the boilers in SCB 2.

- **Sludge Heating System**
  Digester heating for the East Complex is provided by heat recovered from generator engines located in SCB 2. Heat recovered from the generator engines is the primary heat source. Three boilers located in SCB 2 provide supplemental heating for the process hot water system. Each boiler has a rated capacity of 5.9 MMBtu per hour, or 50% of the total process heating requirements with one of the boilers used as a standby. These boilers can use digester gas or natural gas as a fuel source. The existing process heating system is used for heating Digesters No.1, 2, and 3. The three boilers in the Boiler Building have a rated unit capacity of 4.3 MMBtu per hour and will be sufficient for meeting the heating requirements for Digesters 1, 2 and 3 and the West Zone.

- **Digested Sludge Storage Tanks**
  The digested sludge storage tanks provide a reservoir for digested sludge and digester gas to facilitate downstream sludge dewatering and gas utilization operation. Biosolids flow by gravity from the west digesters to the storage tanks and are pumped from the east digesters to the storage tanks. Digester gas is stored in the floating gas holder covers.

- **Gravity Belt Thickeners**
  The GBT feed pumps pump digested sludge from either sludge storage tank to the GBTs for thickening. One of the GBTs also serves as a backup to the DAF thickeners in the event one of the DAF thickeners is out of service. The thickened sludge is transferred to the Metrogro Storage Tanks. GBT filtrate and belt washwater flow by gravity to the recycle wet well. The GBT recycle pumps pump recycled water to the plant flow splitter.

  The GBT polymer system consists of both a dry polymer feed process and a liquid feed process and is located in the GBT Building and the GBT Polymer Building. The polymer system is sized for two GBTs operating at peak capacity. When one of the DAF thickeners is out of service, polymer can be pumped to the operating DAF thickener to assist in thickening of the WAS.

- **Centrifuge**
  The existing centrifuge was installed during the plant's Tenth Addition to produce a dewatered cake material needed to support the development of a Class A soil-like end product (MetroMix). The centrifuge is in good condition. The centrifuge dewaters digested sludge, which is then transported to the Biosolids End-Use Production Building on a belt conveyor. One centrifuge is installed, and sufficient space is available in the Dewatering Building for a second unit. The centrifuge polymer system is sized to provide polymer for two centrifuges. To date, the centrifuge has been used on a limited basis due to the challenges the District has encountered with implementing a Class A digestion process.

- **Metrogro Storage Tanks**
  Thickened biosolids (Metrogro) are stored in the three existing Metrogro storage tanks with a total volume of 19.4 million gallons. Each tank is covered with an aluminum dome to collect odorous air and is equipped with six 15-horsepower submersible propeller mixers to provide a uniform feed for the Metrogro land application program.

- **Biosolids End-Use Production Facility**
  The Tenth Addition Facility Plan called for approximately 10-25% of the biosolids to be dewatered and mixed with amendment materials to produce a "soil-like" end product (MetroMix). The end-product is designed to complement but not compete with the Metrogro Liquid Land Application Program. Space is provided in the facility to store dewatered cake, amendment materials, and final product. The facility has a covered asphalt pad and an additional paved work area that is not covered.

- **Plant Water System**
  The existing plant water system was installed during the plant's Tenth Addition to provide non-potable water use for the treatment processes. The plant water system is equipped with booster pumps, automatic strainers, and a disinfection system. The plant water system serves gravity belt thickeners, centrifuge, digester/storage tank cleaning, liquid ring gas compressors for the digester confined gas mixing system, headworks facility, odor beds, polymer make-up systems and general washdown.

5.2.3. Operation and Maintenance Facilities

The MMSD operation and maintenance facilities include Operations Building, Maintenance Shop Nos. 1 and 2; Storage Building Nos. 1, 2 and 3; Service Building, and Vehicle Loading Buildings. The plant maintenance staff has identified the following items to be addressed and improved for the operation and maintenance facilities:

Personnel Facilities:

- Laundry area
- Lunchroom
- Locker room facilities with showers
- Restrooms

Office and Support Facilities:

- Office area for supervisors
- Office area for purchasers
- Purchasing section is close to the Maintenance Shop and induction heater, which makes a lot of noise.
- Library area – O&M manuals, plans, vendor manuals, etc.
- Computer work space/desks
- Parking area
- Wireless network

Maintenance Facilities

- Work space for mechanics and electricians
- Work space for Monitoring Services/Sewer Maintenance
- Work space for Building and Grounds crew
- Welding and machining areas
- Vehicle maintenance area
- Loading dock and drive up delivery area
- Sandblasting and pump washing areas
- Painting room
- Drive-through vehicle parking areas

Storage Facilities

- Inventory area and un-inventoried parts storage area
- Tool and equipment storage for mechanics
- Tool and equipment storage for electricians
- Tool and equipment storage for Monitoring Services/Sewer Maintenance
- Tool and equipment storage for Building and Grounds crew
- Vehicle storage
- Portable pump storage
- Large parts storage area for spares, mixers, maci pumps, heads, old breakers, etc.

5.2.4. Plant Hydraulics Analysis

The plant consists of four main sections – headworks, West Complex treatment train, East Complex treatment train, and ultraviolet (UV) disinfection and effluent pumping. The headworks contain fine band screens, grit basins, and a weir flow splitter to control the flow distribution between the East Complex and the West Complex. Each treatment train includes rectangular primary clarifiers and aeration basins followed by circular secondary clarifiers. The UV disinfection receives flow from both the East and West Complexes and discharges final treated effluent by pumps to receiving water bodies. Gravity flow and a series of weirs govern the hydraulics of the NSWTP with the final effluent pumps and return activated sludge pumps being the only pumps affecting the overall plant hydraulics of the liquid stream.

During the master planning, a plant hydraulic spreadsheet was developed to determine the overall hydraulic capacities and to identify potential hydraulic bottlenecks at the NSWTP. The hydraulic model is programmed to allow the user to examine each section of the plant separately. Figure 5-5 shows the schematic diagram of the NSWTP hydraulics as modeled in this analysis.

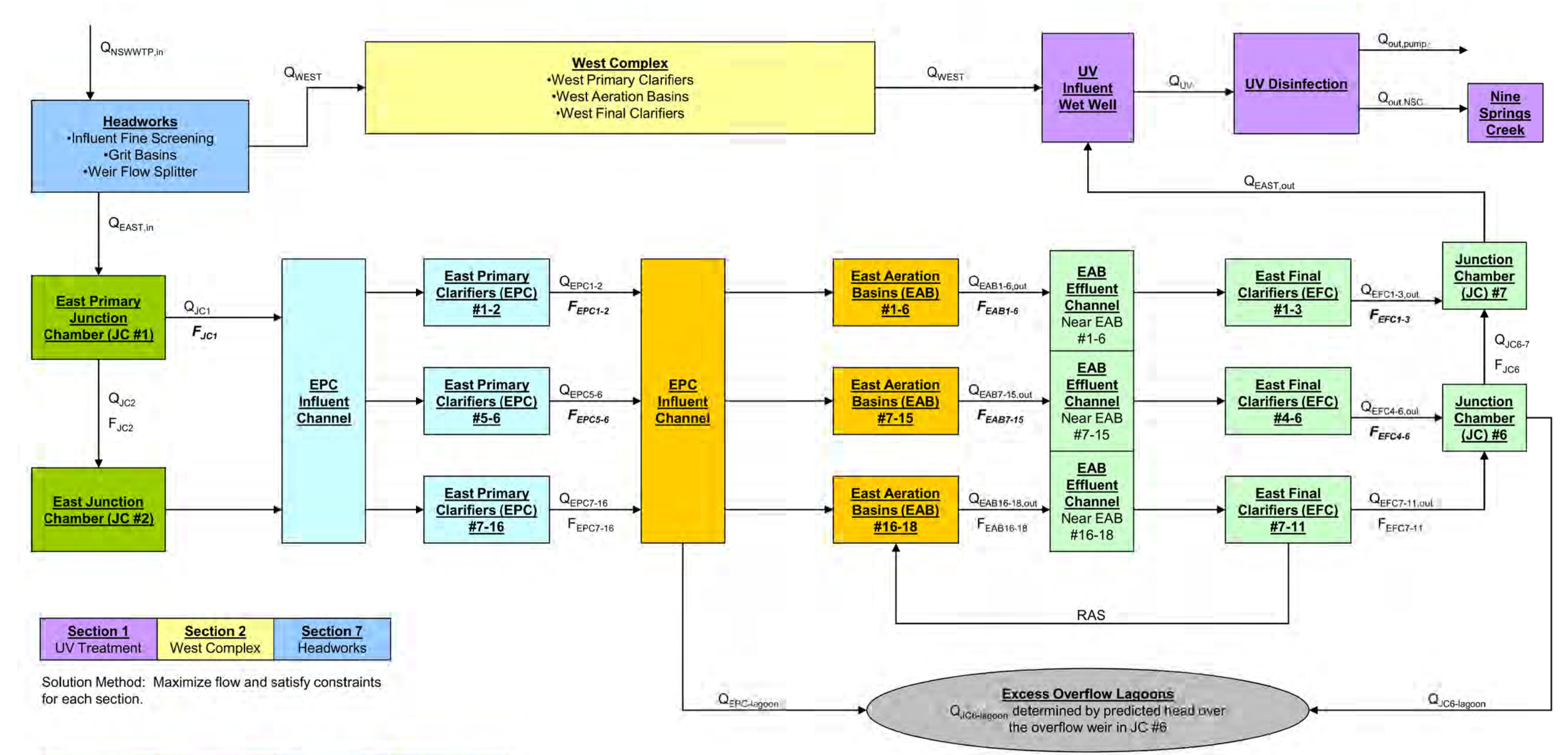
The hydraulic model was utilized to estimate maximum flows under different conditions. The following 4 scenarios were analyzed to determine the maximum hydraulic capacities of the plant:

- **Scenario No.1 – Status Quo without Constraint**
  No modification was made to the existing facilities. The structure minimum freeboard was set to zero. No limit was set to the primary and secondary clarifier overflow rates.

- **Scenario No.2 – Status Quo with Constraints on Clarifier Overflow Rates**
  No modification was made to the existing facilities. The primary and secondary clarifier overflow rate upper limits were set to the pre-determined values.

- **Scenario No.3 – Status Quo with Constraints on the freeboard Heights**
  No modification was made to the existing facilities. The structure minimum free board was set to be 1.0 ft. No limit was set to the primary and secondary clarifier overflow rates.

- **Scenario No.4 – Diversion from Splitter Structure to Effluent Storage Lagoon**
  In this scenario, a 72" excess diversion flow pipe was added from the flow splitter structure to the excess flow storage lagoons. Excess flow during peak flows will be diverted to lagoons. The structure minimum freeboard



Section 1 UV Treatment	Section 2 West Complex	Section 7 Headworks

Solution Method: Maximize flow and satisfy constraints for each section.

Section 3 East Final Clarifiers	Section 4 East Aeration Basins	Section 5 East Primary Clarifiers	Section 6 East Primary Junctions

Solution Method: For a given flow in a section, adjust flow splits until predicted upstream water surface elevation at end of section is equal through all possible flow paths for that section.

*Predicted water surface elevation in EAB Effluent Channel Near EAB #16-18 does not need to equal that Near EAB #1-15, but predicted water surface elevation in EAB Effluent Channel Near EAB #1-6 must equal that Near EAB #7-15.

Q = Flow

F = Flow Split

F (bold italics) indicates split adjusted by Solver

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REVISIONS			
NO.	BY	DATE	REMARKS

DES ______
DWN ______
CKD ______

MMSD
50-YEAR MASTER PLAN

Figure 5-5
NSWTP HYDRAULIC MODELING SCHEMATIC

DATE ______
SHEET ___ OF ___
CAD REF. NO. ______

was set to zero. No limit was set to the primary and secondary clarifier overflow rates.

The hydraulic model analysis results are presented in Table 5-3.

Table 5-3. Plant Hydraulic Analysis Results

Parameters	Scenario #1	Scenario #2	Scenario #3	Scenario #4
Maximum Total Plant Flow (mgd)	169	76	140	249
East Complex Flow (mgd)	91	36	71	91
West Complex Flow (mgd)	78	40	68	78
Excess Flow Diverted from Flow Splitter to Lagoons (mgd)	0	0	0	80
Effluent Building Flow (mgd)	125	76	116	125
Disinfected Effluent Overflow (mgd)	44	0	34	44
Secondary Treatment Effluent Overflow (mgd)	44	0	24	44
Effluent Return Pump Pumping Rate (mgd)	82	76	82	82
East Primary Clarifier Overflow Flow (gal/sf/day)	4,600	1,500	3,439	4,600
West Primary Clarifier Overflow Flow (gal/sf/day)	3,890	2,000	3,425	3,890
East Secondary Clarifier Overflow Flow (gal/sf/day)	1,605	753	1,423	1,605
West Secondary Clarifier Overflow Flow (gal/sf/day)	915	471	806	915

With no constraints other than preventing wastewater from overflowing the treatment structures, the plant can accommodate a peak flow of 169 mgd with approximately 88 mgd being discharged to either the effluent storage lagoons or the Nine Springs Creek (Scenario #1). When constraints such as clarifier overflow rate and freeboard height, were added to the model, the overall plant hydraulic capacity dropped to 76 mgd and 140 mgd for Scenarios #2 and #3, respectively. Plant hydraulic capacity can be expanded to 249 mgd by adding a bypass pipe from the flow splitter to the effluent storage lagoon (Scenario #4). Under this scenario, the bypassed wastewater flow needs to be pumped back to the plant for treatment after wet flows subside. A detailed hydraulic analysis is included in Appendix A, TM1 – Review of Existing Treatment Facilities.

5.3. Other Regional Wastewater Treatment Plants

The following three regional wastewater treatment plants have potential to serve as satellite treatment plants for the MMSD system and decentralize the wastewater treatment operations:

- Village of Oregon Wastewater Treatment Plant
- Stoughton Wastewater Treatment Plant
- Sun Prairie Wastewater Treatment Plant

MMSD and consultants have made initial contact with those treatment plants. Currently, none of these treatment plants has shown an interest in joining MMSD.

5.4. Current Capacities and Deficiencies

A thorough evaluation was conducted in TM-1 to determine the actual capacities of each process component at the NSWTP. The projected loadings at each process component were then calculated using a mass balance model developed and calibrated during the MMSD Tenth Addition Facilities Plan. The mass balance model is configured to follow both the liquid and solids treatment trains at the plant.

The results of internal flows and loadings were then compared with the rated plant unit process capacities determined in TM-1 to determine any capacity deficiencies at different planning years. The comparison results are presented in Table 5-4.

Table 5-4. Unit Process Rated Capacity and Projected Utilizations

Parameter	Units	Rated Max Capacity	Year 2020		Year 2030		Year 2060	
			Flow & Loading	Capacity Utilization	Flow & Loading	Capacity Utilization	Flow & Loading	Capacity Utilization
Influent Flowmeter Max Hour Flow	MGD	163.3	129	79%	135	83%	173	106%
Fine Screening System Max Hour Flow	MGD	180	129	72%	135	75%	173	96%
Grit System Max Hour Flow	MGD	180	129	72%	135	75%	173	96%
Primary Tank Max Day Flow	MGD	102	66.5	66%	71	70%	84	83%
Primary Tank Max Hour Flow	MGD	102	129	126%	135	132%	173	170%
Aeration Basins Average Loading	lb O2/d	107,744	99,924	93%	117,002	109%	164,295	152%
Aeration Basins Max Day Loading	lb O2/d	154,604	153,711	99%	180,306	117%	250,329	162%
Aeration Blower Average Day Loading	scfm	88,000	46,425	53%	54,356	62%	76,327	87%
Aeration Blower Max Day Loading	scfm	88,000	71,414	81%	83,765	95%	116,296	132%
Secondary Clarifier Solids Loading	lbs/d	5,537,560	3,544,193	64%	4,321,631	78%	8,307,678	150%
Secondary Clarifier Max Day Hydraulic Loading	MGD	190	82.5	43%	87.9	46%	104.6	55%
Gravity Thickener Max Day Solids Loading	lbs/d	118,800	162,966	137%	185,438	156%	289,334	244%
Gravity Thickener Max Day Hydraulic Loading	gpm	1,980	900	45%	1,000	51%	1,400	71%
Dissolved Air Flotation Thickener Max Day Solids Loading	lbs/d	83,160	107,716	130%	123,595	149%	179,539	216%
Dissolved Air Flotation Thickener Max Day Hydraulic Loading	gpm	3,089	1,700	55%	1,800	58%	2,000	65%
Anaerobic Digester Max Month Solids Loading	lbs/d	153,000	153,746	100%	175,806	115%	264,160	173%
Anaerobic Digester Max Month Hydraulic Loading	gpd	389,000	386,461	99%	442,324	114%	661,333	170%
Gravity Belt Thickener Max Week Solids Loading	lbs/h	6,600	4,072	62%	4,994	76%	9,420	143%
Gravity Belt Thickener Max Week Hydraulic Loading	gpm	500	303	61%	346	69%	519	104%
Metrogro Biosolids Storage Tank Loading	gallon/180 d	19,403,232	25,006,320	129%	29,704,140	153%	54,649,440	282%

The unit process capacity utilization analysis shows that the projected loadings at the following facilities will exceed their respective rated capacities by 2030. These capacity deficiency issues are either being addressed by the current ongoing projects by MMSD or can be resolved by minor facility improvements:

- Primary Clarifiers (max hour flow) – As presented in TM1, the current operational data suggests that the existing primary clarifiers may be able to accommodate higher than rated hydraulic loadings and still maintain satisfactory TSS removal. Due to lack of recorded high hydraulic loading data, full scale hydraulic capacity tests are recommended to determine the actual hydraulic capacity of the existing primary clarifiers.

- Aeration Basins (average and max day organic loadings) – The capacity analysis shows that the projected average and maximum day organic loadings slightly exceed the rated aeration tank organic capacities. Design documents from the MMSD 9th Addition show that the existing aeration tanks still have space to install additional aeration diffusers. Preliminary analysis shows that, by adding additional diffusers, the organic capacity of the existing aeration tanks can be increased by 10 to 15 percent to meet the future capacity demand at 2030.

- Biosolids Disposal Facilities:
    - Gravity Thickener (max day solids loading)
    - Dissolved Air Floatation Thickener (max day solids loading)
    - Anaerobic Digester (max month solids loading)
    - Anaerobic Digester (max month hydraulic loading)
    - Metrogro Biosolids Storage Tank (not adequate for 180 days of storage)

MMSD has retained a consultant firm to review the existing biosolids disposal facilities at the NSWTP and to address the capacity needs to 2030.

Based on the previous discussion, MMSD could provide sufficient treatment capacity at the NSWTP to meet capacity needs to 2030 through facility capacity validations and minor facility improvements/upgrades. Therefore, before year 2030, any additional plant expansions or improvements will likely be driven by more stringent regulatory requirements rather than by capacity needs. After 2030, expansion of existing facilities will be required for capacity requirements. There is sufficient land available at the NSWTP site for any expansion required in the Master Plan.

5.5. Existing Facility Condition Assessment

The condition assessment of the existing building and structural facilities, mechanical equipment, and electrical equipment at the NSWTP was conducted by MMSD staff in late June and early July in 2008. A condition rating was provided for each asset considered, and an estimated year when major repair and/or replacement of the asset would be needed was also provided. The following scale was used for the condition rating:

1 – Excellent
2 – Good
3 – Adequate
4 – Poor
5 – Very Poor

The following scale as presented in Table 5-5 was used for estimating when major rehabilitation and replacement was needed:

Table 5-5. Condition Assessment Scale Definition

Period	Remaining Service Life	Years When Repair/Replacement Required
A	1-5 years	2011-2015
B	6 - 10 years	2016-2020
C	11-15 years	2021-2025
D	16-20 years	2025-2030
E	20-30 years	2040-2050
F	30-40 years	2050-2060

For the purposes of this assessment, work that would typically be included as part of the annual operations and maintenance budget was not considered major rehabilitation or replacement. For instance, the MMSD funded replacement of many building roofs at the plant in past years out of the O & M budget. The same work needed again in 20 to 30 years was not considered major rehabilitation or replacement for the purposes of the condition assessment. The asset breakdown for condition assessment matched the processes used in the Master Planning TM 1. The assets that supported particular processes were broken down into assets of a reasonable size and grouping for rating. The results of the condition assessment for the NSWTP facilities are summarized in Table 5-6. The more detailed condition assessment report is attached in Appendix L.

Table 5-6. Summary of the NSWTP Facility Condition Assessment

No.	Facility	Years When Repair/Replacement Required		
		Structural	Mechanical	Electrical
1	Headworks	F	B-F	E
2	Flow Splitter	F	F	-
3	Primary Clarifiers	C-E	A-D	B-C
4	Blower Buildings	F	C-D	A-E
5	Aeration Basins	F	A-F	D
6	Secondary Clarifiers	E-F	E	B-D
7	UV Disinfection Facilities	-	F	B
8	Sludge Control Buildings	E-F	C-F	-
9	Plant Effluent Pumping Facilities	F	C-E	E
10	Oil Storage Building	F	-	-
11	Primary Sludge Pumping Stations	F	-	-
12	Gravity Thickeners	F	D	C-E
13	Dissolved Air Floatation Thickeners	F	D	C-E
14	Anaerobic Digesters	D-F	B-F	E
15	Digester Sludge Storage Tanks	D	-	-
16	GBT Facilities	F	A-E	A-D
17	GBT Polymer Facilities	F	D-E	D
18	Centrifuge Facilities	F	D-E	D-E
19	Metrogro Storage Tanks	F	A-E	D
20	Vehicle Loading Facilities	F	A	C
21	Biosolids End-Use Facilities	F	-	-
22	Plant Water Facilities	-	C-E	D
23	Side Stream P Removal Facilities	F	C	A
24	Engine Generator			A-E
25	Gas Control Building	F	D-E	D
26	Boiler Building	E	D-E	C-D
27	Metrogro Vehicles	-	C	-
28	Nine Springs Power Distribution System	F	-	A-F
29	Process Control System	-	A-D	B-E
30	Plant Roads	B-E	-	D
31	Operations Building	F	C-D	A-C
32	Underground Piping at Nine Springs	-	A	
33	Badfish Creek Effluent Pumping Facilities	-	D-E	
34	Badger Mill Creek Effluent Pumping Facilities	-	F	
35	Maintenance Shops	F	D	C
36	Service Building	E	D	C
37	Storage Buildings	E-F	D	D

5.6. Budgeted Capital Improvement Projects

MMSD has prepared a list of capital improvement projects that have been scheduled for implementation between 2009 and 2018 to address the foreseeable system capacity needs and to improve the existing facility conditions. The original report is attached in Appendix K. A summary of the budgeted treatment facility capital improvement projects is presented in Table 5-7.

Table 5-7. Summary of Treatment Facility Capital Improvement Projects (2009-2018)

Account #	Project	Year of Implementation
822-00-40	Sugar River Plant Site Purchase	2009
440-00-21	Sugar River USGS Gauging Station	2009
822-00-55	Solids Handling Improvements	2009-2010
822-00-56	Septage Receiving Improvements	2009-2010
822-00-57	Eleventh Addition	TBD
822-00-59	Process Control System Upgrade	2009-2011
822-00-65	Operations Building HVAC Rehab	2011
828-55/440	MMSD Long Range Master Facility Plan	2009
828-57/440	Treatment Plant Asset Management Plan	2009
828-00-58	Solids Handling Facility Planning	2009
830-00-54	Telemetry System - Third Upgrade	2010-2011

6. PLANNING VARIABLES, REGULATORY ANALYSES & PUBLIC INVOLVEMENT

To prepare for the subsequent planning alternative development and evaluation, a series of studies were conducted to identify and evaluate the major factors that will impact MMSD's operations and planning efforts in the 50 year planning period. These studies include: planning variables identification; regulatory trend analyses; and public involvement. The details are described as follows:

- The consultants worked with the TAC and MMSD staff to identity the major planning variables that will govern or impact MMSD's available options for continuing to provide high quality service over the 50-year master planning period. Details are provided in Appendix D, TM4 – Planning Variables.

- A preliminary regulatory review was conducted to evaluate the existing and foreseeable future regulatory issues potentially affecting MMSD's planning and operations in the next 50 years. Details are provided in Appendix E, TM5 – Regulatory Review and Analyses.

- The public involvement efforts were conducted with interested communities and agencies. Phase I efforts included the development and distribution of an educational "fact sheet" and questionnaire, compilation of the questionnaire responses, and presentations of the master planning process and elements to numerous audiences. Details are provided in Appendix J, Phase 1 Public Involvement Summary. A second phase involved presentations of preliminary planning results at eighteen public meetings and an open house at Nine Springs. The second phase public meetings are also listed in Appendix J.

6.1. Planning Variable Identification

The planning variables and driving forces that were identified served as the basis for the development and evaluation of planning alternatives. The following planning variables were identified:

- Location of Treatment Plants

  New satellite treatment plants may be constructed to address the issues with regard to capacity deficiencies, imbalanced inter-basin water transfer, and treated effluent reuse. New satellite treatment facilities should be close to population centers and potential effluent end users. Proximity to wetlands for the use of effluent polishing could be desirable.

- Biosolids Management

  MMSD currently uses anaerobic digestion to produce biosolids that are recycled to agricultural land. An initiative is underway to add flexibility by developing a soil like product that can be used in non-agricultural settings. The biosolids currently contain significant levels of phosphorus. There is already an excess level of phosphorus within certain portions of the Yahara watershed. For this reason, there may be a need in the future to export biosolids from the watershed. Emerging compounds of concern in biosolids may drive future regulations and limit the ability to beneficially reuse biosolids.

- Effluent Discharge and Reuse

  Increasing regulatory pressure and energy costs may limit the long term viability of pumping all treated effluent to Badger Mill Creek and Badfish Creek. Also, water conservation within the watershed is considered a primary issue to address in the future. The volumes and locations at which MMSD discharges its effluent will be a major factor in sustaining water levels in streams and aquifers throughout the watershed.

- Regulatory Trends

  Future regulatory requirements could significantly impact MMSD's planning and operations over the planning period. Areas of particular importance include: phosphorus criteria; total nitrogen criteria; chlorides; mercury and other toxics; thermal standards; microconstituents in effluent and biosolids; water quality assessments; Rock River TMDL development; water balance issues; groundwater rules for discharges to land and subsurface; and requirements for land application of biosolids.

- Stormwater management

  Currently communities served by MMSD have separate storm and sanitary sewer systems. Stormwater is captured in dedicated storm sewers and discharged to detention basins or directly into adjacent water bodies. Currently the MMSD has no involvement in stormwater management, but might become involved if the following three conditions were met:

  a. A stormwater problem with water quality implications requires a regional solution;
  b. The involved municipalities are unable to implement a coordinated plan; and
  c. There is consensus that the MMSD is the appropriate agency to deal with the issue.

- Environmental Impacts

  The overall environmental impact of MMSD's facilities and operations should be considered in the planning efforts. Examples include carbon footprints, waste stream/hazardous material, water resource consumption, and air quality.

- Future Flow Projections

  Future flow projections have significant impacts on capacity requirements for both the collection system and treatment facilities. The following scenarios will impact the future flow projections:

  a. The Madison Design Curve currently being used for estimating peak flows could be too conservative.
  b. Impacts of water conservation
  c. Impacts of Inflow/Infiltration improvement
  d. Population growth rate
  e. Population growth distribution
  f. Increased precipitation associated with climate change

- Construction/Operational Costs

  Construction and operational costs will be a major driver for all scenarios and alternatives. These costs include: energy, construction materials, land acquisition, manpower, contracted services, chemicals, fuel and utilities.

- Public Acceptance

  Public acceptance will play an important role as MMSD evaluates effluent reuse opportunities; construction of regional treatment plants; construction of un-manned neighborhood treatment plants; and alternative biosolids management options.

6.2. Regulatory Review and Analyses

The following existing and foreseeable future regulatory issues potentially affecting MMSD's planning and operations in the next 50 years were reviewed during the planning process for certain operational situations.

- NSWTP Continued Discharge to Badfish Creek

  1. Rock River Basin phosphorus and sediment TMDL: The TMDL is being developed by EPA. It appears MMSD will have a waste load allocation (WLA) for total P as a result of this TMDL. This will impact the District's discharge to Badfish Creek.

2. Statewide phosphorus criteria: Based on current draft administrative code language, the resulting phosphorus water quality criteria (WQC) for Badfish Creek and the Yahara River would be 0.075 mg/L and 0.100 mg/L, respectively. Depending on the background concentration of phosphorus in Badfish Creek (i.e., from groundwater or other sources of dilution water), some dilution may be allowed when determining the associated water quality-based effluent limit (WQBEL) for phosphorus.

3. A revised permit phosphorus limit should be anticipated in the District's 2014 WPDES permit.

- NSWTP with Increased Discharge to Badger Mill Creek

  Permitted pollutant loadings to Badger Mill Creek included in MMSD's current WPDES permit are based on a discharge volume of 3.6 mgd. MMSD may consider alternatives that increase this discharge. Badger Mill Creek is a tributary to the Sugar River. The Sugar River has been designated an exceptional resource water (ERW). For an increased discharge, the effluent limits could be impacted by the more stringent rules related to the Sugar River.

  1. Phosphorus criteria: Based on current draft administrative code language, the resulting phosphorus water quality criteria would be 0.075 mg/L for Badger Mill Creek. Depending on background concentrations, some dilution may be allowed when determining the WQBEL for P. However, the P concentration for an increased discharge at this location may be limited further because the Sugar River is designated as an exceptional resource water (ERW).

  2. DNR interpretation of antidegradation requirements: Antidegradation rules are contained in NR 207. Since the Sugar River is an ERW, it is subject to more stringent antidegradation requirements. In general, a new discharge to an ERW needs to meet upstream water quality. Regulations are not as stringent for an increased existing discharge; however, the permittee would still need to demonstrate there will either be no significant lowering of water quality or that the project has sociological and economic benefits.

- NSWTP with Discharge to Lake Waubesa via Nine Springs Creek

  MMSD may consider discharging highly treated effluent to Nine Springs Creek or wetlands tributary to Mud Lake and Lake Waubesa. However, the effluent limits would likely be most impacted by the more stringent regulations and statutes related to Lake Waubesa as follows:

1. Thermal standards: If this discharge location is construed as an existing outfall for MMSD, it is possible that it would be eligible for a variance to the proposed thermal standards outlined in draft revisions to NR 102 and NR 106. Otherwise, some mitigation of effluent temperature may need to be included for a discharge at this location.

2. P criteria: The current draft administrative code language for P criteria would result in a P WQC around 0.040 mg/L for shallow lakes like Lake Waubesa. Depending on the background concentration of P in the lake, some dilution may be allowed when determining the WQBEL for P.

3. DNR interpretation of requirements in Wisconsin State Statute 281.47: This statute was the driver for MMSD diverting effluent around the Madison lakes beginning in the late 1950s. The statute does not explicitly prohibit direct discharge of effluent to the chain of lakes including Lake Waubesa, but it does place conditions that must be met for direct discharges to occur. The DNR is given authority to determine whether these conditions are met. Based on DNR discussions during Madison Gas and Electric's (MGE's) cogeneration facility planning, it appears the effluent quality would need to be close to background surface water quality for P prior to approval of a Lake Waubesa discharge. Background concentrations may be lower than the 0.040 mg/L proposed shallow lake criteria.

- Upper Lake Mendota Watershed Discharge

MMSD may consider constructing a satellite WWTP with discharge of highly treated effluent to the upper Yahara River or wetlands tributary to Lake Mendota. The effluent limits would likely be subject to the more stringent state statutes related to Lake Mendota as follows:

1. Phophorus criteria: Based on current draft administrative code language, the resulting phosphorus water quality criteria would be around 0.015 mg/L for Lake Mendota. Depending on the background concentration of P in the lake, some dilution may be allowed when determining the WQBEL for P. The DNR has noted that a TMDL-like approach could be required before setting WLAs, LAs, and WQBELs for a Lake Mendota discharge so that load and wasteload allocations can be assigned to all the sources of P to the lake.

- Sugar River Watershed Discharge

  MMSD is considering construction of a satellite WWTP with discharge of highly treated effluent to the Sugar River or its tributaries. A discharge to the Sugar River would be affected by the issues summarized below:

  1. Phosphorus criteria: Based on current draft administrative code language, the resulting phosphorus water quality criteria would be 0.075 mg/L for the Sugar River; however, antidegradation requirements contained in NR 207 would also apply. For an Exceptional Resource Water (ERW), this essentially means the new discharge would need to meet background water quality. For example, if the background P concentration in the Sugar River is 0.050 mg/L, the effluent limit could be 0.05 mg/L.

  2. Chlorides: Since the Sugar River is designated an ERW, it is possible the chloride concentrations in the discharge would need to meet background concentrations in accordance with NR 207. The DNR has expressed some willingness to discuss this issue further with the MMSD, particularly if there is a net environmental benefit associated with the discharge such as restoration of water balance or other benefits.

  3. Ammonia, biochemical oxygen demand (BOD), and other limits: It is possible that the effluent limit for ammonia, BOD, total suspended solids (TSS), and other parameters may need to be equal to background concentrations of these parameters because of the ERW designation for the Sugar River. The DNR Guidance on the “13 pound rule” contains calculations related to assimilative capacity and may impact BOD limits for non-variance streams; this guideline may apply if the background concentration does not.

- Koshkonong Creek Discharge

  Another alternative MMSD may consider is a cooperative agreement with Sun Prairie to treat a portion of MMSD’s wastewater flow. This would result in an increased discharge to Koshkonong Creek. An increased discharge to Koshkonong Creek may be affected by the issues summarized below.

  1. Phosphorus criteria: Based on current draft administrative code language, the resulting phosphorus water quality criteria would be 0.075 mg/L for Koshkonong Creek and 0.040 mg/L for Lake Koshkonong. Depending on background P concentrations, some dilution may be allowed when determining the WQBEL for P.

- Other Surface Water Discharge Locations Including Stream Base Flow Augmentation

  Other surface water discharge locations may be considered, such as a new discharge to the Yahara River just downstream of Lake Waubesa. A discharge at this location would likely have similar issues and benefits as those discussed above for a discharge to Nine Springs Creek and Lake Waubesa.

  Base flow augmentation using highly treated WWTP effluent may also be considered in the future, particularly for urban streams. For example, relatively small volumes of effluent could be further treated at the Sun Prairie WWTP or a future north MMSD WWTP and discharged to streams in the northeast portion of the Lake Mendota or north Lake Monona watersheds. Starkweather Creek has experienced a reduction in dry weather base flows over the years, possibly caused by the high percentage of impervious surfaces in the watershed and pumping of groundwater in Madison, and could be a good candidate to receive flow augmentation in this manner. A discharge of treated effluent at this location would have similar issues and benefits as those discussed above for a discharge to the upper Yahara River and Lake Mendota.

- Groundwater Recharge

  Groundwater recharge using effluent is being practiced in several locations around the state, particularly in the Wisconsin River Valley and other locations where soils are sandy and thus conducive to infiltration. A typical method of effluent groundwater recharge is to use seepage cells (also called absorption ponds), which are regulated under NR 206. Current effluent limitations for discharge to absorption ponds include:

BOD	50 mg/L
TN	10 mg/L
TDS	500 mg/L
Chloride	250 mg/L

  Groundwater monitoring is usually required for absorption ponds and the relevant groundwater standards at the design management zone boundary (250 feet from the seepage cell boundary) or at the property line would apply. These are contained in NR 140. The groundwater preventive action limit (PAL) for chloride is 125 mg/L and the enforcement standard (ES) is 250 mg/L.

  For this type of discharge, it appears the largest hurdles for MMSD to overcome would be TN and chloride effluent concentrations. Biological nitrogen removal can be used to reduce TN to below 10 mg/L. If a variance could not be obtained, chloride concentrations would need to be reduced through source reduction or reverse osmosis treatment prior to discharge to an infiltration gallery and may also need to be reduced prior to a discharge to absorption ponds.

- Nonresidential Irrigation

  The current MMSD permit contains provisions related to use of effluent on the Nine Springs Golf Course in Fitchburg as a demonstration project. This type of discharge would be regulated under NR 206. Current regulations include a BOD effluent limitation of 50 mg/L. Hydraulic loading rates and load and rest cycles are determined on a case-by-case basis and generally depend on the soil type. Likewise, TN and fecal coliform limits are determined on a case-by-case basis. Groundwater monitoring is often required for these systems, particularly when significant pretreatment is not provided. Groundwater standards for chloride (125 mg/L PAL and 250 mg/L ES) may be of greatest concern for MMSD's effluent.

  Nonresidential irrigation would generally involve spray or drip irrigation of treated wastewater onto agricultural fields, grass lands, golf courses, or similar areas. Generally TN applications are limited to crop uptake rates, which are on the order of 165 lb/acre-year for corn and 300 lb/acre-year for certain grasses like reed canary grass. Groundwater monitoring is often required for determining compliance with groundwater standards.

- Industrial or Commercial Reuse

  Wastewater effluent can be used for industrial noncontact cooling and other noncontact uses. Wisconsin currently has no standards for the treatment of effluent for use in an industrial facility.

  It may also be possible for effluent to be reused for noncontact industrial cooling water. Several individuals responding to the MMSD interest survey indicated that commercial car wash use may be another viable alternative; however, the locations of such facilities may be too diffuse for cost-effective conveyance of the treated effluent. The concept should be initially explored with the largest water users in Dane County who are believed to use fresh water for nonpotable uses.

- Residential Reuse

  It has been proposed by several individuals that treated effluent could be reused for toilet flushing, residential lawn irrigation, and other residential nonpotable water uses. Such a concept would require effluent treatment to a very high level (potentially California Title 22 standards as noted above for food crop irrigation), require force mains to convey the treated effluent to the residential developments, and require a new infrastructure similar to the "purple pipe" reuse water distribution systems used in the Southwest and elsewhere. This concept may be worth considering for new developments where installation costs would be lower compared to existing developments. However, it is likely that costs of such systems would outweigh the benefits, at least in the short term in the Madison area. For the short term, it appears that residential water conservation measures may provide similar benefits at a significantly lower cost.

- Wetlands Restoration

The DNR has indicated that a discharge to wetlands may be subject to less stringent requirements than a discharge to an ERW stream or the Madison lakes, particularly for restored wetlands. A viable option for a potential Mendota Plant would be to discharge effluent to wetlands to provide the base flow for the wetland system that has been lost because of groundwater table lowering from water supply withdrawals in Madison, Waunakee, DeForest, Windsor, and Sun Prairie. This option may also be useful in lieu of a direct stream or lake discharge in the vicinity of the Sugar River or Nine Springs Creek/Lake Waubesa.

Wetland discharges are regulated under NR 103. NR 103 applies to natural and restored wetlands but not to constructed wetlands for wastewater treatment or polishing; the latter systems are typically constructed with liners separating them from natural waters and are considered a wastewater treatment unit process.

- Biosolids Management

The following biosolids regulations have been identified as possibly being applicable to MMSD's future operations. Within the next 20 years, these regulations along with increased development in the Madison area may result in the requirement for more land and increased hauling distances in the Metrogro program. These regulations may also place additional restrictions on the MetroMix program. In the longer term, MMSD may need to consider additional alternatives for at least a portion of its biosolids such as landfilling. Landfilling may still be considered a beneficial reuse option if biosolids are used as cover material, are used to facilitate decomposition, are part of a landfill bioreactor, or if biosolids additions promote the formation of landfill gas that is then recovered and used to generate electricity.

1. State (NR 204) and federal (40 CFR Part 503) biosolids regulations
2. Runoff management rule (NR 151)
3. Impaired waters (303(d)) listings and TMDLs

4. Local ordinances relating to the use of lawn fertilizers containing phosphorus

6.3. Public Involvement

The initial (Phase 1) public involvement efforts were conducted with interested communities and agencies in early 2008. Phase I efforts included the development and distribution of an educational “fact sheet” and questionnaire, compilation of the questionnaire responses, and presentations of the master planning process and elements to numerous audiences.

- Fact Sheet and Questionnaire

  A two-sided color fact sheet was developed to summarize some of the key concepts of the master planning process. In particular, the fact sheet was designed to educate the audience with respect to some important statistics and history of MMSD and introduce some key issues that MMSD will need to address over the next 50 years. These key issues include the impacts of wastewater effluent diversion around the Madison Lakes on surface water and groundwater resources, as well as potential wastewater reuse concepts.

  The questionnaire was mailed to the public works committee, utilities committee (or commission) or governing body, the administrative and management staff of each of MMSD’s customer communities. It was also mailed to the City of Sun Prairie, City of Stoughton, and Village of Oregon. In addition, the questionnaire was mailed to the Dane County Lakes and Watershed Commission, the Capital Area Regional Planning Commission, and approximately 40 environmental advocacy groups that are active in the Madison area. Approximately 260 questionnaires were mailed.

  MMSD also posted the questionnaire on its Web site and invited interested parties to complete the form on-line. MMSD developed a summary of the questionnaire response statistics and comments made by the respondents and distributed that summary to the Master Planning Advisory Committee in a March 14, 2008, memorandum from MMSD.

- Summary of Presentations

  Fourteen presentations were made by the MMSD staff during the months of February, March, and April in 2008. The presentations and following discussion typically lasted from 20 minutes to one hour, depending on the number of questions and comments received. Presentations were made to the following audiences on the indicated dates:

1. February 19, 2008–City of Verona Public Works Committee
2. February 20, 2008–City of Middleton Water Resources Committee
3. February 28, 2008–Village of DeForest Public Works Committee
4. March 11, 2008–Village of McFarland Public Utilities Committee
5. March 13, 2008–Dane County Lakes and Watershed Commission
6. March 17, 2008–City of Madison Commission on the Environment
7. March 17, 2008–City of Fitchburg Public Works Committee
8. March 25, 2008–Town of Blooming Grove Board
9. March 26, 2008–City of Madison Board of Public Works
10. March 27, 2008–Capital Area Regional Planning Commission
11. March 31, 2008–Village of Waunakee Utilities Commission
12. April 1, 2008–City of Sun Prairie Committee of the Whole
13. April 21, 2008–Village of Shorewood Hills Village Board
14. April 29, 2008–Village of Maple Bluff Public Works Committee

All comments and questions were noted at each of the meetings, as were the responses to any questions. A summary of comments and questions was developed for each of the presentations.

- Presentation Responses – Common Themes and Comments

While each presentation resulted in a unique set of comments and questions, there were a few common themes that came up during the discussions following many of the presentations. A listing of these common themes follows:

1. Many of the audiences were very familiar with the water resources issues in Dane County. It is noted that the audiences are likely more educated with respect to water resources issues than the general public would be.

2. MMSD’s customers are supportive of the master planning process and would like to see MMSD investigate wastewater reuse alternatives. Many commented that new subdivisions could start requiring that wastewater reuse infrastructure be constructed with other utilities.

3. Groundwater depletion seems to be more of a concern than low flows in surface waters, although these are directly related to each other in some locations.

4. Other areas of the country, especially in the south and west, are already reusing treated wastewater.

5. Water conservation was brought up at several presentations.

6. The potential risk of pharmaceuticals in the environment is a concern.

7. The question of how to pay for wastewater reuse infrastructure, as well as potential satellite WWTPs, was asked at several of the presentations.

7. ALTERNATIVE DEVELOPMENT

7.1. Background

The current MMSD model is conveyance of all wastewater to a centralized treatment facility (NSWTP) for treatment with subsequent discharge of the treated effluent to Badfish Creek (75 mgd maximum flow rate) and Badger Mill Creek (3.6 mgd permit-based flow rate). There may be advantages to altering this model by decentralizing treatment through the construction of satellite treatment plants or altering the conveyance system to route wastewater from certain parts of the service area to an existing municipal treatment plant in a nearby community. These advantages could include lower capital costs in the conveyance system and at the NSWTP, reduced operational costs associated with pumping the wastewater and effluent, and environmental benefits realized by returning the effluent closer to the original source of the water.

Implementation of projects to decentralize treatment will take a decade or longer to implement, either because of issues related to the receiving water into which effluent from the satellite plant would be discharged, or due to the length of time it would take to reach agreement with a community with an existing treatment plant. Due to these constraints and the fact that the MMSD has immediate needs to address capacity and condition issues in the conveyance system, there are few near-term decentralization projects that can achieve conveyance system construction cost savings. Projects that address capacity needs of the Nine Springs Valley Interceptor (NSVI) are the exception. Additional capacity in the NSVI will be required in about ten years. This would allow sufficient time to implement a decentralized project in this part of the MMSD's service area. Such a project would have the highest potential to produce capital cost savings in the conveyance system and at the NSWTP where future capacity expansions could be avoided, delayed, or reduced in size. Conveyance capacity needs on the east side of the MMSD are more immediate, and thus decentralized projects in this part of the service area will generally be more costly overall since the opportunity to achieve near-term conveyance system construction cost savings will not be available.

The following key principles were used to develop the projects presented in this chapter:

1. Peak Hourly Flows (PHFs) to Badfish Creek will not exceed the rated 75 mgd of the effluent force main.

2. The growth rate projections for the conveyance system, which include an uncertainty factor to reflect the unknowable location and timing of growth,

will be used for determining when loadings to various conveyance components will reach the design capacity.

3. For evaluation purposes, it is assumed that the NSWTP will need to be upgraded to achieve a lower effluent phosphorus concentration in 2020 and a lower total nitrogen effluent concentration in 2030. Also, the solids processing facilities at the NSWTP will require capacity expansion in 2030.

4. Discharge at Badger Mill Creek at a minimum of 3.6 mgd will be maintained for all alternative projects.

5. Average Daily Flows (ADFs) are based on the 2008 MMSD Collection System Evaluation as prepared by CARPC. Peak hourly flows were based on ADF and the Madison Design Curve (MDC).

Projects were developed based on addressing the projected future needs for either the current NSWTP or the existing MMSD conveyance facilities. Projects are presented for the east side of the MMSD system (Service areas for Pumping Stations (PSs) 6, 7, 9, 10, 13, and 14) and the west side of the MMSD system (Service Areas for PSs 1, 2, 3, 4, 5, 8, 11, 12, 15, 16 and 17).

Projects are organized into near-term projects and long-term projects. Near-term projects are those that would address the need for capacity expansion in the conveyance system required in the next ten to twenty years. Long-term projects are those which, while still viable, cannot be implemented prior to the time the collection system capacity improvements would be required. Examples of long-term projects would include those that would discharge highly treated effluent to Lake Mendota or Lake Monona, effluent reuse projects that would be primarily driven by the economic need to reuse water, or turf irrigation projects on a larger scale that would require the development of a distribution network for the highly treated effluent. Details of the master planning alternative development are provided in Appendix G, TM7 – Development of Planning Alternatives.

7.2. Planning Alternative Projects

Base on the projected capacity needs for the interceptors and pumping stations on the west side of the MMSD service area, the following 4 alternative projects were identified:

- **Project W1 – Nine Springs Valley Interceptor Relief**
  This project includes construction of a new gravity relief sewer paralleling the existing NSVI or construction of a new force main from PS 12 to either PS 11 or the NSWTP. This project will address the capacity deficiency of the existing NSVI during the planning period.

- **Project W2 – Sugar River WWTP**
  This project includes construction of a new wastewater treatment plant (WWTP) in the Verona area. The new WWTP would receive wastewater generated in the PS 17 service area and discharge effluent to the Sugar River downstream of its confluence with Badger Mill Creek, or alternatively, to the headwaters of the Sugar River at CTH PD. This project would eliminate all capacity improvements for the NSVI and PS 12 and PS 11 force mains provided it is constructed prior to 2020.

- **Project W3 – Dual Sugar River Satellite Plants (CTH PD Plant and Nesbitt Road Plant)**
  This project would include construction of two new WWTPs in the Verona area and return of effluent to the headwaters of the Sugar River at CTH PD and to Bader Mill Creek at the City of Madison storm water ponds on Nesbitt Road. This project would eliminate all capacity improvements for the NSVI and PS 12 and PS 11 force mains provided it is constructed prior to 2020.

- **Project W4 – Village of Oregon Discharge to PS 11**
  This project would include incorporating the Village of Oregon into MMSD's service area. Flow from the Village of Oregon would be directed to MMSD PS 11 service area and then be pumped to the NSWTP for treatment.

Based on the projected capacity needs for the interceptors and pumping stations on the east side of the MMSD service area and potential expansion of the NSWTP, the following 6 alternative projects were identified:

- **Project E1 – Mendota WWTP**
  This project would include construction of a new WWTP north of Lake Mendota near the Yahara River to serve the Yahara River and Six Mile Creek watersheds north of Lake Mendota. Effluent from this plant could provide stream flow augmentation to the Yahara River, be used for infiltration to recharge the groundwater aquifers, or be reused for industry or turf irrigation.

- **Project E2 – Starkweather Creek WWTP**
  This project would redirect the gravity flow tributary to PS 13 to a new Starkweather Creek WWTP. Effluent from this plant could provide stream flow augmentation to Starkweather Creek, be used for infiltration to recharge the groundwater aquifers, or be reused by industry or for turf irrigation.

- **Project E3 – PS 13 and PS 14 Service Area WWTP**
  This project would redirect the flow tributary to PS 13 and 14 to a new WWTP located northeast of Madison. Effluent from this plant could provide stream flow augmentation to Starkweather Creek, be used for infiltration to recharge the groundwater aquifers, or be reused by industry or for turf irrigation.

- **Project E4 – Stoughton WWTP Expansion**
  This project would redirect flow from PS 7 and PS 9 service areas to an expanded Stoughton WWTP. Implementation of this project includes the construction of a parallel treatment plant to treat the wastewater diverted from the MMSD system. Biosolids treatment would be provided by expanding the existing biosolids treatment train at the Stoughton WWTP.

- **Project E5 – Centralized High Quality Effluent Treatment Facilities**
  This project would include construction of high quality effluent treatment facilities on the NSWTP property (Refer to Figure 5-2). The high quality effluent could be returned to Badger Mill Creek or other outfalls in the Sugar River watershed, discharged directly to Nine Springs Creek or other surface waters in the Yahara River watershed, used by industry, used for irrigation, used as a water source for infiltration to recharge groundwater, or discharged to maintain wetlands.

- **Project E6 – Sun Prairie WWTP Expansion**
  This project would provide sewer service for the portion of the MMSD's future service area in the Koshkonong Creek watershed by directing flow from this area to the City of Sun Prairie WWTP for treatment.

7.3. Master Planning Alternative Development

After the preliminary screening of the identified alternative projects, master planning alternatives, which are combinations of the projects described above, were developed for further evaluation. Those alternatives are classified into two groups: near-term alternatives which could be implemented between 2010 and 2030 and long-term alternatives which could be implemented between 2030 and 2060.

7.3.1. Near-Term Planning Alternatives

Except for Alternative MP-1 (base planning alternative), the following two key principles were incorporated in making the selection of near-term planning alternatives:

- The proposed alternative project must have an implementation date that allows sufficient time for the MMSD to site and construct the alternative project prior to the time necessary to alleviate an existing MMSD capacity need.
- Alternatives must provide sufficient capacity so that any future expansion of the current advanced secondary treatment facilities at the NSWTP beyond the existing 57 mgd capacity will not be required before 2060.

Based on these criteria, two near-term master planning alternatives were selected for further evaluation. Implementation of either of these alternatives between 2010 and 2030 will address the wastewater treatment and conveyance system capacity needs in the MMSD service area:

- **Alternative MP-1 – Westside Conveyance System Expansion**: This alternative would expand the existing conveyance system and continue the current model of centralized treatment at the NSWTP. This alternative includes four variations to pump treated effluent to different receiving water bodies.
- **Alternative MP-2 – Sugar River WWTP**: This alternative would construct a new high quality effluent treatment plant in the Sugar River watershed to treat wastewater generated in the PS 17 service area, and discharge its effluent to the Sugar River. This alternative includes two variations to discharge treated effluent to different locations of the Sugar River.

The Alternative MP-1 includes the following 4 variations:

- **Alternative MP-1A –** This alternative includes returning 3.6 mgd of effluent from the NSWTP to Badger Mill Creek through the existing outfall in Badger Prairie Park, and returning the rest of the effluent to Badfish Creek. This alternative represents the current operation by MMSD. It serves as the base alternative to be compared to other alternatives.

- **Alternative MP-1B –** This alternative includes returning a total of 7.9 mgd of effluent consisting of 3.6 mgd of regular effluent (effluent generated by the existing processes) and 4.3 mgd of high quality effluent (effluent generated by future high quality effluent processes) from the NSWTP to Badger Mill Creek through the existing outfall in Badger Prairie Park. The rest of the effluent will be returned to Badfish Creek. The regular effluent and the high quality effluent will be blended and pumped to the Badger Mill Creek outfall location through the existing effluent force main. This alternative represents a centralized effluent reuse and watershed balance solution (i.e. it returns water to the watershed from where it was withdrawn).

- **Alternative MP-1C –** This alternative includes returning 7.9 mgd of effluent to the Badger Mill Creek outfall location through the existing force main with 3.6 mgd being discharged to Badger Mill Creek directly and 4.3 mgd being discharged downstream of the confluence of Badger Mill Creek and Sugar River through a new pumping station and a new force main at the vicinity of the existing Badger Mill Creek outfall. The rest of the effluent from the NSWTP will be returned to Badfish Creek. The regular effluent and the high quality effluent will be blended and pumped to the Badger Mill Creek outfall location through the existing effluent force main. This alternative represents a centralized effluent reuse and watershed balance solution. It could achieve similar effluent reuse and watershed balance benefits as the decentralized alternative (MP-2A) discussed later.

- **Alternative MP-1D –** This alternative includes returning 7.9 mgd of effluent to the Badger Mill Creek outfall location through the existing force main with 3.6 mgd being discharged to Badger Mill Creek directly and 4.3 mgd being discharged to the Sugar River at the County Highway PD (CTH PD) through a new effluent pumping station and a new force main in the vicinity of the existing Badger Mill Creek outfall. The rest of the effluent will be returned to Badfish Creek. The regular effluent and the high quality effluent will be blended and pumped to the Badger Mill Creek outfall location through the existing effluent force main. This alternative represents a centralized effluent reuse and watershed balance solution. It could achieve similar effluent reuse and watershed balance benefits as the decentralized alternative (MP-2B) discussed later.

For alternatives MP-1B, 1C, and 1D, 7.9 mgd of effluent needs to be pumped to the existing Badger Mill Creek outfall. Preliminary analysis shows that the existing 20" force main has sufficient capacity for the increased flow, but new pumps would be needed. The layouts of these alternatives are shown in Figures 7-1 through 7-4. The major component projects included in these alternatives are presented in Table 7-1.

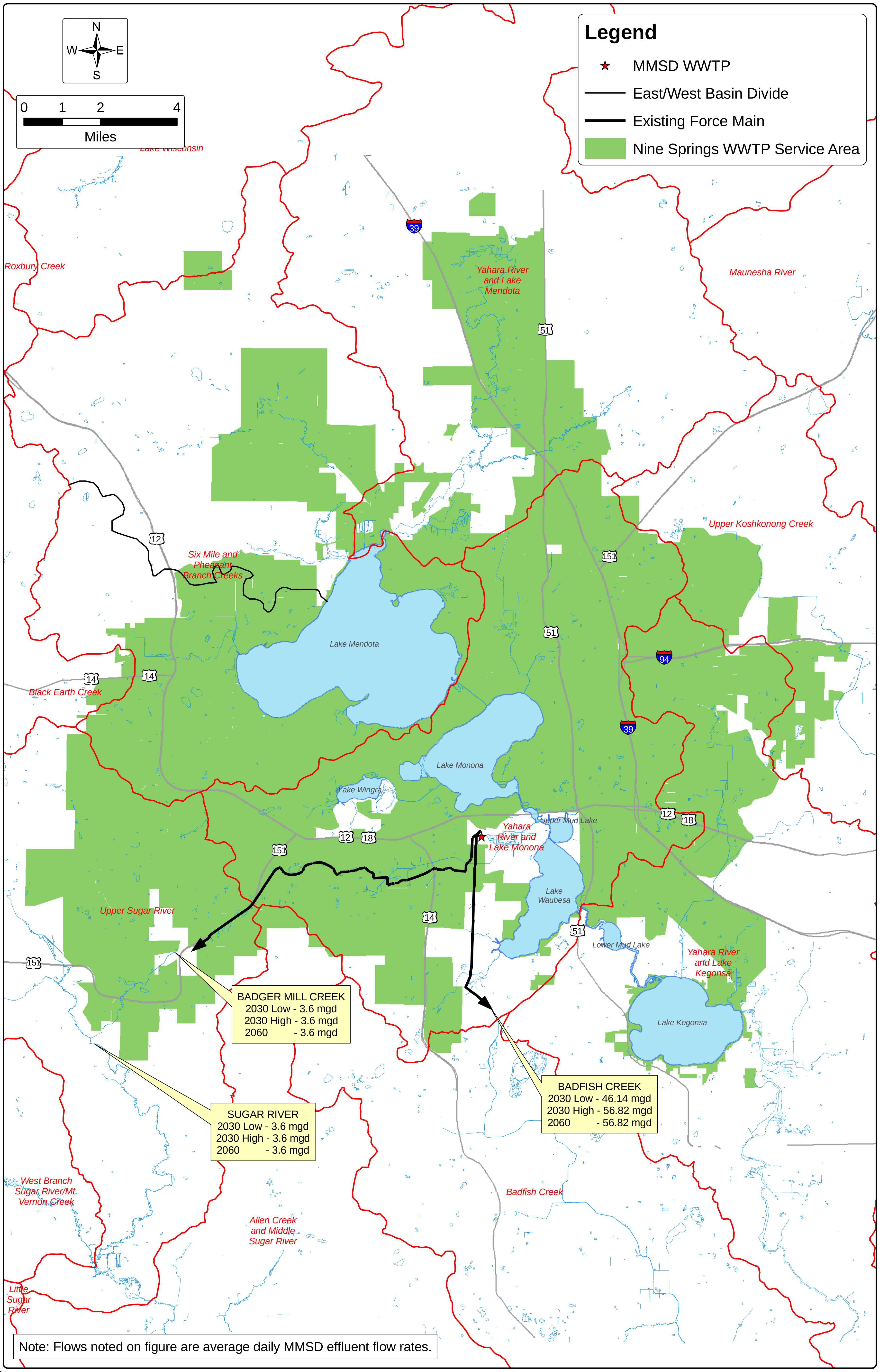

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FIGURE 7-1

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MMSD

ALTERNATIVE MP-1A

MMSD 50-YEAR MASTER PLAN

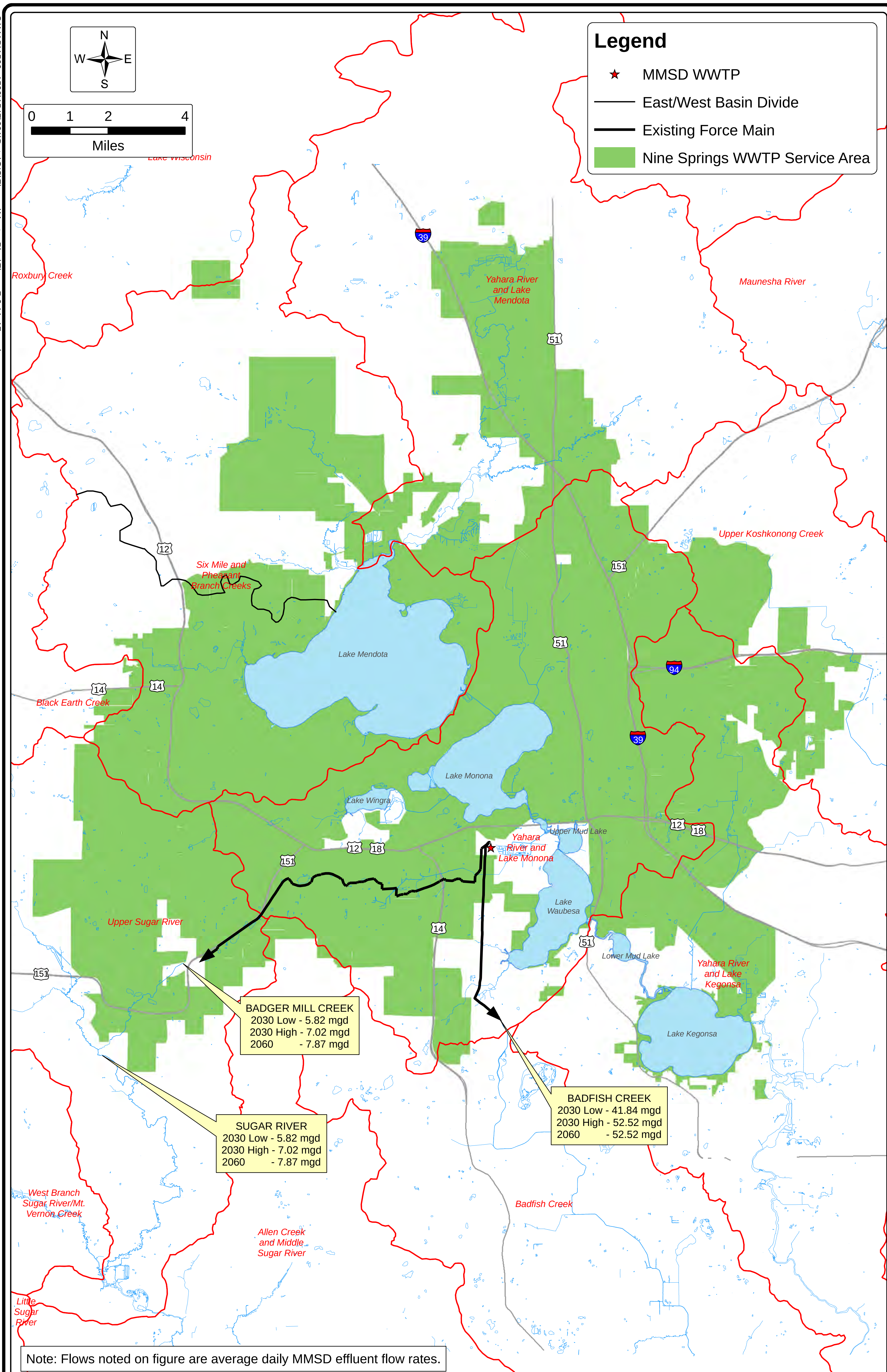

FIGURE 7-2

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ALTERNATIVE MP-1B

MMSD 50-YEAR MASTER PLAN

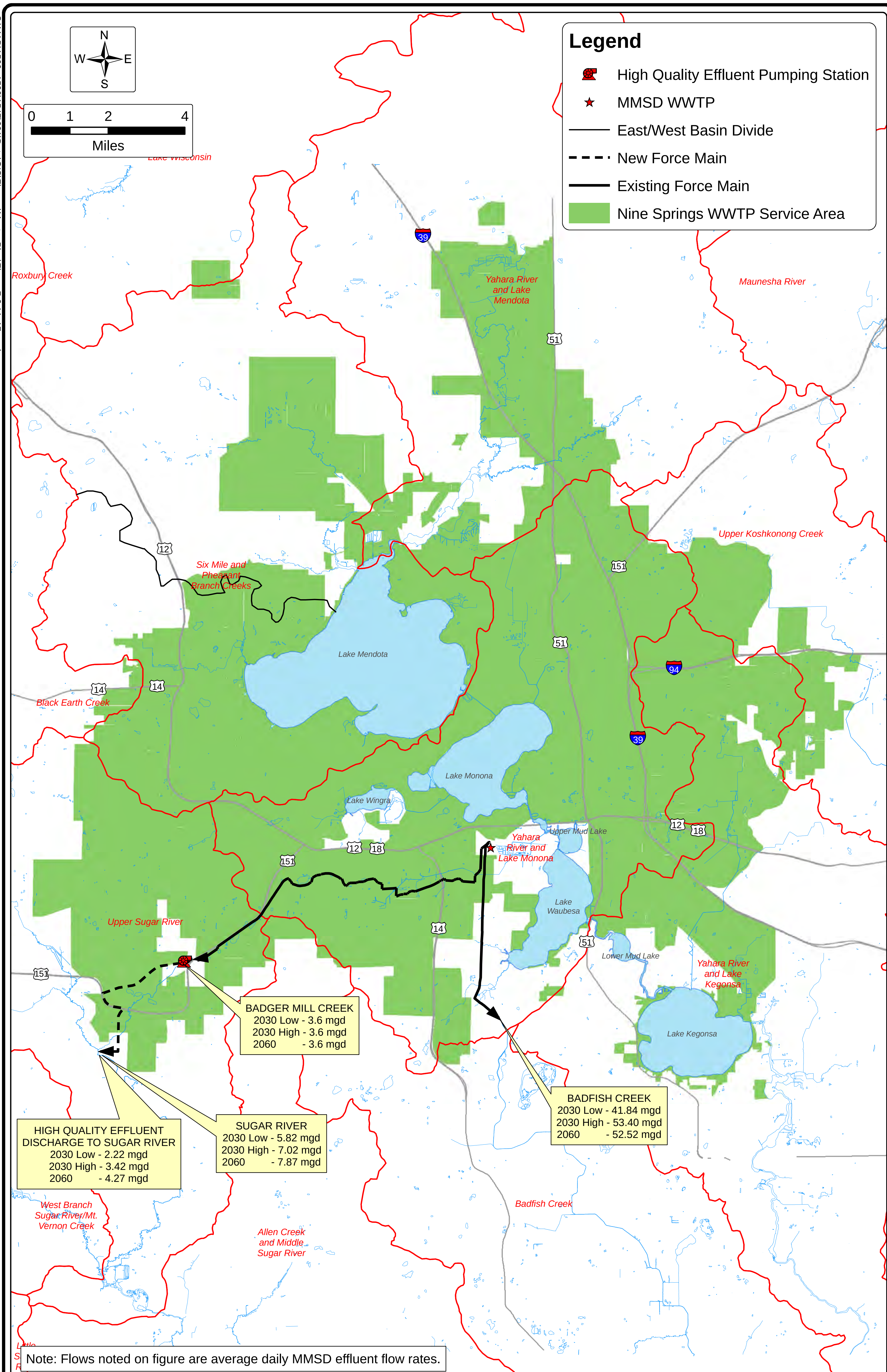

FIGURE 7-3

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ALTERNATIVE MP-1C

MMSD 50-YEAR MASTER PLAN

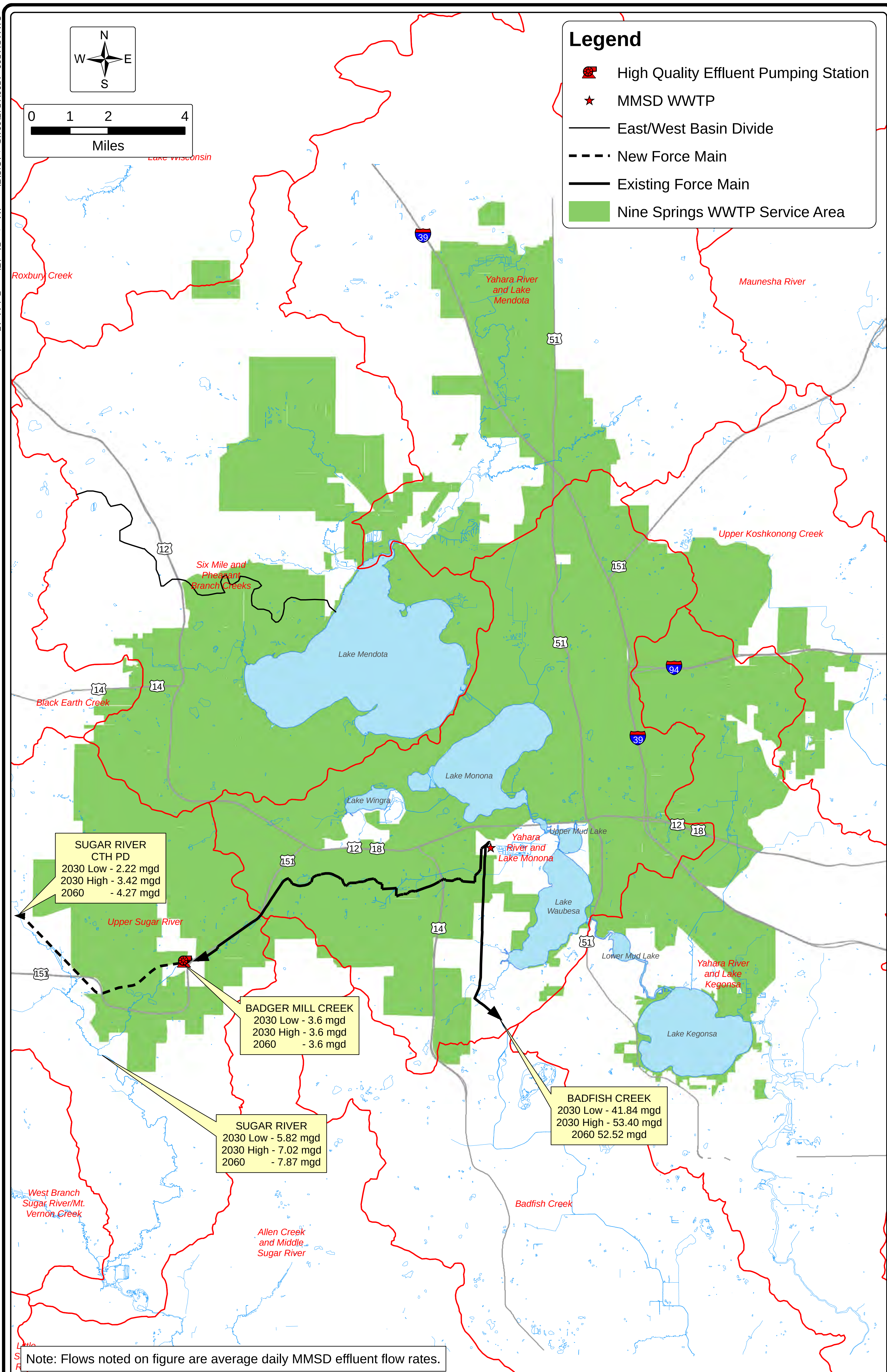

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FIGURE 7-4

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ALTERNATIVE MP-1D

MMSD 50-YEAR MASTER PLAN

Table 7-1 Component Projects of Master Planning Alternative MP-1

Facility Name	Component Project	MP-1A	MP-1B	MP-1C	MP-1D
PS11	Condition improvement and firm pumping capacity expansion. The cost for this improvement is already budgeted and included in the scheduled PS 11 condition improvement project.	Yes	Yes	Yes	Yes
PS11	Install a new 36” diameter force main parallel to the existing force main.	Yes	Yes	Yes	Yes
PS12	Condition improvement and firm pumping capacity expansion. The cost for this improvement is already budgeted and included in the scheduled PS 12 condition improvement project.	Yes	Yes	Yes	Yes
PS17	Firm pumping capacity expansion to average daily flow of 4.37 mgd and peak flow of 13.6 mgd.	Yes	Yes	Yes	Yes
PS17	Force main expansion	Yes	Yes	Yes	Yes
NSVI	Expand capacity of interceptor section from PS11 to PS12.	Yes	Yes	Yes	Yes
NSVI	Expand capacity of section upstream of PS12.	Yes	Yes	Yes	Yes
NSVI	Relining the entire length of the NSVI	Yes	Yes	Yes	Yes
Badger Mill Creek Effluent Pumps	Expand the current average effluent pumping capacity to 7.9 mgd.	No	Yes	Yes	Yes
Sugar River Effluent Pumping Station	Construction of a new pumping station at the vicinity of the Badger Mill Creek outfall with an average capacity of 4.3 mgd.	No	No	Yes	Yes
Sugar River Force Main	Construction of a new force main for the new effluent pumping station to downstream of confluence of Badger Mill Creek and Sugar River.	No	No	Yes	No
Sugar River Headwaters Force Main	Construction of a new force main for the new effluent pumping station to the Sugar River headwaters near CTH PD.	No	No	No	Yes
High Quality Effluent Treatment facility at the NSWTP	Construction of a new high quality effluent treatment facility at the NSWTP with capacities of 4.3 mgd (DAF) and 13.7 mgd (DMF). The facility would include processes for effluent polishing to meet the 5 mg/L limit for BOD_5 and TSS. The facility would also be designed to meet a potential 0.075 mg/L TP limit and a 3 mg/L Total N limit.	No	Yes	Yes	Yes

The Alternative MP-2 is based on pumping all of the wastewater flows generated within the service area of PS 17 to a new satellite treatment plant in the Sugar River watershed for treatment. This alternative includes the following two variations:

- **Alternative MP-2A –** Construction of a new advanced secondary wastewater treatment plant by 2020 with discharge to the main branch of the Sugar River downstream of the confluence with Badger Mill Creek. Average day flows in 2060 will be 4.3 mgd.

- **Alternative MP-2B –** Construction of a new advanced secondary wastewater treatment plant by 2020 with discharges to the main branch of the Sugar River downstream of the confluence with Badger Mill Creek and to the headwaters of Sugar River near CTH PD northwest of Verona. Average daily flows in 2060 will be 2.5 mgd and 1.8 mgd, respectively, at these two locations.

For both of these planning alternative variations, 3.6 mgd of treated effluent would continue to be pumped from the NSWTP to Badger Mill Creek. Layouts for each of these alternatives are shown in Figure 7-5 and 7-6. The major component projects for these two alternatives are listed in Table 7-2.

Table 7-2 Component Projects of Master Planning Alternative MP-2

Facility Name	Component Project	MP-2A	MP-2B
Sugar River WWTP	Construction of a new Sugar River WWTP with capacities of 4.3 mgd (DAF), and 13.7 mgd (DMF). Facility would include processes for effluent polishing to meet the 5 mg/L limit for BOD_5 and TSS. Facility would also be designed to meet a potential 0.075 mg/L TP limit and a 3 mg/L Total N limit.	Yes	Yes
PS17	Firm pumping capacity expansion to average daily flow of 4.37 mgd and peak flow of 13.6 mgd.	Yes	Yes
PS17	Force main from PS 17 to Sugar River WWTP	Yes	Yes
Effluent Pumping Station	Construction of a effluent pumping station to pump flow to the headwaters of the Sugar River near CTH PD	No	Yes
Effluent Force Main	Construction of an effluent force main to convey flow to the headwaters of the Sugar River near CTH PD	No	Yes
PS11	Pumping station condition improvement. The cost for this improvement is already budgeted and included in the scheduled PS 11 condition improvement project.	Yes	Yes
PS12	Pumping station condition improvement. The cost for this improvement is already budgeted and included in the scheduled PS 12 condition improvement project.	Yes	Yes
NSVI	Relining the entire length of the NSVI	Yes	Yes

For Alternatives MP-2A and MP-2B, two options are available for biosolids disposal. First, the waste biosolids can be hauled to the NSWTP for anaerobic digestion and then used for land application or other utilization. Second, onsite anaerobic digestion and 180 days of biosolids storage can be constructed at the new Sugar River plant. The biosolids can be used for land application in the vicinity of the plant site.

7.3.2. Long-Term Planning Alternatives

Long-term alternatives are those planning alternatives that cannot be implemented soon enough to provide relief in the conveyance system; however, they remain potentially viable options beyond the year 2030 for mitigating inter-basin transfers of water, or providing high quality effluent for reuse options. Due to growing demands on available groundwater supplies and the long-term goal of stabilizing the groundwater aquifer

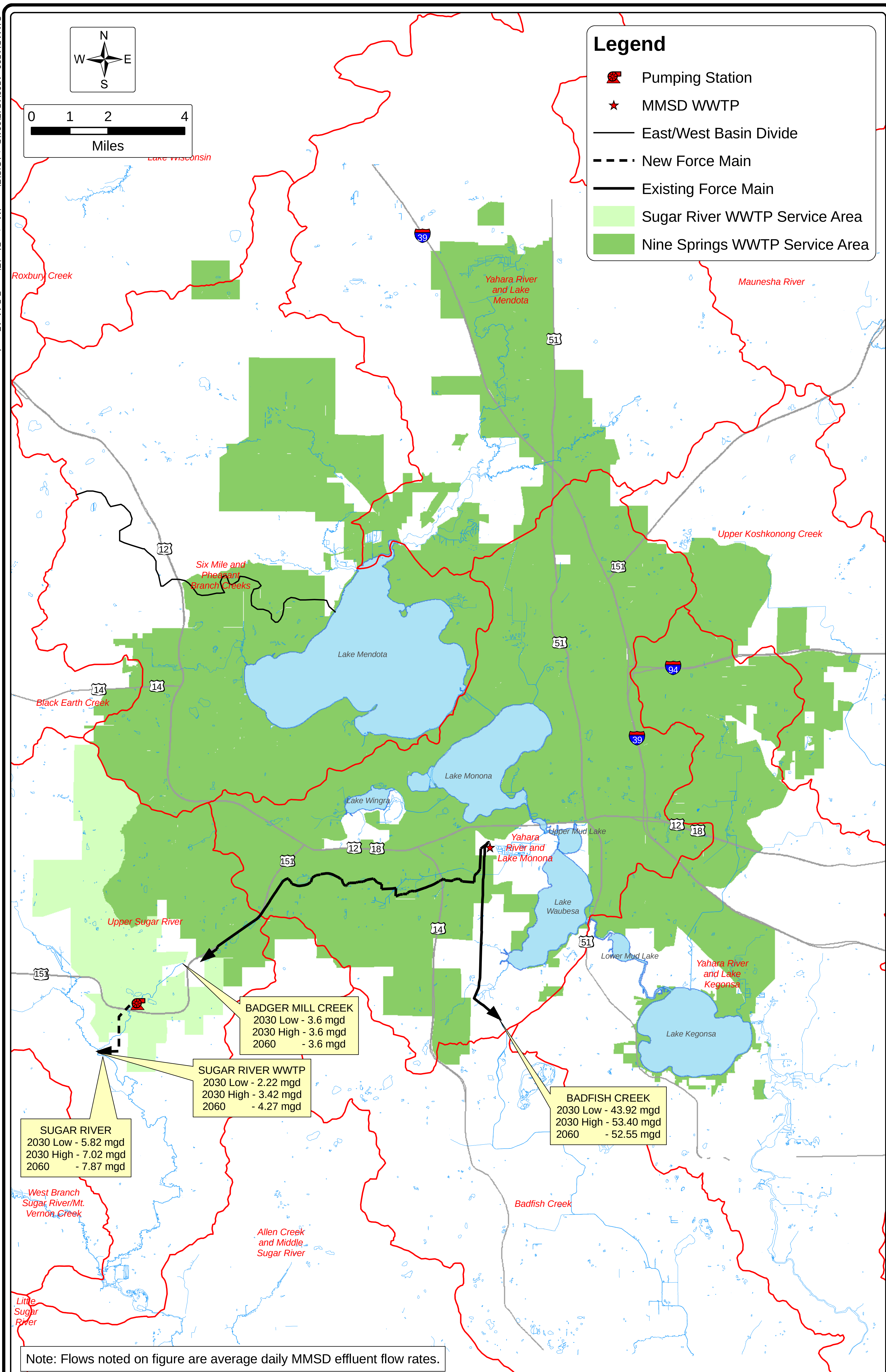

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FIGURE 7-5

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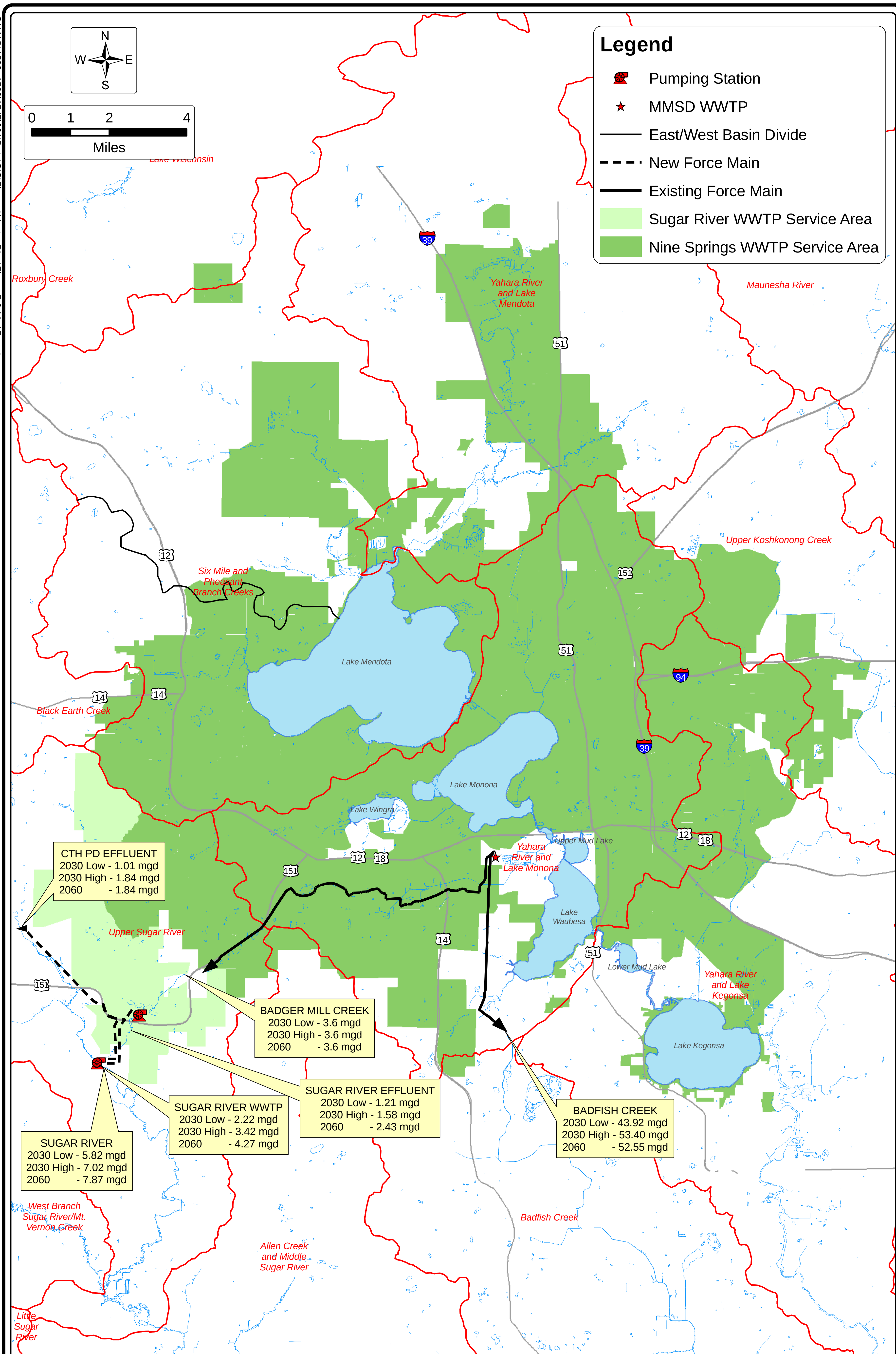
ALTERNATIVE MP-2A

MMSD 50-YEAR MASTER PLAN



Note: Flows noted on figure are average daily MMSD effluent flow rates.

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FIGURE 7-6

STRAND ASSOCIATES, INC. ENGINEERS

MALCOLM PIRNIE

MMSD

ALTERNATIVE MP-2B

MMSD 50-YEAR MASTER PLAN

operating level in the Dane County area, high quality effluent utilization could be a promising way to solve these issues in the future, especially if population growth occurs as expected. The following two long-term alternatives emphasizing effluent reuse were selected for further evaluation. These two alternatives have potential to be implemented after 2030 and provide high quality effluent to various locations for reuse options and to mitigate inter-basin transfer of water.

- **Alternative MP-3 – Centralized High Quality Effluent Treatment & Distribution:** This alternative would include construction of facilities at the NSWTP that would produce a high quality effluent for use in various applications, including stream flow augmentation, infiltration, industrial reuse, or turf irrigation. It also includes a new effluent pumping station and effluent force main to convey the effluent from Nine Springs to a point of use near PS 13.
- **Alternative MP-4 – Decentralized High Quality Effluent Treatment Facilities**:

  This alternative would include construction of facilities northeast of the Dane County Regional Airport. The new treatment plant would receive wastewater flows tributary to PS13 or both PS13 and PS14. Effluent from this facility could be used for stream flow augmentation to Starkweather Creek, wetland restoration at Cherokee Marsh, groundwater infiltration, industrial reuse water or turf irrigation.

Due to the long planning horizon, specific effluent reuse projects cannot be clearly defined at this stage. However, the MMSD would like to take a proactive approach to study the potential economic, technical and environmental factors that may impact implementation of future effluent reuse programs. To facilitate the study, high quality effluent facilities with capacities of 4 mgd and 10 mgd, representing both small and medium sized effluent reuse programs, were chosen for this evaluation. The PS13 service area was selected as the location for the reuse facility. Potential reuse applications could include industrial reuse, stream flow augmentation, turf irrigation, and groundwater infiltration. Layouts for the reuse facilities are shown in Figure 7-7 and 7-8.

Alternative MP-3 would include construction of high quality effluent treatment facilities with a capacity of either 4 mgd (Alternative MP-3A) or 10 mgd (Alternative MP-3B). Alternative MP-3A is directly comparable to Alternative MP-4A, and Alternative MP-3B is directly comparable to Alternative MP-4B.

Planning alternatives MP-3A and MP-3B would include the additional treatment facilities at the NSWTP to produce high quality effluent as well as the pumping facilities to return the high quality effluent to the PS 13 site northeast of the Dane County Regional Airport.

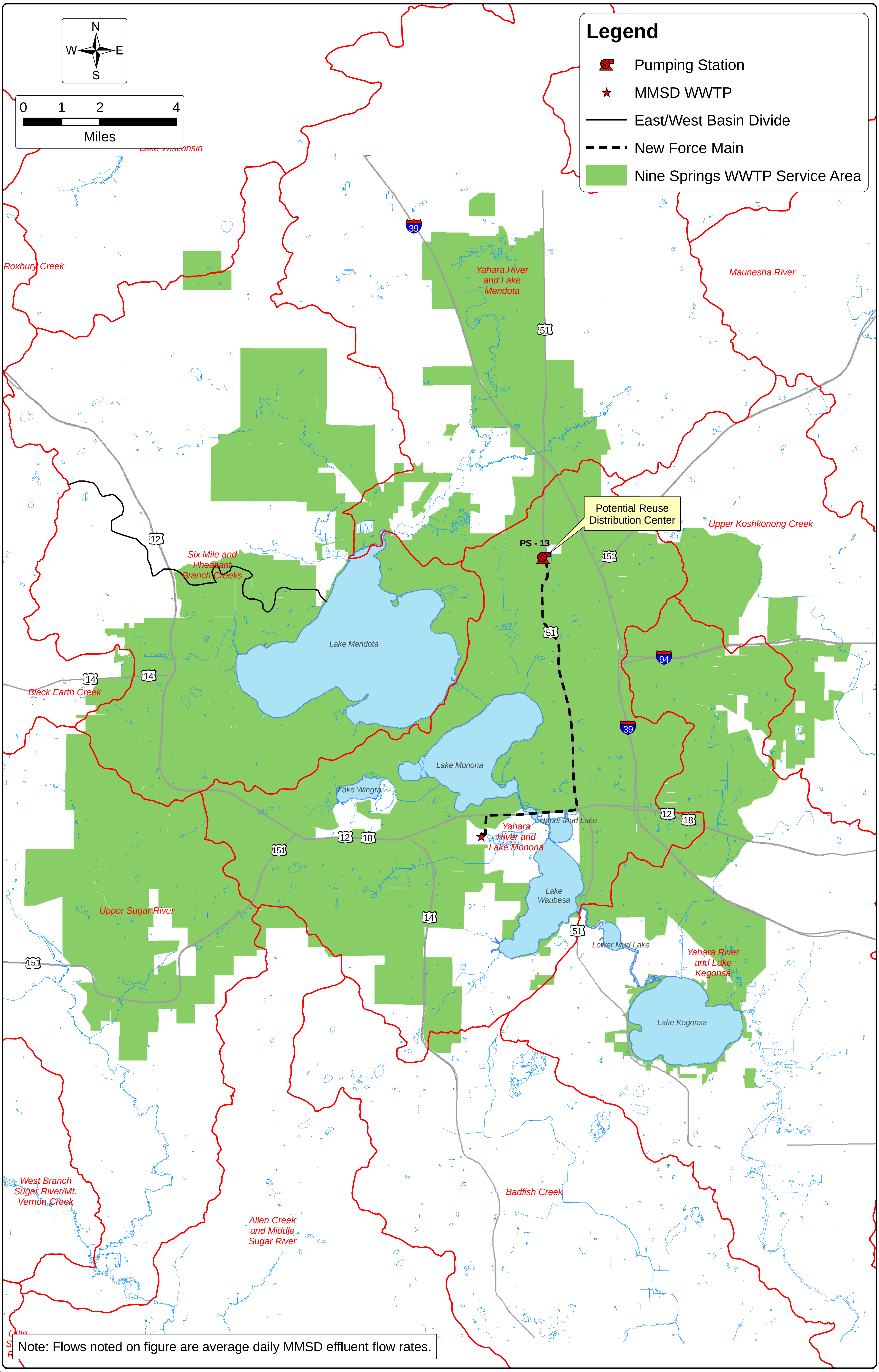

FIGURE 7-7

ALTERNATIVE MP-3A MP-3B

MMSD 50-YEAR MASTER PLAN

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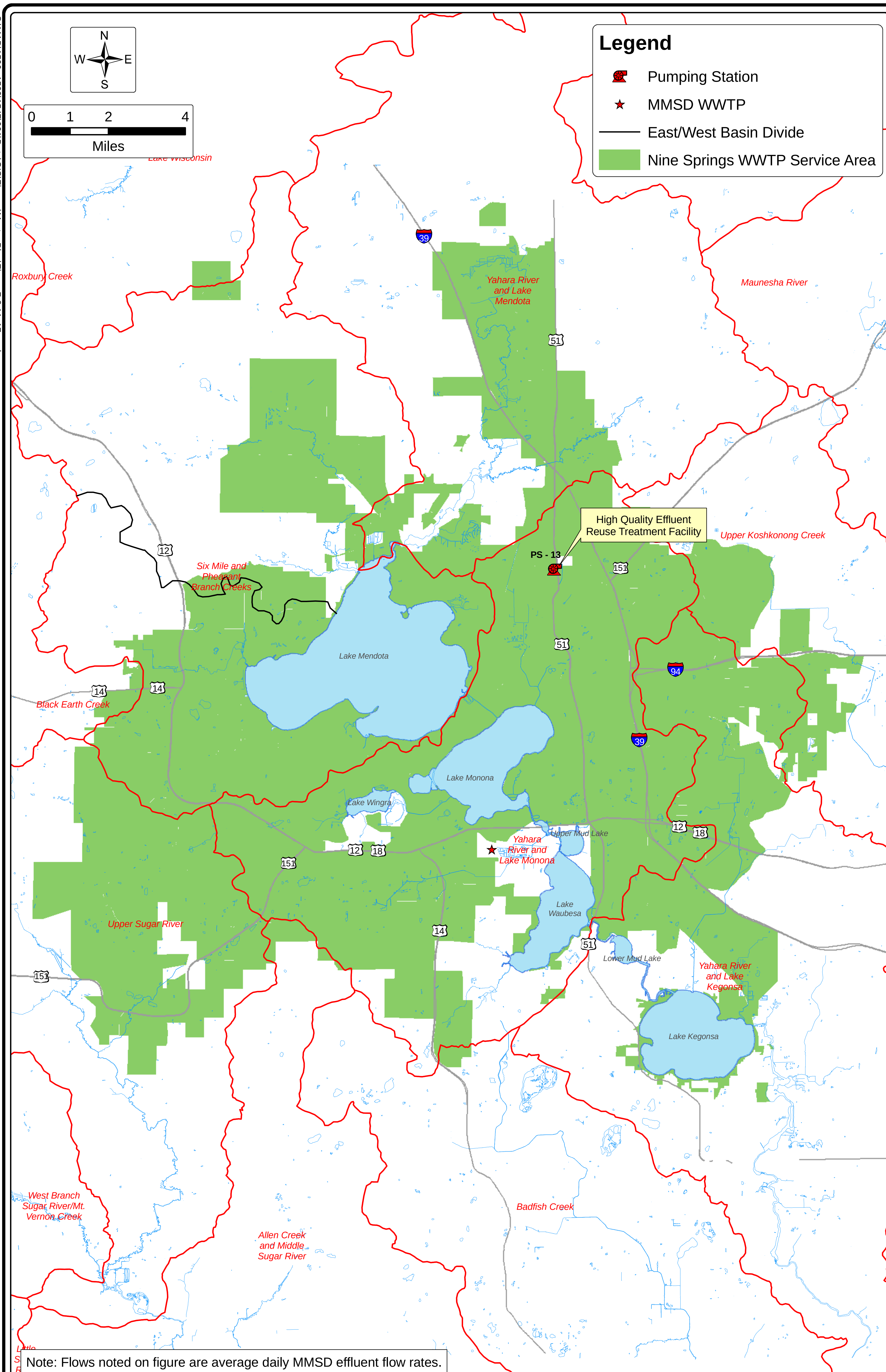

FIGURE 7-8

STRAND ASSOCIATES, INC. ENGINEERS

MALCOLM PIRNIE

MMSD

ALTERNATIVE MP-4A AND MP-4B

MMSD 50-YEAR MASTER PLAN

Actual implementation of effluent reuse may not require water quality as stringent as would be produced by a high quality effluent facility if the end use water quality requirements are lower, or if the end user provides additional treatment that would meet their specific needs and comply with Wisconsin Administrative Code requirements. The component projects included in this alternative are listed in Table 7-3.

Table 7-3 Component Projects of Master Planning Alternative MP-3

Facility Name	Component Project
High Quality Effluent Treatment Facilities at NSWTP	Construction of new high quality effluent facilities with capacity of either 4 mgd (Alternative MP-3A) or 10 mgd (Alternative MP-3B) at the NSWTP. Facilities would include processes for effluent polishing to meet the 5 mg/L limit for BOD_5 and TSS. Facility would also be designed to meet a potential 0.075 mg/L TP limit and a 3 mg/L Total N. The level of treatment will be further determined after the treated effluent utilizations become better defined in the future.
Effluent Return Pumping Station	Construction of a new pumping station at the NSWTP to return high quality effluent to PS 13 area.
Effluent Return Force Main	Construction of a new force main to return high quality effluent to PS 13 area.

Alternative MP-4 includes construction of stand-alone high quality effluent treatment plant with a capacity of either 4 mgd (Alternative MP-4A) or 10 mgd (Alternative MP-4B). Alternative MP-4A is directly comparable to Alternative MP-3A, and Alternative MP-4B is directly comparable to Alternative MP-3B. Actual implementation of effluent reuse may not require water quality as stringent as would be produced by a high quality effluent facility if the end use would not require such a high quality or if the end user would provide additional treatment that would meet their specific needs and comply with Wisconsin Administrative Code requirements. The component projects included in this alternative are listed in Table 7-4.

Table 7-4 Component Projects of Master Planning Alternative MP-4

Facility Name	Component Project
Starkweather Creek WWTP	Construction of a new high quality effluent treatment plant with a capacity of 4 mgd. The plant would include processes for effluent polishing to meet the 5 mg/L limit for BOD_5 and TSS. Facility would also be designed to meet a potential 0.075 mg/L TP limit and a 3 mg/L Total N limit.
PS13 and PS14 Service Area WWTP	Construction of a new high quality effluent treatment plant with a capacity of 10 mgd. The plant would include processes for effluent polishing to meet the 5 mg/L limit for BOD_5 and TSS. Facility would also be designed to meet a potential 0.075 mg/L TP limit and a 3 mg/L Total N limit.

To determine the optimum near-term and long-term planning alternative(s) to be implemented during the 50 year planning period, these alternatives were evaluated and ranked with the ranking criteria developed in the following chapter.

8. ALTERNATIVE RANKING CRITERIA

8.1. Background

Several meetings and workshops were conducted with the MMSD and TAC to identify applicable evaluation criteria and determine their levels of importance. A survey was also conducted to solicit opinions on evaluation criteria from TAC members and MMSD staff. Details of the master planning ranking criteria development are attached in Appendix H, TM8 – Planning Alternative Evaluation Criteria.

The identified planning criteria are categorized into the following 4 groups:

- Economic criteria

  The impacts the planning alternatives have on the economic conditions of the MMSD's stakeholders and on the MMSD's own financial performance.

- Technical criteria

  The impacts the planning alternatives have on the technical aspects of the MMSD operation, such as the ease of maintenance, system reliability, system flexibility, etc.

- Social criteria

  The impacts the planning alternatives have on the social systems within which the MMSD operates, including public acceptance, staffing requirements, etc.

- Environmental criteria

  The impacts the planning alternatives have on natural systems, including ecosystems, land, air and water, and the alternatives' carbon footprints.

These evaluation criteria incorporate the major elements of typical sustainability evaluations of water and wastewater utilities. Adoption of these criteria in the evaluation process will allow evaluating and ranking planning alternatives from a multiple dimension perspective.

All the identified evaluation criteria were described and discussed. Levels of importance (Low, Medium and High) were then assigned to each of 10 criteria based on the combined efforts of the TAC, MMSD and the consultant.

8.2. Economic Criteria

Life Cycle Cost

An MMSD mandate is to provide cost-effective wastewater conveyance, treatment and biosolids management services. Life cycle cost is used as a basis for making economic comparisons between alternatives. The life cycle cost is the total discounted dollar cost of owning, operating, maintaining, and disposing of the planning alternatives over the 50 year planning period. The life cycle cost includes the components listed below:

- Initial Capital Costs

  Initial capital costs include the purchase of land, buildings, equipment, and construction activities to bring all the component projects associated with a planning alternative to a fully operable status. Initial costs do not include labor costs except for the labor used for construction.

- 50-Year Replacement Cost

  All of the costs associated with the replacement of the structures, equipment, and other major components of the facilities included in a planning alternative to maintain the proper operation efficiency and physical conditions of the facilities during the 50 year planning period.

- Annual Operation/Maintenance Costs

  The annual operation/maintenance costs are composed of all the expenses including labor, materials, and other expenses for maintaining day-to-day facility functions and preserving the operating efficiency and physical condition of the facilities included in a planning alternative.

8.3. Technical Criteria

Regulatory Constraints

Alternatives must meet all regulatory requirements. However, the regulatory requirements associated with any given planning alternative may be easier or more difficult to meet, depending on a number of factors. For example, the regulatory requirements associated with an effluent discharge to an Exceptional Resource Water (ERW) or to a lake would be more stringent than those associated with discharge to a warm water stream.

Proven Effectiveness

The selected alternative(s) must be able to provide reliable service during the planning period. This criterion is used to evaluate planning alternatives for their reliability in providing required service. For example, fifteen years ago, biological phosphorous removal was not as proven a technology for removing phosphorous as chemical addition. As such, it would not have been considered to be as well "proven" as chemical addition. The proven ability of an alternative to meet the regulatory goals will need to be considered.

Flexibility, Expandability, and Compatibility

The selected alternative(s) must have the ability over time to be easily connected with the existing system. This allows for ease of construction and financial burden to the MMSD. The selected alternative(s) must be compatible with the existing collection system and treatment facilities, and maximize continued use of the existing facilities. The selected alternative(s) must also be compatible with other regional planning goals. This criterion is used to rank alternatives for their potential to meet the following requirements:

- Can the alternative be readily modified to meet potential future needs such as re-routing wastewater, meeting more stringent future permit limits and regulations, etc?
- Can it be readily expanded to meet future flows and loadings?
- Is the alternative compatible with the existing collection system and treatment facilities?
- Does it maximize continued use of existing facilities?
- Can it be easily connected to existing system over time?
- Is it compatible with other regional planning goals?

Ease of Operation

Some alternatives may be more difficult or challenging to operate. For example, operation of a facility utilizing membrane filtration may be more difficult than operating the MMSD's current facility. The selected alternative(s) must consider the level of complexity involved in operating the facilities included in the planning alternatives. This criterion will be used to rank all planning alternatives for efforts involved in the facility operation.

8.4. Social Criteria

Public Acceptance

Public acceptance has significant impacts on the implementation of planning alternatives. The selected planning alternative(s) must have the support of the public or a plan must be developed to gain this support. This criterion ranks all the planning alternatives for the likelihood of being accepted or resisted by the public.

Staffing Implications

Alternatives may have different staffing implications, both in terms of staffing level and required skills. For example, operation of multiple plants may be more labor intensive than operation of a centralized system. In addition, operating an advanced treatment system to produce high quality effluent may require a more skilled workforce than operating a secondary treatment system. This criterion will be used to rank all planning alternatives for these staffing requirements.

8.5. Environmental Criteria

Maintains Watershed Balance

Stream flow augmentation and water balancing within the watershed are issues to address in the Master Plan. The volumes and locations at which the MMSD discharges its effluent based on recommendations in the Master Plan will have significant impacts on sustaining water levels in streams and aquifers, and maintaining watershed balancing throughout the watersheds. This criterion will be used to rank all the planning alternatives for their potential in augmenting low flow in streams and alleviating imbalanced inter-watershed water transfer.

Opportunities for Effluent Reuse

One of the potential outcomes of the Master Plan is to maximize the use of treated effluent as a resource. Effective effluent reuse could reduce the need for groundwater withdrawals from the Madison area aquifer and improve the sustainability in water resource utilization in the Madison and the Dane County areas. The available effluent reuse options include:

- Turf irrigation
- Groundwater recharge
- Industrial water use
- Other uses

Some alternatives may present greater opportunity to beneficially reuse effluent because of location of facilities, level or treatment, etc. This criterion will be used to rank all the planning alternatives for their potential to beneficially reuse effluent.

Carbon Footprint

Carbon footprint is a measure of the impact that the planning alternatives have on the environment in terms of the amount of the greenhouse gases produced. It will be evaluated for the utilization of electricity, natural gas, gasoline, etc. Some alternatives may have a larger carbon footprint than others. This criterion will be used to rank all the planning alternatives for their magnitude of carbon emissions.

8.6. Determination of Level of Importance

The levels of importance for all planning alternatives were determined based on independent rankings by the TAC, MMSD and the consultant. Each evaluation criterion was assigned a weighting score ranging from 1 to 50 according to their relative importance. The more important an evaluation criterion is, the higher score it was assigned. To force a differentiation among the criteria, the sum of the weighting scores for all 10 criteria was set to 100. The importance scores from the TAC, MMSD staff and the consultant team were then averaged to calculate the final scores for all evaluation criteria. The details of the process for determining the relative importance of all evaluation criteria are shown in Table 8-1.

Table 8-1. Planning Alternative Evaluation Criteria

No.	Evaluation Criteria	TAC Ranking Score	MMSD Ranking Score	Consultant Ranking Score	Average
1	Life Cycle Cost	15	33	30	26
2	Public Acceptance	10	14	15	13
3	Watershed Balance	12	10	10	11
4	Flexibility/Expandability/Compatibility	12	9	7	9
5	Effluent Reuse	13	7	8	9
6	Regulatory Constraints	8	9	10	9
7	Proven Effectiveness	10	7	8	8
8	Carbon Footprint	9	3	3	5
9	Ease of Operation	6	5	4	5
10	Staffing Implications	5	4	5	5

The evaluation criteria and their respective levels of importance were then used to evaluate and rank the near-term and long-term master planning alternatives to determine the optimum alternatives for implementation during the 50 year master planning period.

9. ALTERNATIVE RANKING & EVALUATION

9.1. Background

All identified near-term and long-term alternatives were ranked and evaluated with the evaluation criteria developed previously. Alternatives were assigned a relative score of 1 to 10 for each criterion by the project team according to criteria defined in Table 9-1. Then, the score for each criterion was multiplied by the level of importance for that criterion. For example, if the life cycle cost criterion (with a level of importance of 26) received a score of 10, the weighted score for that criterion is 26 x 10 = 260. Total scores for each alternative were then calculated by adding the weighted score for each criterion for that alternative. Planning alternatives with higher total scores represent more favorable alternatives than those with lower scores. Details of the master planning alternative ranking and evaluation are provided in Appendix I, TM9 – Planning Alternative Ranking and Evaluation.

Table 9-1 Planning Alternative Evaluation Criteria Definitions

Evaluation Criteria	Definition
Life Cycle Cost	Life cycle cost is the total discounted dollar cost of owning, operating, and maintaining the planning alternatives. A ranking score of "10" for this criterion represents the lowest life cycle cost.
Public Acceptance	Public acceptance is the support level of the public to a planning alternative. A ranking score of "10" for this criterion represents the highest level of public acceptance.
Watershed Balance	Watershed balance is the potential of a planning alternative to mitigate imbalanced inter-basin water transfer. A ranking score of "10" for this criterion represents the highest potential to mitigate imbalanced inter-basin water transfer.
Flexibility Expandability Compatibility	This criterion is used to gauge alternatives for their potential to be readily modified or expanded to meet future needs, and their compatibility with the existing system. A ranking score of "10" for this criterion represents the highest level of flexibility, expandability and compatibility.
Effluent Reuse	This criterion is used to evaluate all planning alternatives for their potential to provide treated effluent utilization. A ranking score of "10" for this criterion represents the highest potential for effluent reuse.
Regulatory Constraints	This criterion is used to evaluate the potential regulatory constraints that may affect the implementation of the planning alternative. A ranking score of "10" for this criterion represents the lowest level of regulatory constraints.
Proven Effectiveness	This criterion is used to evaluate planning alternatives for their proven reliability in providing the required level of conveyance and treatment. A ranking score of "10" for this criterion represents the highest level of effectiveness.
Carbon Footprint	This criterion is used to evaluate planning alternatives for their impacts on the environment in terms of the amount of the greenhouse gases produced. A ranking score of "10" for this criterion represents the lowest carbon footprint.
Ease of Operation	This criterion is used to rank the efforts involved in the facility operation. A ranking score of "10" for this criterion represents the relatively easiest operation.

Evaluation Criteria	Definition
Staffing Implications	This criterion is used to rank all planning alternatives for the staffing requirements in terms of staffing level and required skills. A ranking score of "10" for this criterion represents staffing level and skill requirements most similar to or less than the current requirements.

9.2. Near-Term Alternative Evaluation

In this section, all four variations of the Alternative MP-1 are compared to the two variations of the Alternative MP-2 to determine the planning alternative to be implemented between 2010 and 2030. Life cycle costs were calculated for these planning alternatives based on the costs of construction, operation and maintenance costs, replacement/rehabilitation costs, and salvage values. Operation costs include the costs for pumping wastewater to the treatment plants and pumping effluent from the plants to the various discharge locations, and the operational costs for high quality effluent facilities. The results were presented in Tables 9-2 and 9-3 and were used for subsequent planning alternative evaluation and ranking.

Table 9-2 Alternative MP-1 Life Cycle Cost Analysis

Item	MP-1A	MP-1B	MP-1C	MP-1D
Initial Capital Costs	$50,881,000	$68,581,000	$75,068,000	$75,068,000
Life Cycle O/M Costs	$18,881,000	$30,843,000	$32,901,000	$33,029,000
Life Cycle Costs for Facility Improvement & Replacement	$2,758,000	$7,313,000	$7,698,000	$7,698,000
Salvage Value	($3,298,000)	($3,702,000)	($4,115,000)	($4,115,000)
50-Year Total Present Value	$69,222,000	$103,034,000	$111,552,000	$111,680,000

Table 9-3 Alternative MP-2 Life Cycle Cost Analysis

Item	MP-2A	MP-2B
Initial Capital Costs	$67,905,000	$72,305,000
Life Cycle O/M Costs	$32,407,000	$34,036,000
Life Cycle Costs for Facility Improvement & Replacement	$14,028,000	$14,414,000
Salvage Value	($2,261,000)	($2,516,000)
50-Year Total Present Value	$112,079,000	$118,239,000

The implementation dates and rankings for planning alternatives MP-1 and MP-2 are summarized in Tables 9-4 and 9-5.

Table 9-4 Near-Term Master Planning Alternative Evaluation

Project Variable	**Sugar River Watershed Service Alternatives**					
	1A	**1B**	**1C**	**1D**	**2A**	**2B**
Treatment Plant Location	Nine Springs	Nine Springs	Nine Springs	Nine Springs	Sugar River	Sugar River
High Quality Effluent Treatment Design DAF (mgd)	0.0	4.3	4.3	4.3	4.3	4.3
Effluent Discharge Location	Badger Mill Creek	Badger Mill Creek	Sugar River	Headwaters of Sugar River	Sugar River	Headwaters of Sugar River
NSVI Improvements - Year Required						
PS 11 Firm Pumping Capacity	2015	2015	2015	2015	2045	2045
PS 11 Major Condition Upgrade	2015	2015	2015	2015	2015	2015
PS 11 Force Main Capacity	2025	2025	2025	2025	2051	2051
PS 12 Firm Pumping Capacity	2015	2015	2015	2015	2025	2025
PS 12 Major Condition Upgrade	2015	2015	2015	2015	2015	2015
PS 12 Force Main Capacity	>2060	>2060	>2060	>2060	>2060	>2060
PS 17 Firm Capacity	2015	2015	2015	2015	2020	2020
PS 17 Force Main Capacity	2015	2015	2015	2015	2020	2020
NSVI from PS 11 to PS 12	2020	2020	2020	2020	>2060	>2060
NSVI above PS 12	2020	2020	2020	2020	>2060	>2060
Effluent Return Facilities Required						
To Sugar River South of Verona	No	No	Yes	No	No	No
To Sugar River Headwaters	No	No	No	Yes	No	Yes
To Badger Mill Creek	No	Yes	Yes	Yes	No	No
To Badfish Creek	No	No	No	No	No	No

Table 9-5 Near-Term Master Planning Alternative Evaluation

Project Variable	**Sugar River Watershed Service Alternatives**					
	1A	**1B**	**1C**	**1D**	**2A**	**2B**
Treatment Plant Location	Nine Springs	Nine Springs	Nine Springs	Nine Springs	Sugar River	Sugar River
Evaluation Criteria						
Life Cycle Cost (in millions)	69.2	103.0	111.6	111.7	112.1	118.2
Relative Life Cycle Cost	1.0	1.5	1.6	1.6	1.6	1.7
Ranking Score	10.0	6.7	6.2	6.2	6.2	5.9
Level of Importance	26					
Weighted Score	**260**	**175**	**161**	**161**	**160**	**152**

Project Variable	Sugar River Watershed Service Alternatives					
	1A	1B	1C	1D	2A	2B
Public Acceptance						
Ranking Score	8	9	6	5	4	3
Level of Importance	13					
Weighted Score	**104**	**117**	**78**	**65**	**52**	**39**
Watershed Balance						
Ranking Score	5	8	9	10	9	10
Level of Importance	11					
Weighted Score	**55**	**88**	**99**	**110**	**99**	**110**
Flexibility/Expandability/Compatibility						
Ranking Score	5	6	8	8	8	8
Level of Importance	9					
Weighted Score	**45**	**54**	**72**	**72**	**72**	**72**
Effluent Reuse						
Ranking Score	6	7	8	8	7	8
Level of Importance	9					
Weighted Score	**54**	**63**	**72**	**72**	**63**	**72**
Regulatory Constraints						
Ranking Score	9	7	4	3	4	3
Level of Importance	9					
Weighted Score	**81**	**63**	**36**	**27**	**36**	**27**
Proven Effectiveness						
Ranking Score	8	6	4	4	4	4
Level of Importance	8					
Weighted Score	**64**	**48**	**32**	**32**	**32**	**32**
Carbon Footprint						
Ranking Score	8	6	5	5	10	9
Level of Importance	5					
Weighted Score	**40**	**30**	**25**	**25**	**50**	**45**
Ease of Operation						
Ranking Score	10	7	6	6	3	2
Level of Importance	5					
Weighted Score	**50**	**35**	**30**	**30**	**15**	**10**
Staffing Implications						
Ranking Score	10	9	8	8	5	5
Level of Importance	5					
Weighted Score	**50**	**45**	**40**	**40**	**25**	**25**
Total	**803**	**718**	**645**	**634**	**604**	**584**

9.3. Long-Term Planning Alternative Evaluation (4 MGD Capacity)

Planning alternative MP-3A includes the 4 mgd additional treatment facilities at the NSWTP to produce high quality effluent as well as the pumping facilities to return the high quality effluent to the PS 13 site northeast of the Dane County Regional Airport. Planning alternative MP-4A includes the 4 mgd stand-alone high quality effluent treatment plant at the PS 13 service area northeast of the Dane County Regional Airport.

Life cycle costs were calculated for these two planning alternative based on the costs of construction, operating and maintenance costs, replacement/rehabilitation costs, and salvage values. The results are presented in Tables 9-6 and are used for subsequent planning alternative evaluation and ranking.

Table 9-6 Alternative MP-3A/MP-4A Life Cycle Cost Analysis

Item	Alternative MP-3A	Alternative MP-4A
Initial Capital Costs	$27,100,000	$40,000,000
Life Cycle O/M Costs	$20,024,000	$26,167,000
Life Cycle Costs for Facility Improvement & Replacement	$4,631,000	$10,292,000
Salvage Value	($1,103,000)	($912,000)
50-Year Total Present Value	$50,652,000	$75,547,000

The rankings for planning alternatives MP-3A and MP-4A are summarized in Tables 9-7 and 9-8.

Table 9-7 Long-Term Master Planning Alternative Summary (4 MGD Plant)

	High Quality Effluent Treatment Plant Alternatives	
Project Variable	**MP-3A**	**MP-4A**
Treatment Plant Location	Nine Springs	Northeast of the Dane County Regional Airport
Treatment Plant Design ADF (mgd)	4.0	4.0
Effluent Discharge Location	Yahara River Watershed	Yahara River Watershed
Effluent Return Pump Capacity (mgd)	4.0	0
Effluent Return Force Main Capacity (mgd)	10.0	0

Table 9-8 Long-Term Master Planning Alternative Evaluation (4 MGD Plant)

Project Variable	**High Quality Effluent Treatment Plant Alternatives**	
	MP-3A	**MP-4A**
Treatment Plant Location	Nine Springs	Northeast of the Dane County Regional Airport
Evaluation Criteria		
Life Cycle Cost (in millions)	50.7	75.5
Relative Life Cycle Cost	1.0	1.5
Level of Importance	26	
Weighted Score	**260**	**175**
Public Acceptance		
Ranking Score	6	4
Level of Importance	13	
Weighted Score	**78**	**52**
Watershed Balance		
Ranking Score	8	8
Level of Importance	11	
Weighted Score	**88**	**88**
Flexibility/Expandability/Compatibility		
Ranking Score	7	5
Level of Importance	9	
Weighted Score	**63**	**45**
Effluent Reuse		
Ranking Score	8	6
Level of Importance	9	
Weighted Score	**72**	**54**
Regulatory Constraints		
Ranking Score	3	3
Level of Importance	9	
Weighted Score	**27**	**27**
Proven Effectiveness		
Ranking Score	4	4
Level of Importance	8	
Weighted Score	**32**	**32**
Carbon Footprint		
Ranking Score	5	6
Level of Importance	5	
Weighted Score	**25**	**30**
Ease of Operation		
Ranking Score	5	2
Level of Importance	5	
Weighted Score	**25**	**10**
Staffing Implications		
Ranking Score	5	3
Level of Importance	5	
Weighted Score	**25**	**15**
Total	**695**	**528**

9.4. Long-Term Planning Alternative Evaluation (10 MGD Capacity)

Planning alternative MP-3B includes the 10 mgd additional treatment facilities at the NSWTP to produce high quality effluent as well as the pumping facilities to return the high quality effluent to the PS 13 site northeast of the Dane County Regional Airport. Planning alternative MP-4B includes the 10 mgd stand-alone high quality effluent treatment plant at the PS 13 service area northeast of the Dane County Regional Airport.

Life cycle costs were calculated for these two planning alternative based on the costs of construction, operating and maintenance costs, replacement/rehabilitation costs, and salvage values. The results are presented in Tables 9-9 and are used for subsequent planning alternative evaluation and ranking.

Table 9-9 Alternative MP-3B/MP-4B Life Cycle Cost Analysis

Item	Alternative MP-3B	Alternative MP-4B
Initial Capital Costs	$45,500,000	$80,000,000
Life Cycle O/M Costs	$47,949,000	$56,670,000
Life Cycle Costs for Facility Improvement & Replacement	$8,362,000	$20,584,000
Salvage Value	($1,730,000)	($1,825,000)
50-Year Total Present Value	$100,081,000	$155,429,000

The rankings for planning alternatives MP-3B and MP-4B are summarized in Tables 9-10 and 9-11.

Table 9-10 Long-Term Master Planning Alternative Summary (10 MGD Plant)

	High Quality Effluent Treatment Plant Alternatives	
Project Variable	MP-3B	MP-4B
Treatment Plant Location	Nine Springs	Northeast of the Dane County Regional Airport
Treatment Plant Design ADF (mgd)	10.0	10.0
Effluent Discharge Location	Yahara River Watershed	Yahara River Watershed
Effluent Return Pump Capacity (mgd)	10.0	0
Effluent Return Force Main Capacity (mgd)	10.0	0

Table 9-11 Long-Term Master Planning Alternative Evaluation (10 MGD Plant)

Project Variable	High Quality Effluent Treatment Plant Alternatives	
	MP-3B	MP-4B
Treatment Plant Location	Nine Springs	Northeast of the Dane County Regional Airport
Evaluation Criteria		
Life Cycle Cost	100.1	155.4
Relative Life Cycle Cost	1.0	1.6
Level of Importance	26	
Weighted Score	**260**	**167**
Public Acceptance		
Ranking Score	6	4
Level of Importance	13	
Weighted Score	**78**	**52**
Watershed Balance		
Ranking Score	8	8
Level of Importance	11	
Weighted Score	**88**	**88**
Flexibility/Expandability/Compatibility		
Ranking Score	7	5
Level of Importance	9	
Weighted Score	**63**	**45**
Effluent Reuse		
Ranking Score	8	6
Level of Importance	9	
Weighted Score	**72**	**54**
Regulatory Constraints		
Ranking Score	3	3
Level of Importance	9	
Weighted Score	**27**	**27**
Proven Effectiveness		
Ranking Score	4	4
Level of Importance	8	
Weighted Score	**32**	**32**
Carbon Footprint		
Ranking Score	5	6
Level of Importance	5	
Weighted Score	**25**	**30**
Ease of Operation		
Ranking Score	5	2
Level of Importance	5	
Weighted Score	**25**	**10**
Staffing Implications		
Ranking Score	5	3
Level of Importance	5	
Weighted Score	**25**	**15**
Total	**695**	**520**

9.5. Planning Alternative Implementation Recommendation

9.5.1. Near-Term Planning Alternative Implementation

Based on the evaluation of the six near-term planning alternatives for MMSD's operation in the Sugar River watershed, alternative MP-1A has the highest total score. Its high score is due largely to its lowest life cycle cost, fewer regulatory constraints, long track record of proven effectiveness, ease of operation and minimal staffing implications. Overall alternative MP-1A achieves the highest cost efficiency in providing wastewater conveyance and treatment service in MMSD's westside service area. However, alternative MP-1A will not be able to alleviate the issue of imbalanced inter-basin water transfer. By pumping additional 4.3 mgd of treated effluent to the Sugar River watershed, the Sugar River base flow reduction would be avoided. However the additional total life cycle costs would be $34 million, assuming the current discharge limits to Badger Mill Creek and Badfish Creek stay unchanged, but higher quality effluent discharge limits would be required for discharges in the Sugar River watershed. If higher quality effluent was ever required for both Badfish Creek and Badger Mill Creek discharges due to more stringent regulatory requirements, the cost to avoid this base flow reduction may be insignificant.

Alternative MP-1B includes construction of high quality effluent treatment facilities at the NSWTP and pumping of both regular and high quality treated effluent to the Badger Mill Creek outfall through the existing force main. This alternative has the second highest total score and also can achieve high efficiency in providing MMSD's current service in the area. Unlike MP-1A, in 2060 this alternative returns a total of 7.9 mgd of treated effluent to the Sugar River via Badger Mill Creek, and can effectively alleviate the imbalanced inter-basin water transfer issue. Since the increased flow is high quality effluent, it will not significantly increase the current TP or TN loads to the Badger Mill Creek, and therefore it may have less regulatory constraints for implementation.

Alternatives MP-1C, MP-1D and MP-2A represent centralized and decentralized approaches to solve the watershed balance issue. These three alternatives discharge the same amount of treated effluent to Badger Mill Creek and the Sugar River, and therefore will achieve similar benefits of watershed balance. Alternatives MP-1C and MP-2A would have identical discharge locations. Alternative MP-1D would use a Sugar River headwaters discharge location. Alternatives MP-1C and MP-1D would provide more potential for effluent reuse than alternative MP-2A. Alternatives MP-1C, MP-1D and MP-2A have similar life cycle costs; however, Alternative MP-1C is favored over MP -1D and MP-2A due to its higher rankings in public acceptance, effluent reuse potential, ease of operation, and staffing implications.

Alternative MP-2B represents a decentralized approach to solve the watershed balance issue. This alternative discharges the same amount of treated effluent to Badger Mill Creek and the Sugar River as alternatives MP-1C, MP-1D and MP-2A, but the Sugar River discharge is split between a headwaters location and a location downstream of the confluence with Badger Mill Creek. It will achieve slightly better benefits of watershed balance compared to alternatives MP-1C, MP-1D and MP-2A and slightly higher potential for effluent reuse than alternative MP-2A. The total life cycle cost increases by $6 million to achieve this better result.

According to the evaluation results, Alternative MP-1A appears to be the most effective alternative for providing service in the Sugar River watershed. If more stringent discharge limits are implemented, a high quality effluent treatment process will also be added at the NSWTP, which will make this alternative less favorable over the other alternatives. Currently there is no impact on the base flow in Badger Mill Creek or the Sugar River due to the return of effluent to Badger Mill Creek. As more development occurs in the Sugar River basin, base flow will be reduced in the Sugar River. If the reduction in base flow in the Sugar River were to become an issue that required mitigation, alternative MP-1B should then be considered for implementation to alleviate the base flow reduction while still maintaining relatively high cost efficiency. Alternatives MP-1C and MP-1D address base flow augmentation in the Sugar River and reduce the flow in Badger Mill Creek to its more normal levels. If the higher flows in Badger Mill Creek became an issue, alternative MP-1C or MP-1D could then be considered for implementation. Since the conceptual life cycle costs for alternatives MP-1B, MP-1C, MP-1D, MP-2A, and MP-2B are relatively close, more detailed facility planning is recommended to determine the more accurate costs for these alternatives before the final decisions are made.

Reduction of inflow/infiltration (I/I) to the existing conveyance system is an important element for the areas that experience high groundwater during wet weather conditions. Effective I/I reduction could delay the need for major capital improvement projects required to expand the capacities of the conveyance system and treatment facilities. Therefore programs to reduce I/I are recommended for all planning alternatives.

9.5.2. Long-Term Planning Alternative (4 MGD Capacity) Implementation Recommendation

Based on the evaluation of the two long-term planning alternatives for a high quality effluent treatment plant with 4 mgd capacity, Alternative MP-3A has higher ranking than MP-4A for almost all ranking criteria except for carbon footprint. Alternative MP-3A has significantly lower life cycle cost than MP-4A due to its lower operational cost achieved through economy of scale. Alternative MP-3A also has higher public acceptance since most of the new facilities will be constructed at the current NSWTP property. Alternative MP-3A also has higher flexibility for effluent reuse options. Therefore MP-3A is recommended for implementation of a high quality effluent treatment plant with 4 mgd capacity.

9.5.3. Long-Term Planning Alternative (10 MGD Capacity) Implementation Recommendation

Based on the evaluation of the two long-term planning alternatives for a high quality effluent treatment plant with 10 mgd capacity, Alternative MP-3B has higher ranking than MP-4B for almost all ranking criteria except for carbon footprint. Alternative MP-3B has significantly lower life cycle cost than MP-4B due to its lower operational cost achieved through economy of scale. Alternative MP-3B also has higher public acceptance since most of the new facilities will be constructed at the current NSWTP property. Alternative MP-3B also has higher flexibility for effluent reuse options. Therefore MP-3B is recommended for implementation of a high quality effluent treatment plant with 10 mgd capacity.

10. SCENARIO PLANNING AND SIGNPOSTS

This project includes the master planning for the MMSD's services and operations over the next 50 years, which is a long planning horizon. To compensate for the uncertain nature of the future, the method of scenario planning was used in the planning process. Based on the results of the scenario planning and other planning efforts, signposts (trigger mechanisms) were identified to provide general guidance for the MMSD's operations and facility planning efforts as the future unfolds. Details of the scenario planning processes are provided in Appendix F, TM6 – Scenario Planning Workshops.

10.1. Scenario Planning

Scenario planning is a predictive modeling technique used for risk analysis and planning policy creation. Scenario planning identifies probable outcomes that may result from a combination of factors/planning variables and their associated uncertainties. One of the greatest values of scenario planning lies in its articulation of a common future view to enable coordinated decision-making and action. Though scenario planning does not predict the future, it enables the user to prepare for future outcomes and to identify actions that need to occur to achieve desired outcomes.

The technique grew out of defense planning in the 60's and 70's and was a key element in the successful positioning of Royal Dutch Shell after the Arab oil embargo of the early 70's. Scenario planning has since been successfully used in both the public and private sectors to create situation-specific "alternative futures" while systematically accounting for future uncertainty.

During the master planning process, two scenario planning workshops were conducted with the TAC, MMSD, and consultants to identify factors and uncertainties that could potentially impact MMSD during the 50 year master planning period, with a focus on the far end of the planning period (2030 – 2060). A total of 24 initial planning variables and driving forces were identified in the workshops. These planning variables and driving forces were then ranked for their levels of uncertainty and importance. The following 4 were selected for further evaluation due to their high levels of uncertainty and importance:

- Effluent Discharge and Reuse
- Regulatory
- Public Acceptance
- Protect the Lakes

Based upon the selected planning variables and driving forces, three scenario matrices were developed in the two workshops for group discussions. The variable "Protect the

Lakes" is dependent on the effluent discharge locations, biosolids management alternatives and other planning variables, therefore it is not used as an independent planning variable in the scenario matrices. Workshop attendees had extensive discussion on each of these scenario matrices and their potential implications on the MMSD planning and operations. The discussion is documented in Appendix C of TM6 – Scenario Planning Workshops, which is included in Appendix F of this report.

10.2. Future Alternatives and Signposts

The following four long-term alternatives were identified in TM-7 but not recommended for further evaluation due to the strict regulatory constraints, high construction and operation costs, lack of proven technical feasibilities, and potential strong public resistance. However these alternatives may become more viable in the future with changes in the political environment, water resource demand, or improvements in wastewater treatment technologies.

- **Mendota WWTP –** This project includes construction of a new WWTP north of Lake Mendota near the Yahara River to serve the Yahara River watershed north of Lake Mendota. The new plant would discharge high quality effluent into the Yahara River upstream of Lake Mendota. The implementation of this project will be able to alleviate capacity expansion at the NSWTP and to provide a local source of high quality effluent for infiltration, irrigation or reuse.
- **Sun Prairie WWTP Expansion –** This project provides sewer service for the portion of the MMSD's future service area in the Koshkonong Creek watershed by directing flow from this area to the City of Sun Prairie WWTP for treatment. The project will provide relief in the conveyance system and mitigate inter-basin transfers of water.
- **Stoughton WWTP Expansion –** This project would redirect flow from PS 7 and 9 service areas to an expanded existing Stoughton WWTP. Implementation of this project includes the construction of a parallel treatment plant to treat the wastewater diverted from the MMSD system. Biosolids treatment would be provided by expanding the existing biosolids treatment train at the Stoughton WWTP. The implementation of this project will alleviate capacity expansion at the NSWTP and provide a source of high quality effluent for infiltration, irrigation or reuse.
- **Village of Oregon Discharge to PS 11 –**This is an operational reserve project for a potential annexation of the Village of Oregon by MMSD with treatment of the Village's wastewater at the NSWTP. This project does not include additional treatment capacity away from the NSWTP.

Signposts and trigger mechanisms were generated to provide MMSD the necessary "early warning" for preparing for future scenarios. The signposts and potential corresponding strategies are presented in Table 10-1.

Table 10-1 Signposts for Future Scenarios

No.	Signposts	Potential Strategies
1	Improvement in wastewater treatment technology for high quality effluent processes	• Discharge to Lake Waubesa, which would reduce effluent pumping costs and simplify operation and maintenance. • Discharge to Yahara River upstream of Lake Mendota to provide additional base flow • Increase effluent discharge to Sugar River to match the groundwater withdrawal from the watershed.
2	Local regional wastewater agencies show interest in joining MMSD. This could happen in the following scenarios: • More stringent future regulatory requirements make the small-scale local operations less cost-effective • Local agencies have financial or technical difficulties in meeting the higher discharge limits • The imbalanced inter-basin water transfer becomes a major concern and requires a regional solution and there is a consensus that MMSD is the appropriate agency to deal with the issue.	• Consider forming partnership with regional wastewater agencies • Determination of the provision of sewerage service structure and service charge rates • Negotiate to achieve win-win situations among multiple parties.
3	Imbalanced inter-basin water transfer becomes a major concern in the future	• A new Sugar River plant discharge to the confluence of the Sugar River and the Badger Mill Creek or/and headwater of the Sugar River will become more convincing. • Consider starting planning process for a Mendota Plant to provide additional base flow in the Yahara River upstream of Lake Mendota. • Increase effluent discharge to Starkweather Creek by constructing a new satellite treatment plant or conveying treated effluent from NSWTP to the area. • Expand the existing Sun Prairie WWTP and increase discharge to Koshkonong Creek.
4	Low public support for effluent reuse	• Target potential industrial effluent users. • Manage effluent discharges and reuse, be adaptive to different future scenarios. • Establish credibility with incremental implementation of effluent reuse alternatives • Identify the lead agency for overall water resources management in the area. Develop good relationships with other water sector agencies • Develop good public education program related to effluent reuse to convince the public and regulatory agencies that effluent reuse

No.	Signposts	Potential Strategies
		alternatives are protective for the public health and the environment. • Monitor the developments in the technical fields associated with effluent reuse • Identify the target environmental groups that would have an interest in water reuse and engage these groups in the water resource management discussions. • Construction of demonstration facilities to show benefits of effluent reuse alternatives and to determine capital and M/O costs.
5	High public support for effluent reuse	• Be selective in which alternatives to be implemented and to adopt the alternatives with high cost efficiency and environmental benefits. • Conduct training and prepare workforce for effluent reuse applications. • Purchase land for additional treatment and conveyance facilities. • Develop lists of potential customers for effluent reuse. • Address the seasonal demand variance for treated effluent. Provide contingency plans for effluent reuse systems.
6	Higher than projected peak flows due to increased precipitation and resulting higher rates of I/I and high groundwater levels	• Harden the conveyance system components to eliminate points of entrance for I/I. • Encourage sound management of collection systems in satellite communities • Increase the capacity of new and rehabilitated conveyance system components.
7	Water resource needs low due to: • Water conservation efforts • Lower than expected growth rate	• Delay construction of additional capacity for the conveyance system and treatment facilities. • Public education to cultivate public acceptance for new effluent discharge locations and reuse alternatives. • Monitor regulatory trends and their impacts on the effluent discharge and reuse alternatives. • Construction of demonstration facilities to determine costs and show benefits of effluent reuse alternatives.
8	Water resource needs high due to: • Higher than expected growth rate • Population shift	• Public education to cultivate public acceptance for new effluent discharge locations and reuse alternatives. • Construction of demonstration facilities to determine costs and show benefits of effluent reuse alternatives. • Conduct training and prepare workforce for effluent reuse applications. • Purchase land for additional treatment and conveyance facilities. • Develop lists of potential customers for effluent reuse. • Promote water conservation efforts • Implement programs to reduce inflow/infiltration, which will delay the need for major capital improvement projects required to expand the capacity of the conveyance system.

No.	Signposts	Potential Strategies
9	High regulatory requirements	• Upgrade the existing treatment facilities and effluent pumping system. • Diversify the treated effluent discharge locations and effluent reuse alternatives. • Diversify the biosolids utilization alternatives. • Take proactive action to identify alternative users for biosolids other than agricultural crop land. The production of a Class A biosolids material is critical to assure that a full range of alternate uses can be investigated. • Construction of new satellite treatment plants with high quality effluent processes

The purpose of the 50-Year Master Plan is to provide MMSD with a general guidance tool for providing service in the next 50 year planning period. More detailed Facility Plans will be developed as time progresses (about every 5-10 years). These Facility Plans will review the Master Plan, and evaluate the signposts/trigger mechanisms presented in the Master Plan. Based upon a re-evaluation of these issues, appropriate strategies will be determined. The individual Facility Plans may continue with the plans in the Master Plan or may make some modifications. Essentially, the Master Plan will be a dynamic document and will be reviewed and updated with each Facility Plan and allow MMSD to modify the projects that will be implemented.

Technical Memorandum No. 1

To: Madison Metropolitan Sewerage District

From: Steve McGowan, P.E., BCEE
Project Manager, Malcolm Pirnie, Inc.

Eric Wang, P.E.
Project Engineer, Malcolm Pirnie, Inc.

Date: August 11, 2008 (Final)

Subject: **50-Year Master Plan**
Review of Existing Treatment Facilities

Madison MSD Project No. 8425001
Malcolm Pirnie Project No. 6100-001

1. Introduction

This memorandum provides an evaluation of the existing flows and loadings to the Nine Springs Wastewater Treatment Plant (NSWWTP), unit capacities of the existing liquid treatment and solids disposal facilities, plant hydraulics, site considerations, electrical distribution systems, and operation and maintenance facilities.

The Madison-MSD owns and operates the Nine Springs Wastewater Treatment Plant, which treats wastewater collected from the greater Madison metropolitan area. Figure 1-1 shows the general layout of the treatment and support facilities at the plant. Wastewater treatment at the NSWWTP involves preliminary treatment, primary clarification, nitrifying activated sludge treatment incorporating biological phosphorus removal, ultraviolet disinfection, and effluent pumping. The liquid stream treatment processes are shown schematically in Figure 1-2. Treated wastewater is discharged to Badfish Creek in the lower Rock River basin in Dane County and Badger Mill Creek in the Sugar River basin via effluent force mains. Both discharge outfalls are regulated by a WPDES Permit issued by the WDNR on March 30, 2004, which will expire on March 31, 2009. The major discharging limits are summarized in Table 1-1. Biosolids processing, as shown in Figure 1-3, includes primary sludge thickening by gravity thickeners, waste activated sludge thickening by DAF thickeners, temperature phased anaerobic digestion, digested sludge thickening by

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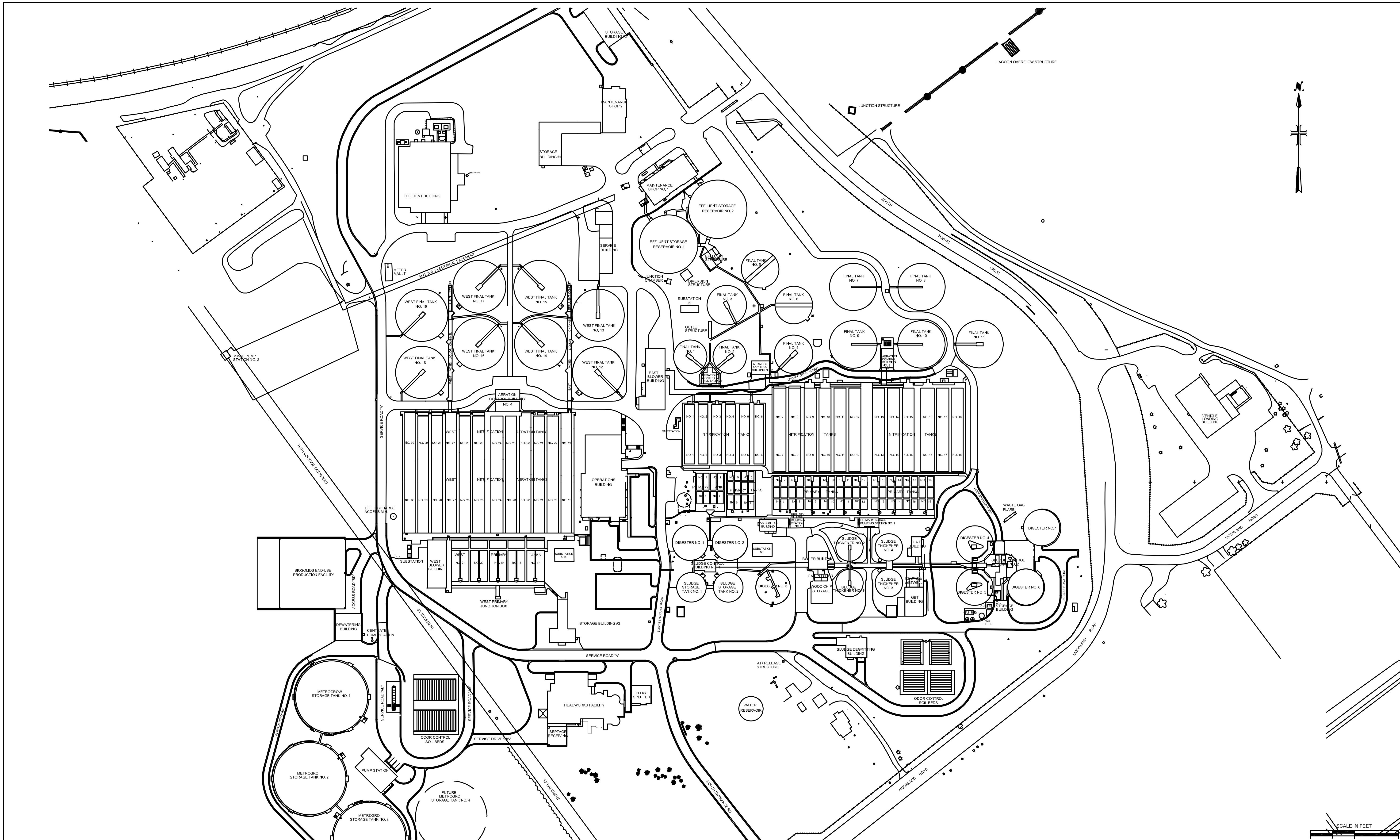

REVISIONS			
NO.	BY	DATE	REMARKS

MALCOLM PIRNIE

STRAND ASSOCIATES, INC.® ENGINEERS

DES ________
DWN ________
CKD ________

MMSD
50-YEAR MASTER PLAN

Figure 1-1
NSWTP SITE PLAN

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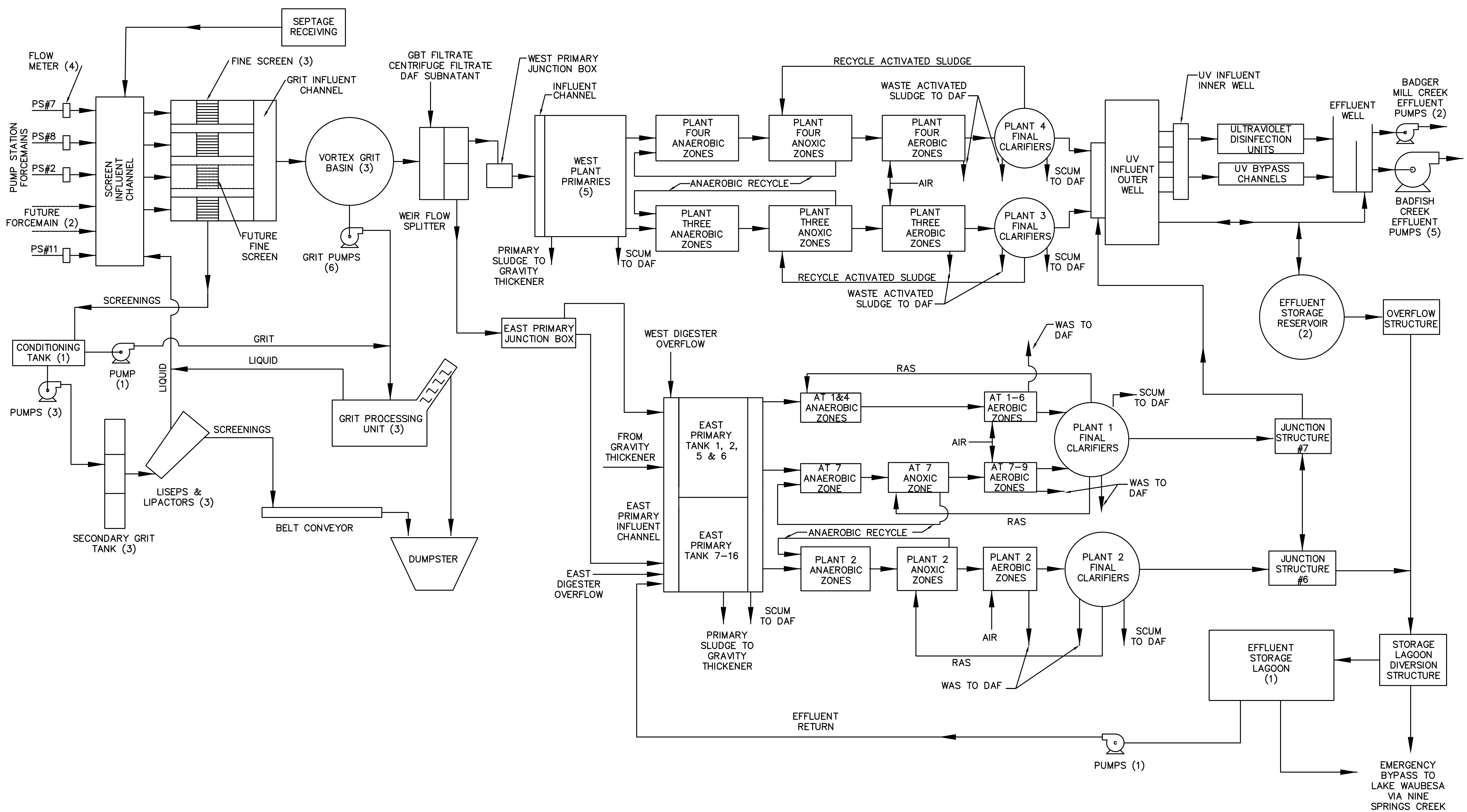

MALCOLM PIRNIE

STRAND ASSOCIATES, INC. ENGINEERS

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MMSD
50-YEAR MASTER PLAN

Figure 1-2
LIQUID PROCESS SCHEMATIC

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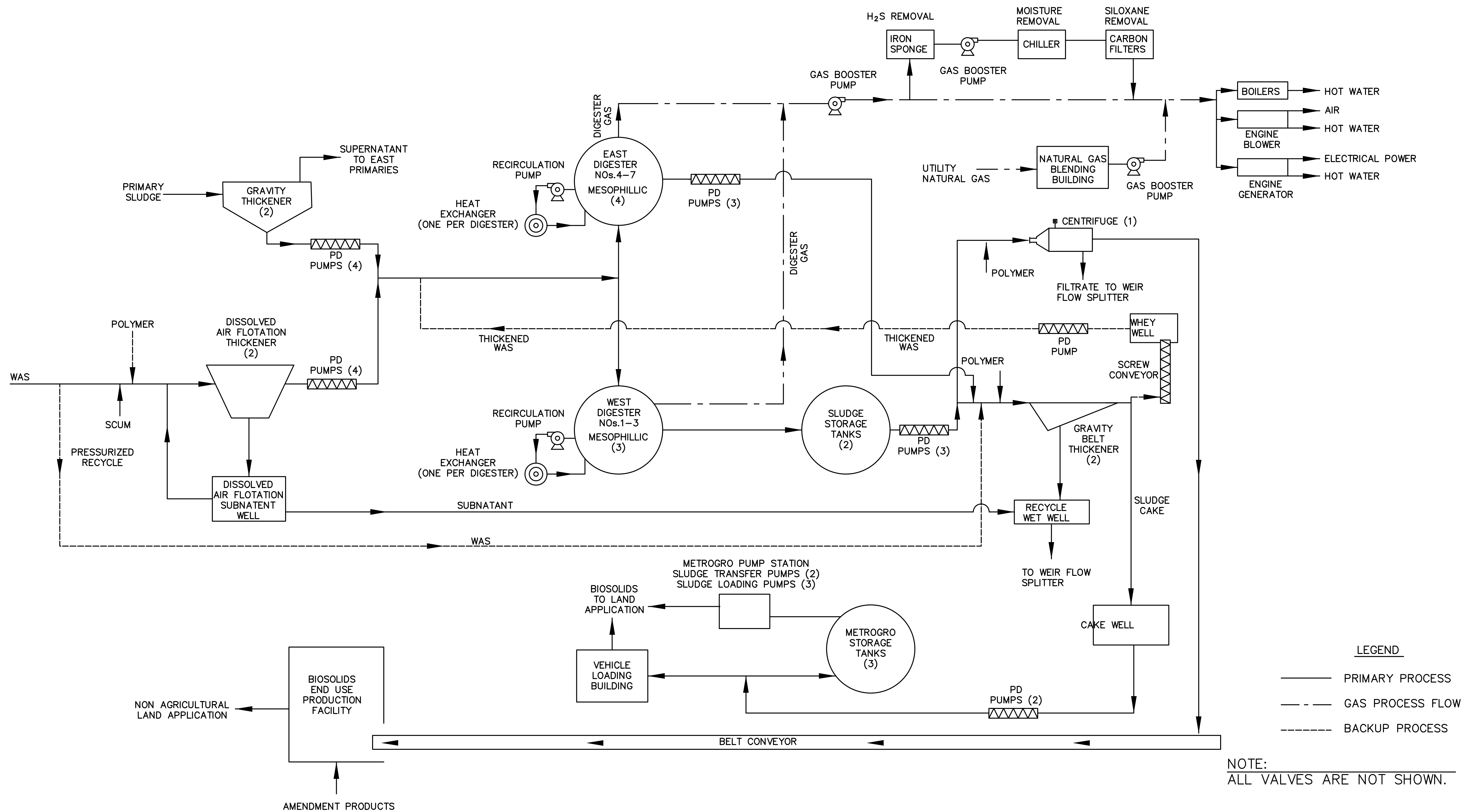

LEGEND

- PRIMARY PROCESS
- GAS PROCESS FLOW
- BACKUP PROCESS

NOTE:
ALL VALVES ARE NOT SHOWN.

MALCOLM PIRNIE

STRAND ASSOCIATES, INC.® ENGINEERS

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MMSD
50-YEAR MASTER PLAN

Figure 1-3
SOLIDS PROCESS SCHEMATIC

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gravity belt thickeners, digested sludge dewatering by centrifuge, and onsite biosolids storage. The stored biosolids, termed Metrogro, are land applied to agricultural land.

Table 1-1. Summary of Current WPDES Permit for the Nine Springs WWTP (April 1, 2004 - March 31, 2009)

Effluent Characteristics	Units	Monthly Average	Weekly Average	Daily Minimum	Daily Maximum	Geometric Mean
Badfish Creek Outfall						
BOD_5, Total	mg/L	19	20			
BOD_5, Total*	lb/day[1]	7,923	8,340			
TSS	mg/L	20	23			
TSS*	lb/day[1]	8,340	9,591			
DO	mg/L			5.0		
pH				6.0	9.0	
Phosphorus, Total	mg/L	1.5				
Fecal Coliform (April 15 – October 15)	#/100 ML					400
NH_4-N (May – September)	mg/L	1.8	4.4		17	
NH_4-N (October – April)	mg/L	4.1	10		17	
Badger Mill Creek Outfall						
BOD_5, Total (November – April)	mg/L		16			
BOD_5, Total (May – October)	mg/L		7.0			
TSS (November – April)	mg/L	16				
TSS (May – October)	mg/L	10				
DO	mg/L			5.0		
pH				6.0	9.0	
Phosphorus, Total	mg/L	1.5				
Fecal Coliform (April 15 – October 15)	#/100 ML					400
NH_4-N (October – April)	mg/L	4.0	9.1		11	
NH_4-N (May – September)	mg/L	1.3	3.2		11	

** All loadings are calculated based on the nominal design average flow of 50 MGD.*

The plant has experienced 10 major modifications since it was constructed. The major improvements during the last five Additions are listed in Table 1-2.

Table 1-2. Previous Facilities Improvements by the MMSD

Plan Completion Date	Construction Projects at NSWWTP	Major Elements of Project
May 1976	Sixth Addition 1981 Completion	• Operation Building • Metrogro Facilities • Solids Handling
May 1980	Seventh Addition 1986 Completion	• Nitrification • UV Disinfection • Effluent Pumping
February 1991	Eighth Addition 1994 Completion	• Operations Building Addition • Metrogro Storage • Solids Handling • Sludge Lagoon Abandonment
November 1999	Ninth Addition	• Enhanced Biological Phosphorus Removal Process • UV Disinfection • Side Stream Treatment
April 2003	Tenth Addition	• New Headworks • New TPAD System • New Anaerobic Digest #7 • Biosolids End-Use Production Facility • New Dewatering Facility

2. Existing Flows and Loadings

Influent loadings to the plant consist of raw wastewater delivered from the District service area via four force mains and of septage, holding tank, landfill leachate and other wastes that are trucked to the plant.

Raw Wastewater Loadings

The historical flows and loadings to the plant were analyzed by examining daily average plant records for the period of January 1996 through December 2007. The annual average plant flows, concentrations, and loadings are presented in Tables 2-1 and 2-2.

Monthly averages of plant loadings and concentrations for this period are presented in Figure 2-1 through 2-6 for raw influent wastewater flow, BOD_5, total suspended solids (TSS), total Kjeldahl nitrogen (TKN), ammonia (NH_4-N) and total phosphorus (TP). The linear regression trendline is also shown on each figure.

Table 2-1. Historical Daily Average Raw Influent Flow Characteristics for 1996-2007

Year	Flow (MGD)	TSS (mg/L)	BOD_5 (mg/L)	TKN (mg/L)	TP (mg/L)
1996	38.18	203	209	30.3	6.64
1997	36.92	208	220	31.6	6.54
1998	41.12	205	208	30.9	6.35
1999	41.59	208	208	30.9	6.07
2000	42.10	229	218	31.8	6.07
2001	41.76	222	216	32.2	5.88
2002	40.14	248	224	33.6	6.07
2003	38.56	261	243	35.2	6.49
2004	41.93	251	231	33.9	6.21
2005	39.37	243	245	37.5	6.39
2006	41.22	229	245	38.2	6.29
2007	42.88	215	240	36.4	5.95
Average	40.69	226	225	33.5	6.25

Table 2-2. Historical Daily Average Raw Influent Loadings for 1996-2007

Year	TSS (lb/day)	BOD_5 (lb/day)	TKN (lb/day)	TP (lb/day)
1996	68,116	69,918	10,020	2,150
1997	65,162	69,954	9,967	2,036
1998	69,414	71,424	10,569	2,180
1999	70,843	71,481	10,741	2,109
2000	78,127	75,424	11,045	2,102
2001	76,269	74,933	11,162	2,045
2002	81,509	75,107	11,204	2,039
2003	83,769	78,115	11,342	2,087
2004	86,915	80,860	11,915	2,186
2005	80,197	81,648	12,439	2,132
2006	78,214	83,722	13,185	2,165
2007	75,592	84,396	12,955	2,125
Average	76,712	76,796	11,462	2,111

Figure 2-1 through 2-7 present historical raw wastewater characteristics from 1996 through 2007.

Figure 2-1. Raw Wastewater Flows - Monthly Average (1996-2007)

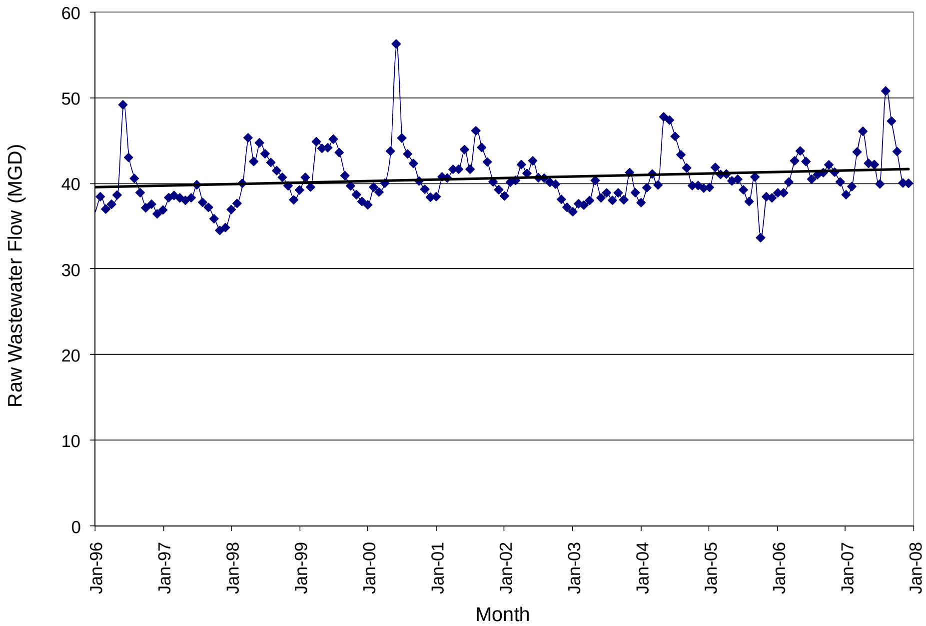


Figure 2-2. Raw Wastewater BOD_5 Loadings & Concentrations - Monthly Average (1996-2007)

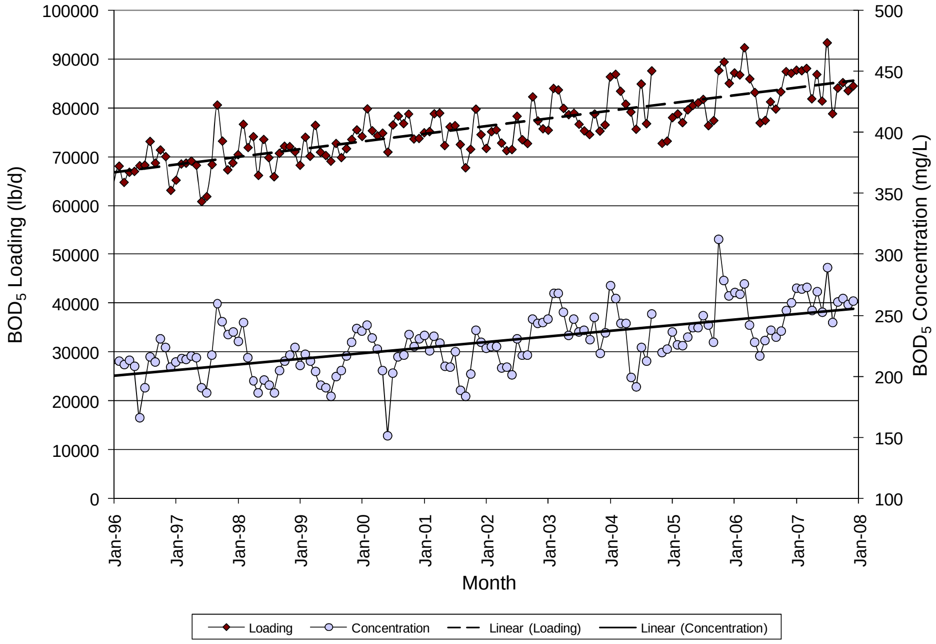


Figure 2-3. Raw Wastewater TSS Loadings & Concentrations -Monthly Average (1996-2007)

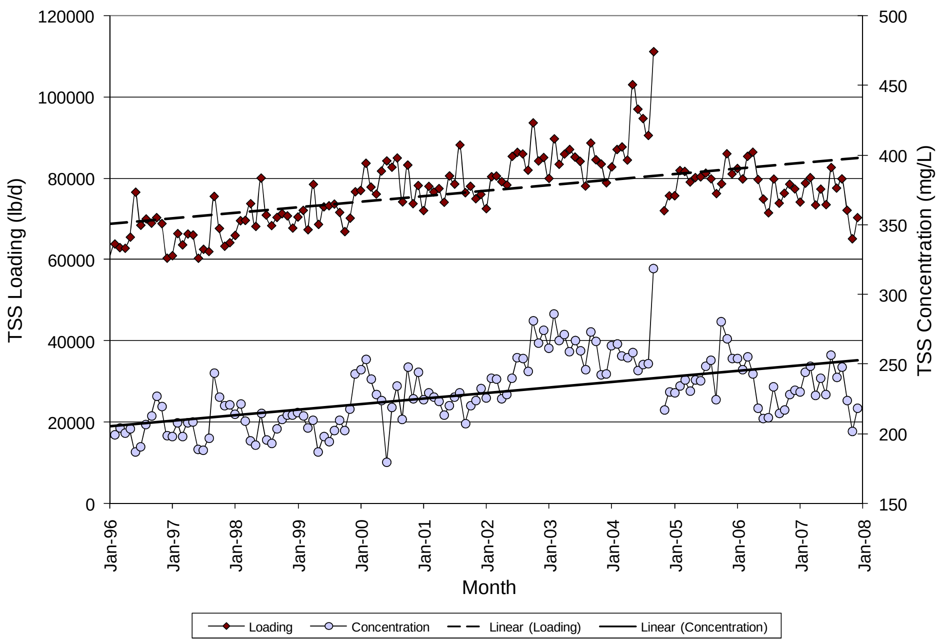


Figure 2-4. Raw Wastewater TKN Loadings & Concentrations - Monthly Average (1996-2007)

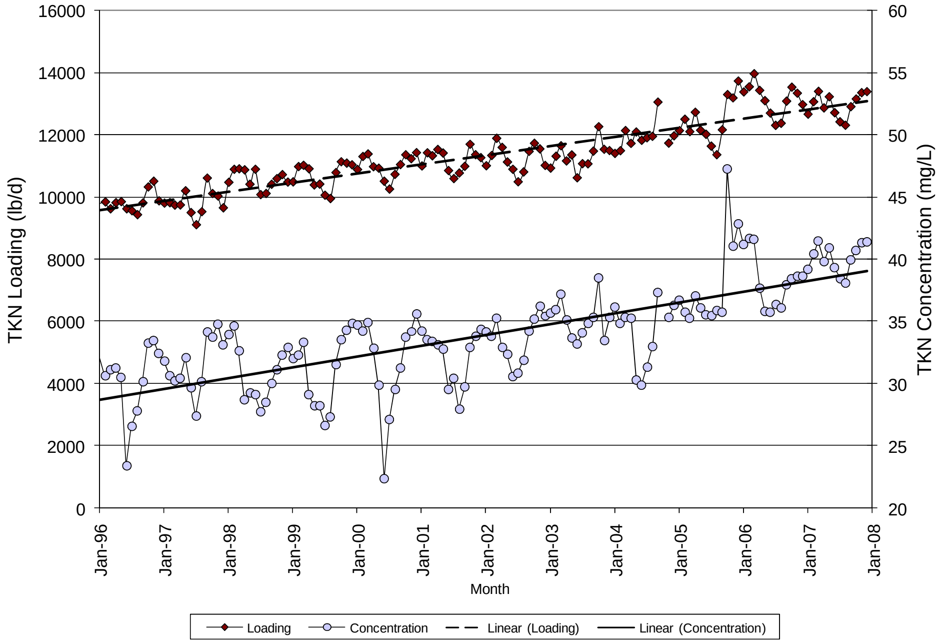


Figure 2-5. Raw Wastewater NH_4–N Loadings & Concentrations - Monthly Average (1996-2007)

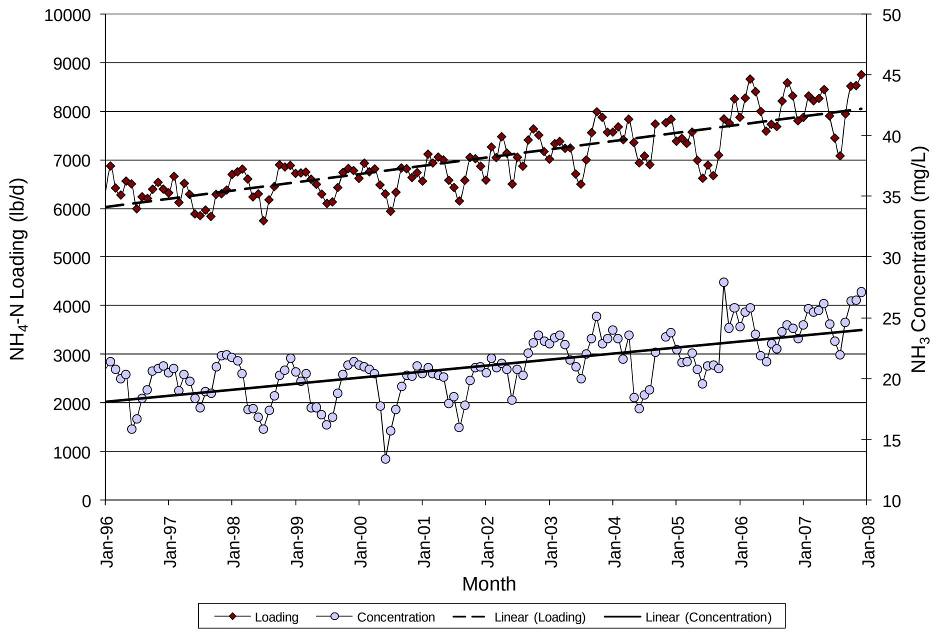


Figure 2-6. Raw Wastewater TP Loadings & Concentrations - Monthly Average (1996-2007)

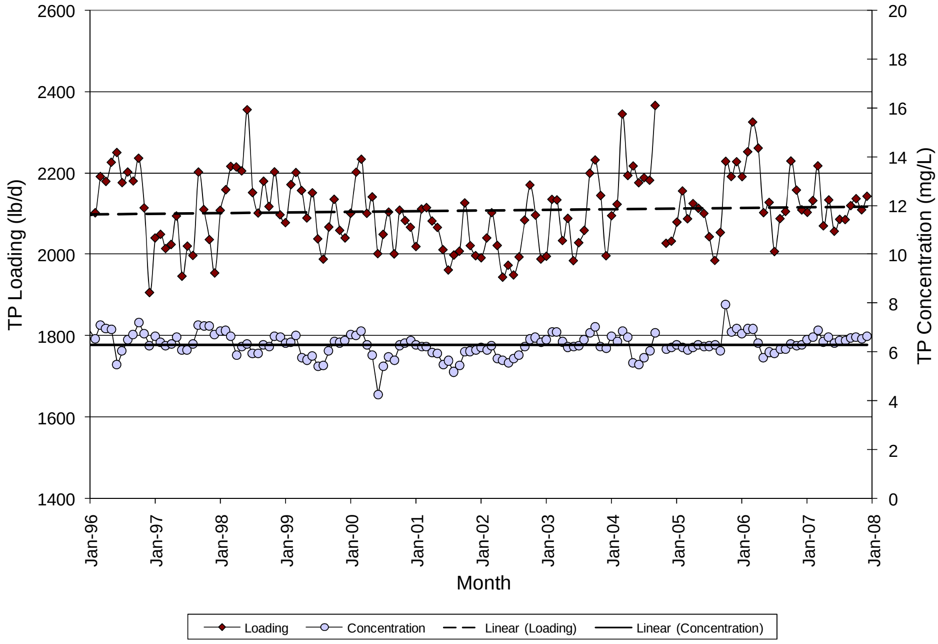


Table 2-3. Raw Wastewater Flow and Loading Projection

Parameter	Year 2000	Year 2007*	Year 2010	Year 2020
Flow (MGD)	40.8	44.2	45.6	50.5
BOD_5 (lbs/d)	73,000	84,900	90,000	107,000
TSS (lbs/d)	71,000	80,800	85,000	98,000
TKN (lbs/d)	10,700	12,170	12,800	14,700
TP (lbs/d)	2,210	2,518	2,650	3,040

** Projected flow and loadings for Year 2007 were calculated assuming linear growth between Year 2000 and 2010.*

Table 2-3 presents the projected flows and loadings presented in the Facilities Plan Report for the 10th Addition to the Nine Springs Wastewater Treatment Plant. The projected flow and loadings for Year 2007 were calculated assuming linear growth from Year 2000 through 2010.

Figure 2-1 presents historical raw wastewater flow to the plant. In spite of the wet weather periods that occurred during 1996, 2000 and 2004, the flow appears to have been relatively stable over the period of record. The recorded average raw wastewater flow for Year 2007 was 43.0 MGD, which is consistent with the flow projection presented in Table 2-3.

Figure 2-2 presents BOD_5 concentrations and loadings in the raw wastewater. Both concentrations and loadings appear to have been steadily rising over the past 12 years. The average BOD_5 loading for Year 2007 was 84,396 lbs/day, which is consistent with the projected loading in Table 2-3.

Figure 2-3 presents TSS concentrations and loadings in the raw wastewater. Both concentrations and loadings steadily rose over the period of record. The average TSS loading for Year 2007 was 75,592 lbs/day, which is slightly lower than the projected value in Table 2-3. MMSD staff believe that TSS data may have been overstated in 2004 due to problems with the raw wastewater samplers.

Figure 2-4 presents TKN concentrations and loadings in the plant raw wastewater. Both concentrations and loadings appear to have been steadily rising. The average TKN loading for Year 2007 was 12,955 lbs/day, which matches the projected value in the Table 2-3.

Figure 2-5 presents NH_4-N concentrations and loadings in the raw wastewater. Like those for TKN, both concentrations and loadings steadily rose over the period of record.

Figure 2-6 presents TP concentrations and loadings in the raw wastewater to the plant. Unlike BOD_5, TSS, TKN and NH_4-N, TP concentrations and loadings appear to have been relatively stable over the period of record. The Year 2007 TP loading averaged at 2,125 lbs/day and was approximately 16% lower than the projected loading.

The historical flow and loading data have been analyzed to define the peaking factors for raw wastewater influent from 1996 through 2007, which will be used to estimate the future peak flows and loadings to the plant. Daily data for the period of January 1996 through December 2007 were analyzed to develop peaking factors for influent flow, BOD_5, TSS, TKN, and TP loadings (see Appendix A). Loading values not within two standard deviations of the annual average loading value were removed as outliers to ensure the calculated peaking factors were representative. Appendix B shows the percentile plots for the various parameters for 1996 through 2007, where the circled data points represent the outliers. To provide a better picture of both long-term and short-term trends of the plant influent flow and loadings, the peak flows and loadings were analyzed for two time frames: Year 1996 through 2007; and Year 2007. The Year 2007 time frame was selected because it represented current plant operational conditions.

The following two methods were used to calculate plant influent peaking factors. The results generated by two methods were then compared and analyzed to determine the appropriate peaking factors to be used in future conditions.

- Method A - Percentile analysis method outlined in WEF Manual of Practice No. 8.
- Method B - Moving average method

For the Method A, 1-, 7- and 30-day maximum values were calculated as 99.7 percentile, 98.1 percentile, and 91.7 percentile values, respectively, of the entire data set.

For the Method B, moving averages were first calculated for the daily data for each category. Then the moving average data were analyzed for average, maximum, 98th percentile, and 95th percentile values. Those calculated values were then divided by the average values to determine the peaking factors. Method B was also used in the District's 10th Addition facilities planning.

Table 2-4 and 2-5 present raw wastewater peak loadings and peaking factors from 1996 through 2007 using Method A.

Table 2-6 and 2-7 present raw wastewater peak loadings and peaking factors from 1996 through 2007 using Method B.

Table 2-8 and 2-9 present raw wastewater peak loadings and peaking factors for 2007 using Method A.

Table 2-10 and 2-11 present raw wastewater peak loadings and peaking factors for 2007 using Method B.

Table 2-4. Influent Peak Load Summary for 1996-2007 (Method A)

Basis	Flow (MGD)	BOD_5 Load (lb/day)	TSS Load (lb/day)	TKN Load (lb/day)	NH_4-N (lbs/day)	TP Load (lb/day)
Average	40.7	76,799	76,715	11,463	7,076	2,111
1 Day Maximum	95.1	109,152	123,729	15,604	9,734	2,725
7 Day Maximum	50.8	99,249	103,376	14,480	9,054	2,553
30 Day Maximum	45.4	92,129	94,319	13,575	8,372	2,427

Table 2-5. Influent Peaking Factors (PF) for 1996-2007 (Method A)

Basis	Flow	BOD_5 Load	TSS Load	TKN Load	NH_4-N	TP Load
1 Day Maximum	2.34	1.42	1.61	1.36	1.37	1.29
7 Day Maximum	1.25	1.29	1.35	1.26	1.28	1.21
30 Day Maximum	1.12	1.20	1.23	1.18	1.18	1.15

Table 2-6. Influent Flows & Loads for 1996-2007 (Method B)

Parameters	Basis	Average	Maximum	98^{th} Percentile	95^{th} Percentile
Flow (MGD)	Daily Value	40.69	95.13	50.76	47.02
	7 Day MA	--	69.10	49.64	46.54
	30 Day MA	--	56.69	49.74	45.73
BOD_5 (lbs/day)	Daily Value	77,079	109,152	99,244	95,012
	7 Day MA	--	96,404	91,067	88,725
	30 Day MA	--	91,791	88,482	87,093
TSS (lbs/day)	Daily Value	76,715	123,729	102,767	97,728
	7 Day MA	--	107,353	93,501	90,178
	30 Day MA	--	97,384	92,021	87,932
TKN (lbs/day)	Daily Value	11,521	15,604	14,478	13,941
	7 Day MA	--	14,649	13,784	13,534
	30 Day MA	--	14,012	13,511	13,372
Ammonia (lbs/day)	Daily Value	7,106	9,734	9,052	8,646
	7 Day MA	--	9,093	8,662	8,449
	30 Day MA	--	8,741	8,538	8,347
TP (lbs/day)	Daily Value	2,110	2,725	2,549	2,474
	7 Day MA	--	2,433	2,326	2,286
	30 Day MA	--	2,353	2,270	2,237

Table 2-7. Influent Flows & Load Peaking Factors for 1996-2007 (Method B)

Parameters	Basis	Maximum	98th Percentile	95th Percentile
Flow (MGD)	Daily Value	2.34	1.25	1.16
	7 Day MA	1.70	1.22	1.14
	30 Day MA	1.39	1.22	1.12
BOD_5 (lbs/day)	Daily Value	1.42	1.29	1.23
	7 Day MA	1.25	1.18	1.15
	30 Day MA	1.19	1.15	1.13
TSS (lbs/day)	Daily Value	1.61	1.34	1.27
	7 Day MA	1.40	1.22	1.18
	30 Day MA	1.27	1.20	1.15
TKN (lbs/day)	Daily Value	1.35	1.26	1.21
	7 Day MA	1.27	1.20	1.17
	30 Day MA	1.22	1.17	1.16
Ammonia (lbs/day)	Daily Value	1.37	1.27	1.22
	7 Day MA	1.28	1.22	1.19
	30 Day MA	1.23	1.20	1.17
TP (lbs/day)	Daily Value	1.29	1.21	1.17
	7 Day MA	1.15	1.10	1.08
	30 Day MA	1.12	1.08	1.06

Table 2-8. Influent Peak Load Summary for 2007 (Method A)

Basis	Flow (MGD)	BOD_5 Load (lb/day)	TSS Load (lb/day)	TKN Load (lb/day)	NH_4-N (lbs/day)	TP Load (lb/day)
Average	42.89	85,129	75,858	12,938	8,067	2,116
1 Day Maximum	75.31	144,939	172,691	15,955	13,432	2,580
7 Day Maximum	59.16	112,894	101,561	15,065	9,635	2,497
30 Day Maximum	48.75	99,741	91,224	14,479	9,100	2,345

Table 2-9. Influent Peaking Factors (PF) for 2007 (Method A)

Basis	Flow	BOD_5 Load	TSS Load	TKN Load	NH_4-N	TP Load
1 Day Maximum	1.76	1.70	2.28	1.23	1.67	1.22
7 Day Maximum	1.38	1.33	1.34	1.16	1.19	1.18
30 Day Maximum	1.14	1.17	1.20	1.12	1.13	1.11

Table 2-10. Influent Flows & Loads for 2007 (Method B)

Parameters	Basis	Average	Maximum	98th Percentile	95th Percentile
Flow (MGD)	Daily Value	42.89	75.31	59.00	51.13
	7 Day MA	--	66.67	59.11	50.76
	30 Day MA	--	54.49	53.80	52.25
BOD_5 (lbs/day)	Daily Value	85,129	144,939	112,411	103,003
	7 Day MA	--	106,246	97,260	93,979
	30 Day MA	--	94,435	92,914	91,270
TSS (lbs/day)	Daily Value	75,858	172,691	101,012	94,578
	7 Day MA	--	89,382	86,866	85,385
	30 Day MA	--	82,612	81,733	81,218
TKN (lbs/day)	Daily Value	12,938	15,955	15,055	14,749
	7 Day MA	--	14,327	13,897	13,799
	30 Day MA	--	13,537	13,471	13,434
Ammonia (lbs/day)	Daily Value	8,067	13,432	9,625	9,365
	7 Day MA	--	9,222	8,906	8,841
	30 Day MA	--	8,804	8,718	8,660
TP (lbs/day)	Daily Value	2,116	2,580	2,495	2,413
	7 Day MA	--	2,363	2,294	2,255
	30 Day MA	--	2,231	2,218	2,187

Table 2-11. Influent Flow & Load Peaking Factors for 2007 (Method B)

Parameters	Basis	Maximum	98th Percentile	95th Percentile
Flow (MGD)	Daily Value	1.76	1.38	1.19
	7 Day MA	1.56	1.38	1.18
	30 Day MA	1.26	1.24	1.21
BOD_5 (lbs/day)	Daily Value	1.70	1.32	1.21
	7 Day MA	1.25	1.14	1.10
	30 Day MA	1.11	1.09	1.07
TSS (lbs/day)	Daily Value	2.28	1.33	1.25
	7 Day MA	1.20	1.16	1.15
	30 Day MA	1.11	1.09	1.09
TKN (lbs/day)	Daily Value	1.23	1.16	1.14
	7 Day MA	1.13	1.09	1.08
	30 Day MA	1.07	1.06	1.06

Table 2-11 (continued)

Ammonia (lbs/day)	Daily Value	1.67	1.19	1.16
	7 Day MA	1.14	1.10	1.10
	30 Day MA	1.09	1.08	1.07
TP (lbs/day)	Daily Value	1.22	1.18	1.14
	7 Day MA	1.12	1.08	1.07
	30 Day MA	1.05	1.05	1.03

Table 2-12 summarizes the peaking factors for each category generated by two methods for the two time frames. Both the plant's Ninth and Tenth Addition facilities planning studies have found that the maximum BOD and maximum TSS loading days raw wastewater flows were approximately equal to 98th percentile values. Therefore for Method B, flow peaking factors were determined based on 98th percentile values and loading peaking factors were based on maximum values.

Table 2-12. Influent Peaking Factor (PF) Summary

Parameters	Basis	Method A 1996-2007	Method B* 1996-2007	Method A 2007	Method B* 2007
Flow (MGD)	1 Day Max	2.34	1.25	1.76	1.38
	7 Day Max	1.25	1.22	1.38	1.38
	30 Day Max	1.12	1.22	1.14	1.24
BOD_5 (lbs/day)	1 Day Max	1.42	1.42	1.70	1.70
	7 Day Max	1.29	1.25	1.33	1.25
	30 Day Max	1.20	1.19	1.17	1.11
TSS (lbs/day)	1 Day Max	1.61	1.61	2.28	2.28
	7 Day Max	1.35	1.40	1.34	1.20
	30 Day Max	1.23	1.27	1.20	1.11
TKN (lbs/day)	1 Day Max	1.36	1.35	1.23	1.23
	7 Day Max	1.26	1.27	1.16	1.13
	30 Day Max	1.18	1.22	1.12	1.07
Ammonia (lbs/day)	1 Day Max	1.37	1.37	1.67	1.67
	7 Day Max	1.28	1.28	1.19	1.14
	30 Day Max	1.18	1.23	1.13	1.09
TP (lbs/day)	1 Day Max	1.29	1.29	1.22	1.22
	7 Day Max	1.21	1.15	1.18	1.12
	30 Day Max	1.15	1.12	1.11	1.05

** Flow peaking factors were determined based upon 98th percentile values and loading peaking factors were determined based upon maximum values.*

The 1-day max flow peaking factors generated by Method A are significantly higher than those generated by Method B. This is because Method A defines the 1-day max value as the 99.7th percentile of the entire data set while the Method B defines it as the 98th percentile. Based on the previous discussion, peaking factors based on the 98th percentile values were selected as recommended peaking factors.

The peaking factors generated by both methods for loadings in each category appear to be very consistent. Those peaking factors for 2007 are generally higher than their counterparts for the period between 1996 and 2007 with exception of TKN and TP.

Selection of the appropriate peaking factors is somewhat subjective, dependent on statistical results and professional judgment. The previous facilities planning studies by the District were also used as reference in selecting peaking factors.

Table 2-13 presents the recommended peaking factors for each category. For comparison purpose, the peaking factors suggested in the 9th and 10th Additions were also listed.

Table 2-13. Recommended Influent Peaking Factors (PF)

Parameters	Basis	9th Addition	10th Addition	Recommended By This Study
Flow (MGD)	1 Day Max	1.25	1.30	1.40
	7 Day Max	n/a	1.25	1.35
	30 Day Max	1.05	1.15	1.25
BOD_5 (lbs/day)	1 Day Max	1.50	1.60	1.70
	7 Day Max	n/a	1.20	1.30
	30 Day Max	1.15	1.15	1.20
TSS (lbs/day)	1 Day Max	2.05	2.00	2.30
	7 Day Max	n/a	1.40	1.40
	30 Day Max	1.15	1.15	1.20
TKN (lbs/day)	1 Day Max	1.40	1.40	1.40
	7 Day Max	n/a	1.15	1.25
	30 Day Max	1.10	1.10	1.20
Ammonia (lbs/day)	1 Day Max	n/a	n/a	1.70
	7 Day Max	n/a	n/a	1.30
	30 Day Max	n/a	n/a	1.20
TP (lbs/day)	1 Day Max	1.40	1.60	1.30
	7 Day Max	n/a	1.15	1.20
	30 Day Max	1.05	1.10	1.10

** The recommended peaking factors are used for calculating corresponding concentrations for various loadings. For plant hydraulics analysis, total capacities of influent pumping stations should be used. For equalization facility analysis, a peaking factor of 2.34 should be used.*

Septage, Holding Tank, Landfill Leachate and Other Wastes

Trucked in wastes from septic tanks, holding tanks, landfills (leachate) and commercial establishments (grease traps, settling basins, and portable toilets) are discharged into the septage receiving station. Table 2-14 presents annual average daily contribution of these wastes, excluding landfill leachate and other waste loadings.

Table 2-14. Septage, Holding Tank, Grease Trap and Settling Basin – Average Daily Influent Loadings

Year	Flow (gpd)	BOD_5 (lbs/d)	TSS (lbs/d)	TKN (lbs/d)	TP (lbs/d)
1996	24,793	387	1,208	42	8
1997	23,975	395	1,123	38	9
1998	27,480	462	1,662	82	16
1999	25,806	483	2,104	67	16
2000	26,362	614	1,633	55	12
2001	31,242	471	1,887	66	14
2002	30,031	384	1,247	57	11
2003	31,197	590	1,447	64	13
2004	37,726	933	1,966	75	17
2005	35,929	612	1,510	67	15
2006	41,268	703	1,886	87	14
2007	45,962	779	1,536	89	17
Average	31,814	568	1,601	66	13

The monitoring records of landfill leachate and other wastes were only studied for January 2005 through December 2007 to determine the loadings. This period was selected for analysis because it represented the plant's current operation, i.e., the District stopped receiving whey from Bancroft Dairy Company since 2005 and the hauled-in waste flow has been significantly reduced since then. The landfill leachate and other wastes consist of a variety of waste streams; the characteristics of some of these waste streams are generally unknown. Due to lack of monitoring data, the concentrations of these waste streams were estimated by calculating weighted averages of other waste streams whose characteristics were better defined. The results are shown in Table 2-15. The combined loadings of all hauled-in wastes were then calculated as percentages of plant daily average loadings in 2007 (Table 2-16).

Table 2-15. Other Wastes – Average Daily Influent Loadings

Year	Flow (gpd)	BOD_5 (lbs/d)	TSS (lbs/d)	TKN (lbs/d)	TP (lbs/d)
2005	2,647	642	142	26	4
2006	2,906	704	155	28	4
2007	2,534	614	136	25	4
Average	2,695	653	144	26	4

Table 2-16. Total Hauled-in Wastes

Parameters	Total Hauled-in Wastes in Year 2007	Total Hauled-in Wastes as Percentage of Raw Wastewater Daily Average Loadings in Year 2007 (%)
Flow	48,496 gpd	0.12
BOD_5	1,393 lbs/d	1.65
TSS	1,672 lbs/d	2.21
TKN	114 lbs/d	0.88
TP	21 lbs/d	0.99

3. Headworks

The existing headworks were constructed during the Tenth Addition project in 2005, which is the latest major capital improvement project to the plant facilities. The existing headworks include influent flow measurement, fine screening, grit removal by vortex grit basins, and a weir flow splitting structure distributing flows to the East and West Plants. It also includes screenings and grit processing equipment, the plant water (W4) system, odor control system, and septage receiving facility.

Wastewater to be processed enters the headworks facility via influent force mains. Flow is measured by Venturi flowmeters on each force main before proceeding through fine screens. After screening, the flow continues to vortex grit basins. Screenings are conveyed by sluice trough to screenings processing units. Grit from the vortex grit basins is pumped to grit processing units. Processed screenings and grit are conveyed to roll-off containers by a reversible belt conveyor.

Flow exiting the grit basins enters the flow splitting structure and is distributed to the East and West Plants through weir troughs with manual stop plates.

Influent Flowmeters

There are a total of four flowmeters installed in the Headworks Building with space reserved for two future flowmeters. The flowmeters are installed on the four force mains connected to six sewage pumping stations. Three pumping stations are connected to one of the force mains. The peak flows measured by flowmeters correspond with the maximum capacities of the pumping stations.

Table 3-1 presents information on the plant influent flowmeters.

Table 3-1. Influent Flowmeter Description

Parameters	Flowmeter #1	Flowmeter #2	Flowmeter #3	Flowmeter #4
Number of Units	1	1	1	1
Type	Venturi	Venturi	Venturi	Venturi
Inlet Diameter	36”	36”	42”	48”
Throat Diameter	21”	25.2”	21”	25.2”
Pumping Stations Served	No. 11	No. 2,3 & 4	No. 8	No. 7

Table 3-2. Influent Flowmeter Capacity Evaluation

Parameters	Flowmeter #1	Flowmeter #2	Flowmeter #3	Flowmeter #4	Total
Design Criteria					
Pumping Station Served	No. 11	No. 2 No. 3 No. 4	No. 8	No. 7	
Pumping Station Maximum Capacity	31.2 MGD	46.7 MGD	30.7 MGD*	45 MGD	153.6 MGD**
Max Flowmeter Design Capacity	30 MGD	48 MGD	30 MGD	55.3 MGD	163.3 MGD
Headloss @ Max Capacity	9.4” wc	12” wc	7” wc	7.1” wc	
Current Utilization of Capacity @ Maximum Flow	104%	97%	102%	81%	

**Capacity of pumping station will increase to 34.1 MGD in 2010.*
*** Total pumping capacity will increase to 157.0 MGD in 2010.*

Table 3-2 summarizes the maximum pumping capacity of each pumping station and design capacity for each flowmeter. The results show that the flowmeters #1 and #3 are slightly overloaded at the maximum pumping capacities and flowmeters # 2 and #4 have sufficient capacity to accommodate the flow when their corresponding pumping stations are working at their maximum capacities. Overall the existing flowmeters provide adequate capacities for the plant influent flows.

Fine Screening System

The existing screening system was installed in the plant’s Tenth Addition and is in very good condition. The plant has four screen channels, three of them have center-flow fine screens installed and the fourth one serves as a by-pass channel and could be used as screen channel for future expansion. Excess flow is bypassed automatically via the two bypass channels and over weirs on the screens as the water level in the channel rises. Material captured on the screens is removed by the high pressure spray wash and transported to a conditioning tank through a screenings trough. From the conditioning tank it is pumped to liquid separator (LISEP) units which use centrifugal action to separate the water from the solids. The liquid is returned to the headworks channels, while the separated solids are dewatered by Lipactor units. The dewatered screenings are conveyed to either of two roll-off containers in the Headworks Building.

Table 3-3 presents detailed information on the screening system equipment.

Table 3-3. Fine Screening System Description

Parameters	Fine Screen	Maci Pumps	Lisep Units	Lipactor Units
Number of Units	3	3	3	3
Type	0.25 in, perforated plate	Centrifugal w/ macerator	--	--
Unit Capacity	60 MGD	126 gpm	126 gpm	25 gpm
Channel Width	4.0 ft	--	--	--
Channel Depth	10.75 ft	--	--	--

Table 3-4. Fine Screening System Capacity Evaluation

Parameters	Fine Screen	Maci Pumps	Lisep Units	Lipactor Units
Unit Capacity	60 MGD	126 gpm	126 gpm	--
Maximum Capacity	180 MGD	378 gpm	378 gpm	--
Firm Capacity	120 MGD	256 gpm	256 gpm	--
Maximum Hour Flow	153.6 MGD	--	--	--
Current Utilization of Maximum Capacity	85%	85%*	85%*	85%*
Current Utilization of Firm Capacity	128%	128%*	128%*	128%*

** Estimated based upon the utilization of maximum capacity of fine screens.*

Table 3-4 summarizes the capacities of the major equipment of the screening system. It appears that, with all three screens in service, the screening system has adequate capacity to accommodate maximum plant flow when all four plant influent sewage pumping stations are working at their maximum capacities. When a fourth screen is installed in the bypass channel, the maximum screen capacity would be expanded to 240 MGD with a firm capacity of 180 MGD.

Grit Processing System

There are three existing vortex grit basins at the headworks facilities with space reserved to allow the construction of a fourth vortex grit basin. Each grit basin is equipped with a bypass gate.

Two grit pumps are provided for each basin, one operational and one standby. Grit slurry from the vortex grit basins is pumped to grit processing units for further processing. Each processing unit includes two cyclone concentrators, one grit classifier, integral washing, and one inclined dewatering screw. Liquid from the grit processing units drains to the screen influent structure. Dewatered grit is discharged to a conveyor which transfers the grit to either of two roll-off containers in the Headworks Building.

Table 3-5 presents detailed information on the grit system equipment.

Table 3-5. Grit System Description

Parameters	Grit Basins	Grit Pumps	Grit Processing Unit
Number of Units	3	6 (2 for each grit basin)	3
Diameter	20 ft	--	--
Depth	11.75 ft	--	--
Type	--	Centrifugal	Cyclone concentrator, screw conveyor
Unit Capacity	60 MGD	250 gpm	500 gpm

Table 3-6 summarizes the capacities of the major equipment of the grit system. As can be seen, with all three grit basins in service, the grit system has adequate capacity to accommodate maximum plant flow when all four plant influent sewage pumping stations are working at their maximum capacities. When a fourth grit basin and grit pumps are installed in the reserved space, the maximum grit processing capacity would be expanded to 240 MGD with a firm capacity of 180 MGD.

Table 3-6. Grit System Capacity Evaluation

Parameters	Grit Basins	Grit Pumps	Grit Processing Unit
Unit Capacity	60 MGD	250 gpm	500 gpm
Maximum Capacity	180 MGD	1,500 gpm	1,500 gpm
Firm Capacity	120 MGD	1,000 gpm	1,000 gpm
Maximum Hour Flow	153.6 MGD	--	--
Current Utilization of Maximum Capacity	87%	87%*	87%*
Current Utilization of Firm Capacity	128%	128%*	128%*

** Estimated based upon the utilization of maximum capacity of grit basins.*

Septage Receiving Station

The existing septage receiving station is located south of the Headworks Building, and was built recently as part of the plant's Tenth Addition. The station is comprised of two parallel bays which allow two vehicles to discharge at the same time. Septage collected is piped directly to the screen influent structure in the Headworks Building. Table 3-7 presents detailed information of the existing septage receiving station. The septage receiving station appears to have sufficient capacity to accommodate haul-in wastes. The plant staff has been experiencing quick blinding of the screens by the rags, grease, and other material in the septage, which decreases the screen capacities and causes operational problems. They have suggested adding additional screening facilities at the septage receiving station.

Table 3-7. Septage Receiving Station Description

Parameters	Values
Number of Units	1
Type	Concrete
Dimensions	40' (L) x 40' (W)
Number of Bays per Unit	2

4. Liquid Process

Flow Splitter

The existing flow splitter was constructed during the plant's Tenth Addition. The structure splits screened and degritted plant flow between the East and West Plants using a fixed weir flow splitting structure. A slide gate and 60" diameter pipe stub were installed for a future Vortex Grit Basin. Space has also been reserved for a 84" future plant pipe and a 72" future excess flow diversion pipe. The District currently has no final plan for the use of these two reserved pipes. The flow splitter structure contains five weir troughs with manually adjustable weir plates, which allow for finite flow control of the flows to the East and West Plants. Table 4-1 presents detailed information on the existing flow splitter.

Table 4-1. Flow Splitter Description

Parameters	Values
Number of Units	1
Type	Fixed weir flow splitting
Number of Troughs	5
Total Weir Length	112.5'
East/West Plant Pipe Size	84" / 72"
Future Plant Pipe Size	84'
Future Excess Flow Diversion Pipe Size	72"

Primary Settling Facilities

Effluent from the splitter proceeds to the primary tanks to remove readily settleable solids and floating materials before being treated in the aeration basins. There are 14 primary clarifiers in the East Plant and 5 primary clarifiers in the West Plant. All clarifiers are rectangular units with chain and flight sludge removal mechanisms. Settled primary sludge is pumped to gravity thickeners for thickening before being digested. Table 4-2 provides detailed information on the existing primary clarifiers at both the East and West Plants.

Table 4-2. Primary Clarifier Description

Parameters	East Primary Tanks (No. 1-2)	East Primary Tanks (No. 5-6)	East Primary Tanks (No. 7-16)	West Primary Tanks (No. 17-21)
Number of Units	2	2	10	5
Dimension (L x W)	85' x 31'	101' x 31'	88' x 33.5'	100' x 40'
Side Water Depth	10'	10'	10'	8'
Unit Surface Area	2,635 sf	3,131 sf	2,948 sf	4,000 sf
Total Surface Area	5,270 sf	6,262 sf	29,480 sf	20,000 sf
Total Surface Area	41,012 sf			20,000 sf

Table 4-3 presents the hydraulic loading statistics for both East and West Plant primary clarifiers based on operational data from 2002 through 2007. Although West Plant primary clarifiers account for only 33% of the total primary clarification surface area, they treat approximate 60% the total plant flow. Table 4-4 presents a capacity evaluation of the existing primary clarifiers. Because the plant does not have the ability to divert flow splitter effluent to either the Nine Springs Creek or the effluent storage lagoon, the primary clarifiers have to accommodate all process flow from the headworks. Based on commonly accepted design criteria, the East Plant primary clarifiers have sufficient firm hydraulic capacity for the maximum day flow, while the West Plant primary clarifiers will be heavily overloaded under both the average and maximum day conditions for hydraulic loadings. West Plant clarifier daily operational data from 2002 through 2007 were analyzed to estimate the actual primary clarification capacities.

Table 4-3. Primary Clarifier High Hydraulic Loading Statistics (2002-2007)

Basis	East Primary Clarifiers			West Primary Clarifiers		
	Flow (MGD)	Hydraulic Loading (gal/sf/d)	Peaking Factor	Flow (MGD)	Hydraulic Loading (gal/sf/d)	Peaking Factor
Average	17.75	433	1.0	23.07	1,153	1.0
Maximum	47.39	1,156	2.7	47.74	2,387	2.1
98th Percentile	23.86	582	1.3	28.36	1,418	1.2
95th Percentile	22.39	546	1.3	26.63	1,331	1.2

Table 4-4. Primary Clarifier Capacity Evaluation

Parameters	East Primary Tanks (No. 1-16)		West Primary Tanks (No.17-21)	
	Maximum Capacity	Firm Capacity	Maximum Capacity	Firm Capacity
Design Average Hydraulic Loading	1,000 gallons / sf day			
Design Maximum Hydraulic Loading	1,500 gallons / sf day			
Design Average Hydraulic Capacity	41.0 MGD		20.0 MGD	
Design Maximum Hydraulic Capacity	61.5 MGD		30.0 MGD	
Current Average Flow	17.75 MGD		23.07 MGD	
Current Utilization of Average Hydraulic Capacity	43%	47%	115%	144%
Current Maximum Day Flow	47.39 MGD		47.74 MGD	
Current Utilization of Maximum Hydraulic Capacity	77%	83%	159%	199%
Current 98th Percentile Flow	23.86 MGD		28.36 MGD	
Current Utilization of Maximum Hydraulic Capacity	39%	42%	95%	118%
Current 95th Percentile Flow	22.39 MGD		26.63 MGD	
Current Utilization of Maximum Hydraulic Capacity	36%	39%	89%	111%

Figure 4-1 presents West Plant primary clarifier effluent TSS removal efficiency distribution under various hydraulic loading categories based upon 2002-2007 operational data. There are 43 events that fall in the “> 1,400 gal/sf/d” category, ranging from 1,402 to 2,387 gal/sf/d. TSS removal efficiencies in this category range from 30% to 75%, with the majority falling between 40% and 70%. Compared to other categories, effluent TSS removal efficiency showed no significant deterioration under high hydraulic loadings.

Figure 4-1. TSS Removal Efficiency Frequency Distribution at Various Hydraulic Loadings for West Plant Primary Clarifiers

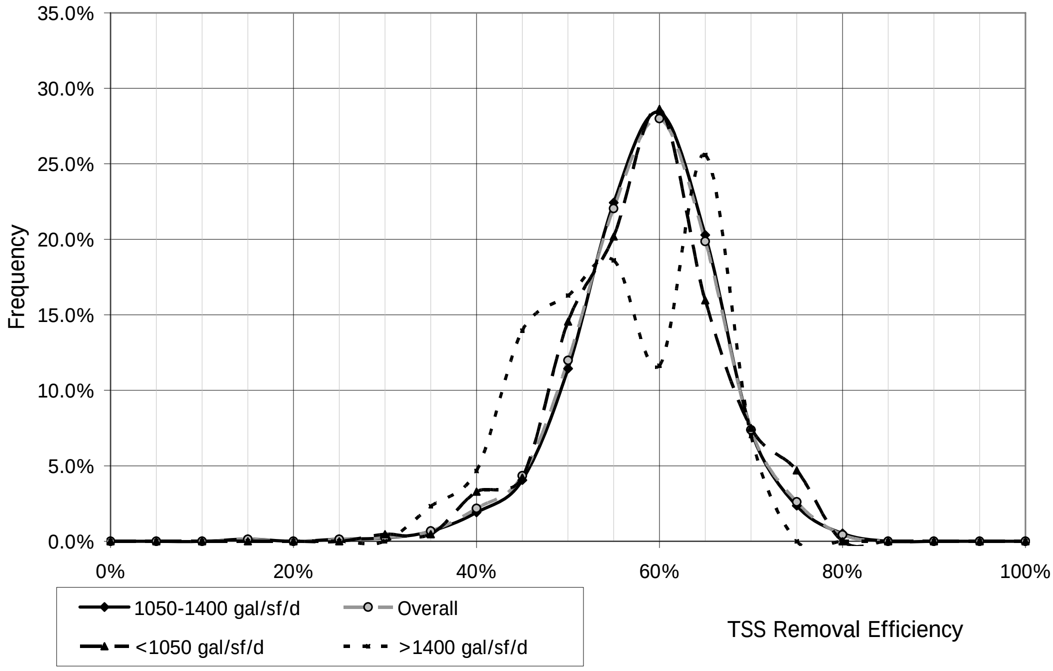
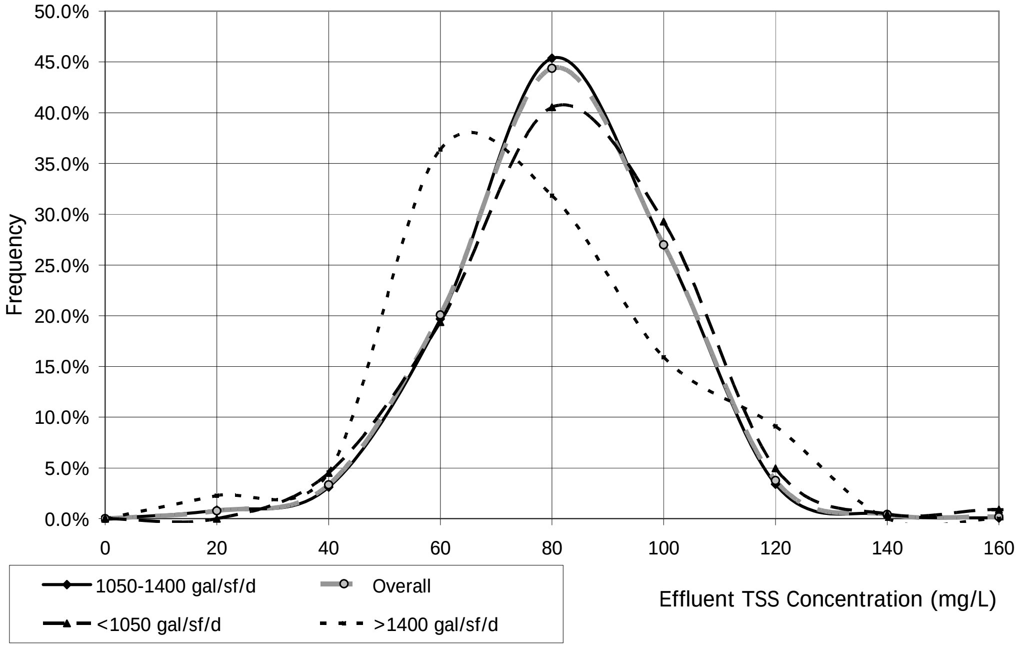


Figure 4-2 presents West Plant clarifier effluent TSS concentration distribution under various hydraulic loadings based upon 2002-2007 operational data. Effluent TSS concentrations in the “>1,400 gal/sf/d” category range from 10 to 160 mg/L, with the majority falling between 50 and 110 mg/L. Compared to other categories, the primary clarifier performance didn’t degrade in terms of TSS concentrations under high hydraulic loadings.

Figure 4-2. Effluent TSS Concentration Frequency Distribution at Various Hydraulic Loadings for West Plant Primary Clarifiers



Based on the analysis on the clarifier operational data, the existing clarifiers should be able to accommodate higher than design hydraulic loadings and maintain satisfactory TSS removal. The actual hydraulic loadings monitored at the West Plant clarifiers were used to estimate the actual capacities of the clarifiers. From 2002 to 2007, the West Plant primary clarifiers experienced 7 events with hydraulic loadings between 1,800 and 2,400 gal/sf/d and maintained satisfactory performance. Selection of appropriate hydraulic capacity criteria between 1,800 and 2,400 gal/sf/d is somewhat subjective and must rely on professional judgment.

The recommended hydraulic primary clarifier capacities are presented in Table 4-5. Due to lack of recorded high hydraulic loading data, the East Plant primary clarifiers were rated based on the original design hydraulic loading of 1,500 gallons/sf/day. Further tests are recommended to estimate the actual capacities.

Table 4-5. Primary Clarifier Recommended Capacity

Parameters	East Primary Tanks (No. 1-16)		West Primary Tanks (No.17-21)	
	Maximum Capacity	Firm Capacity	Maximum Capacity	Firm Capacity
Recommend Hydraulic Loading	1,500 gallons / s.f. day		2,000 gallons / s.f. day	
Recommended Maximum Hydraulic Capacity	61.5 MGD	56.8 MGD	40.0 MGD	32.0 MGD
Combined Maximum Hydraulic Capacity	101.5 MGD			
Combined Firm Hydraulic Capacity	88.8 MGD			

Table 4-6 provides information on the existing primary sludge pumps at both the East and the West Plants. Table 4-7 presents a capacity evaluation for the existing primary sludge pumps. At the maximum day condition the primary sludge pumps have adequate firm pumping capacity under the current condition.

Table 4-6. Primary Sludge Pumps Equipment Description

Parameters	East Primary Pumps	West Primary Pumps
Number of Units	7 total (4 for Plant No. 1, and 3 for Plant No. 2)	3
Type	Centrifugal	Centrifugal
Rated Capacity per Pump	830 gpm	830 gpm
Maximum Total Capacity	5,810 gpm	2,490 gpm

Table 4-7. Primary Sludge Pumps Capacity Evaluation

Parameters	East Primary Pumps	West Primary Pumps
Design Criteria	Transfer expected range of primary sludge to the gravity thickeners	
Firm Pumping Capacity (one pump out of service)	4,980 gpm	1,660 gpm
Current Required Pumping Capacity at 1% Solids (maximum day)	494 gpm*	741 gpm*
Current Utilization of Firm Capacity	60%**	89%**

* *The max day flow is based upon operational data from 2002 through 2007.*
***Assume primary sludge pumps operate continuously, one pump at a time.*

Aeration Basins

Biological treatment of the primary effluent occurs in the aeration basins. There are 18 aeration basins in the East Plant and 12 in the West Plant. The aeration basins are configured such that each group of three aeration basins functions as one "folded" treatment unit. Thus, there are 6 treatment units in the East Plant and 4 treatment units in the West Plant. Aeration tank effluent proceeds into the secondary clarifiers for settling. The existing secondary treatment facility is an enhanced biological phosphorus removal (EBPR) system with two process configurations being utilized – The University of Cape Town (UCT) Variation process, which is utilized for the majority of the plant, and the anaerobic/aerobic (A/O) process.

The UCT process consists of anaerobic, anoxic and aerobic zones. Influent wastewater enters the anaerobic zone and is combined with anaerobic recycle from the anoxic zone. Mixed liquor then flows into the anoxic zone that is created by returning RAS from the final clarifiers. The mixed liquor then proceeds into the aerobic zone for further treatment.

The A/O process is utilized in 2 of the 3 treatment units of Plant 1 (Aeration Basins 1 – 6). In the A/O process, the anoxic zone is eliminated and RAS is combined with the influent wastewater in the anaerobic zone. Following the anaerobic zone, the mixed liquor flows to the aerobic zone.

Tables 4-8 and 4-9 provide information on the existing aeration basins of both East and West Plants. There are six parallel treatment units in the East Plant and four in the West Plant. Each treatment unit includes three passes: anaerobic, anoxic and aerated zones in pass 1, and only aerated zones in pass 2 and pass 3.

Table 4-8. East Aeration Basin Description

Parameters	No. 1-3 No. 4-6	No. 7-9	No. 10-12 No. 13-15 No. 16-18	Total
Number of Treatment Units	2	1	3	6
Process Configuration	A/O	UCT	UCT	
Unit Volume (gallon) Zone 1 (Anaerobic) Zone 2 (Anaerobic) Zone 3 (Anaerobic) Zone 4 (Anoxic) Aerated	 120,000 120,000 -- -- 1,198,000	 131,000 131,000 131,000 131,000 1,838,000	 128,000 128,000 128,000 128,000 1,798,000	 755,000 755,000 515,000 515,000 9,628,000
Total Volume (gallon)	2,876,000	2,362,000	6,930,000	**12,168,000**
No. of Diffusers in 1st Pass	992	1,920		2,912
No. of Diffusers in 2nd Pass	1,092	4,800		5,892
No. of Diffusers in 3rd Pass	812	3,000		3,812
Total No. of Diffusers	2,796	9,720		**12,616**

Table 4-9. West Aeration Basin Description

Parameters	No. 19-21 No. 22-24 No. 25-27 No. 28-30	Total
Number of Treatment Units	4	4
Process Configuration	UCT	UCT
Unit Volume (gallon) Zone 1 (Anaerobic) Zone 2 (Anaerobic) Zone 3 (Anaerobic) Zone 4 (Anoxic) Aerated	 202,000 202,000 202,000 202,000 2,910,000	 808,000 808,000 808,000 808,000 11,640,000
Total Volume (gallon)	3,718,000	**14,872,000**
No. of Diffusers in 1^{st} Pass.	2,480	
No. of Diffusers in 2nd Pass	6,360	--
No. of Diffusers in 3rd Pass	4,114	
Total No. of Diffusers	12,954	**12,954**

Table 4-10 presents the aeration tank loading statistics based upon the plant operational data from 2002 through 2007.

Table 4-10. Aeration Tank Loading Statistics

Basis	East Aeration Tanks			West Aeration Tanks		
	Flow (MGD)	BOD_5 Loadings (lbs/day)	NH_4 –N Loadings (lbs/day)	Flow (MGD)	BOD_5 Loadings (lbs/day)	NH_4 –N Loadings (lbs/day)
Average	17.75	22,876	3,606	23.07	28,247	4,793
Maximum	47.39	60,253	7,746	47.74	68,205	8,944
98^{th} Percentile	23.86	35,058	5,174	28.36	40,860	6,609
95^{th} Percentile	22.39	31,006	4,886	26.63	37,485	6,070

Table 4-11 presents the aeration tank operational parameters including dissolved oxygen set points at different passes, field oxygen transfer efficiencies, and unit diffuse air flux under the average and maximum day conditions. These parameters were used in calculating theoretical oxygen demands in the following aeration tank capacity analysis.

Table 4-11. Aeration Tank Operational Parameters

Location	Average Conditions			Maximum Conditions		
Pass	D.O.	αSOTE	cfm/diffuser	D.O.	αSOTE	cfm/diffuser
1	0.6	10%	1.2	0.6	9%	2.0
2	2.0	18%	1.2	1.5	15%	2.0
3	3.5	18%	1.2	3.0	16%	2.0

The capacities of the existing aeration basins have been extensively studied in the Ninth Addition Facilities Plan Update prepared in 1994 and an MMSD-sponsored research project on simultaneous nitrification/denitrification in activated sludge processes in 2002. A model was developed in the Ninth Addition Facilities Plan Update to simulate the process utilizing the computer modeling program BIOSIM. The model was previously calibrated with pilot test data and predicted the oxygen demands and potential BOD and nitrogen removal in the aeration tanks. In a memorandum prepared during the Ninth Addition, dated January 16, 1995, three methods were discussed and compared in estimating the oxygen demands under maximum month and maximum day conditions. Projected oxygen demand and oxygen demand profiles were proposed for the design of the aeration system. The methods described in the memorandum were used to estimate the theoretical required oxygen demand based upon the BOD_5 and NH_4-N loadings to the aeration tanks. Theoretical oxygen demand under the maximum day, 98th percentile, and 95th percentile conditions were calculated and presented in Table 4-12.

Table 4-12. Aeration Basin Capacity Evaluation

Parameters	East Aeration Basins	West Aeration Basins
Maximum Oxygen Delivery Capacity (based on diffuser arrangements)	Pass 1 – 5,824 cfm (11,283 lbs/d) Pass 2 – 11,784 cfm (38,048 lbs/d) Pass 3 – 7,624 cfm (26,257 lbs/d) **Total – 25,232 cfm (75,588 lbs/d)**	Pass 1 – 4,960 cfm (9,609 lbs/d) Pass 2 – 12,720 cfm (41,070 lbs/d) Pass 3 – 8,228 cfm (28,337 lbs/d) **Total – 25,908 cfm (79,016 lbs/d)**
Average Oxygen Delivery Capacity (based on diffuser arrangements)	Pass 1 – 3,494 cfm (7,522 lbs/d) Pass 2 – 7,070 cfm (27,394 lbs/d) Pass 3 – 4,574 cfm (17,724 lbs/d) **Total – 15,139 cfm (52,640 lbs/d)**	Pass 1 – 2,976 cfm (6,406 lbs/d) Pass 2 – 7,632 cfm (29,570 lbs/d) Pass 3 – 4,937 cfm (19,128 lbs/d) **Total – 15,545 cfm (55,104 lbs/d)**
Theoretical Oxygen Demand (maximum day)	81,089 lbs/day	92,312 lbs/day
Current Utilization of Capacity (maximum day)	107%	117%
Theoretical Oxygen Demand (98th Percentile)	49,159 lbs/day	59,011 lbs/day
Current Utilization of Capacity (98th Percentile)	65%	75%
Theoretical Oxygen Demand (average day)	32,759 lbs/day	41,459 lbs/day
Current Utilization of Capacity (average day)	62%	75%

Table 4-12 also presents a capacity evaluation for the oxygen delivery capacity of the existing aeration tanks based up diffuser arrangement. The results show that both East and West Plant aeration tanks are slightly overloaded under the maximum day condition based upon the operational data from 2002 to 2007. At the 98th percentile and average day conditions, both plants provide sufficient aeration capacities.

Mixers are used in the anaerobic and anoxic zones to keep mixed liquor suspended, and anaerobic recycle pumps are installed in anoxic zones to recycle sludge to anaerobic zones. Tables 4-13 and 4-14 provide information on the existing mixers and recycle pumps at the East and West Plants. Tables 4-15 and 4-16 present a capacity evaluation for the existing mixers and recycle pumps at the East and West Plants. As can be seen, the existing mixers and recycle pumps have adequate capacity to accommodate current loadings.

Table 4-13. East Aeration Basin Mixer/Pump Description

Parameters	Anaerobic Mixers	Anaerobic Recycle Pumps
Number of Units	20 total 2 (No. 1) 2 (No. 4) 4 (No. 7) 4 (No. 12) 4 (No. 13) 4 (No. 18)	4 total 1 (No. 7) 1 (No. 12) 1 (No. 13) 1 (No. 18)
Type	Submersible axial flow	Submersible axial flow
Unit Flow	11,080 gpm	8,000 gpm
Speed	580 rpm	variable

Table 4-14. West Aeration Basin Mixer/Pump Description

Parameters	Anaerobic Mixers	Anaerobic Recycle Pumps
Number of Units	16 total 4 (No. 21) 4 (No. 24) 4 (No. 25) 4 (No. 28)	5 total 1 (No. 21) 1 (No. 24) 1 (No. 25) 1 (No. 28) 1 (Spare)
Type	Submersible axial flow	Submersible axial flow
Unit Flow	11,080 gpm 14,820 (for 4th zone of tank 25 and 28)	9,800
Speed	580 rpm	variable

Table 4-15. East Aeration Basin Mixer/Pump Capacity Evaluation

Parameters	Anaerobic Mixers	Anaerobic Recycle Pumps
Design Criteria	Turn over time equals to 15 -18 minutes. (or 0.02 to 0.04 hp/1,000 gal)	Provide 200% recycle of the aeration basin influent flow at maximum month conditions at maximum pump flow rate.
Current Required Capacity	10.8 minutes (Tank No.1 & 4) 11.8 minutes (Tank 7, 12, 13 & 18)	31,798 gpm
Current Utilization of Capacity	72% (Tank No.1 & 4) 79% (Tank 7, 12, 13 & 18)	99%

Table 4-16. West Aeration Basin Mixer/Pump Capacity Evaluation

Parameters	Anaerobic Mixers	Anaerobic Recycle Pumps
Design Criteria	Turn over time equals to 15 -18 minutes.	Provide 200% recycle of the aeration basin influent flow at maximum month conditions at maximum pump flow rate.
Current Required Aeration Capacity	18.2 minutes 13.6 minutes (4th zone of tank 25 and 28)	31,798 gpm
Current Percent of Capacity	100% 91% (4th zone of tank 25 and 28)	81%

Aeration blowers provide required oxygen to aeration tanks. Table 4-17 provides information on the existing aeration blowers at both East and West Plants.

Table 4-17. Aeration Blower Description

Parameters	East Engine Blower	East Blower No. 2 & 3	East Blower No. 4	East Blower No. 5	West Blower
Number of Units	1	2	1	1	3
Type	PD	Centrifugal (variable inlet)	PD	PD	Centrifugal (variable inlet)
Low Speed Capacity (cfm)	9,100 @ 700 rpm	7,000	7,800	5,900	14,000
High Speed Capacity (cfm)	11,025 @ 840 rpm	11,500	10,900	9,100	24,000

Table 4-18 presents a capacity evaluation for the existing aeration blowers at the East and West Plants. The results show that the existing blowers have adequate capacities to accommodate current loadings under average and maximum day conditions.

Table 4-18. Aeration Blower Capacity Evaluation

Parameters	East Plant	West Plant
Firm Capacity (assume the largest blower out of service)	40,000 scfm	48,000 scfm
Theoretical Required Capacity (maximum day)	27,618 scfm	31,441 scfm
Current Utilization of Capacity (maximum day)	69%	66%
Theoretical Required Capacity (average day)	9,672 scfm	12,241 scfm
Current Utilization of Capacity (average day)	24%	26%

Secondary Clarification Facilities

Effluent from the aeration tanks flows to secondary clarifiers for settling. There are 11 secondary clarifiers in the East Plant and 8 in the West Plant. The effluent of the secondary clarifiers flows to UV disinfection facilities before being discharged. The RAS is pumped to aeration tanks while WAS and scum are pumped to DAF thickeners for thickening before being digested. Tables 4-19 and 4-20 present detailed information on the existing secondary clarifiers at the East and West Plants.

Table 4-19. East Plant Secondary Clarifier Description

Parameters	No. 1-2	No. 3	No. 4-6	No. 7-10	No. 11
Number of Units	2	1	3	4	1
Diameter (ft)	70	85	85	105	105+
Unit Surface Area (sf)	3,850	5,670	5,670	8,660	8,700
Side Water Depth (ft)	12.5	13	12.5	12	12

Table 4-20. West Plant Secondary Clarifier Description

Parameters	Secondary Clarifier No. 12-19
Number of Units	8
Diameter (ft)	116
Unit Surface Area (sf)	10,562
Side Water Depth (ft)	13

Table 4-21. Secondary Clarifier Capacity Evaluation

Parameters	East Plant No. 1	East Plant No. 2	West Plant No. 3	West Plant No. 4
Total Surface Area (sf)	30,380	43,340	42,248	42,248
Design Solids Loading Rate (lbsl/sf.d)	20			
Design Hydraulic Loading Rate (gal/sf.d)	1,200			
Max Design Solids Capacity (lbs/d)	607,600	866,800	844,960	844,960
Max Design Hydraulic Capacity (MGD)	36.5	52.0	50.7	50.7
Firm Design Solids Capacity (lbs/d)	494,200	692,800	633,720	633,720
Firm Design Hydraulic Capacity (MGD)	29.7	41.6	38.0	38.0
Current Maximum Day Flow (MGD)	32.6	33.0	28.4	29.4
Current 98th Percentile Flow (MGD)	17.5	21.1	24.1	25.9
Current 95th Percentile Flow (MGD)	16.7	19.8	23.1	23.9
Current Average Day Flow (MGD)	14.4	16.4	20.5	20.7
Current Maximum Day Solids Loading (lbs/d)	951,447	862,353	1,166,539	1,290,816
Current 98th Percentile Solids Loading (lbs/d)	504,918	553,951	738,019	781,802
Current 95th Percentile Solids Loading (lbs/d)	454,533	470,760	690,047	691,596
Current Average Day Solids Loading (lbs/d)	267,431	305,961	512,852	502,348
Current Utilization of Design Max Hydraulic Capacity @ Maximum Day	89%	63%	56%	58%
Current Utilization of Design Solids Capacity @ Maximum Day	157%	99%	138%	153%

Table 4-21 provides a capacity evaluation of the existing secondary clarifiers. Operational data from 2001 to 2008 were analyzed to determine the hydraulic and solids loading statistics for all four plants. At maximum day conditions, the clarifiers have sufficient hydraulic capacities to accommodate peak flows. However, the existing clarifiers will be overloaded based upon peak solids loading. The existing clarifier solids loadings were studied in the Ninth Addition Facilities Plan Updates prepared in 1994. A design solids loading of 20 lbs/sf/d was recommended in the memorandum "Capacity Analysis of the Madison MSD NSWWTP Secondary System after EBPR Implementation" dated August 2, 1994 and it is somewhat conservative according to commonly accepted design criteria. The secondary clarifier operational data from 2001 through 2007 were studied to estimate the maximum solids capacity of the existing clarifiers.

Figures 4-3 to 4-6 presents the solids loadings to clarifiers of all 4 plants and the corresponding effluent TSS concentrations based upon the operational data from 2001 through 2007. All 4 plants have experienced events with solids loadings higher than

20 lb/sf/d, and effluent TSS concentrations after those high solids loading events all remained below 10 mg/L. Plant 1, 2 and 3 have experienced peak solids loadings higher than 28 lb/sf/d, and the effluent quality showed no sign of deterioration in terms of effluent TSS concentrations. All 4 plants have recorded high effluent TSS events ranging from 16 to 37 mg/L, but they do not appear to be associated with high influent solids loadings.

Figure 4-3. Plant 1 Secondary Clarifier Performance (2001-2008)

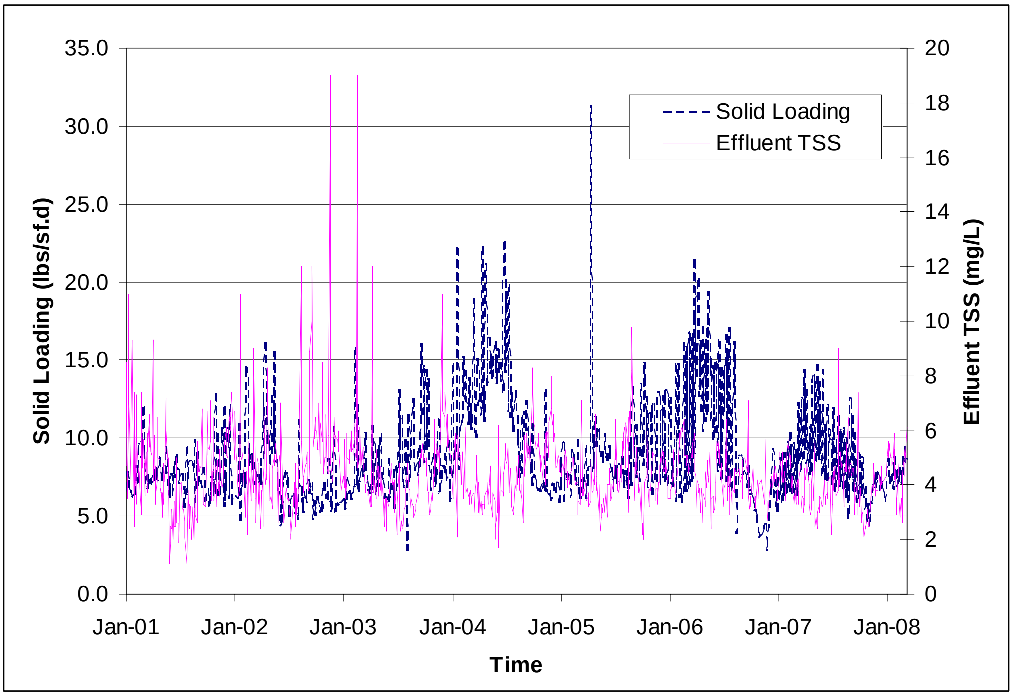


Figure 4-4. Plant 2 Secondary Clarifier Performance (2001-2008)

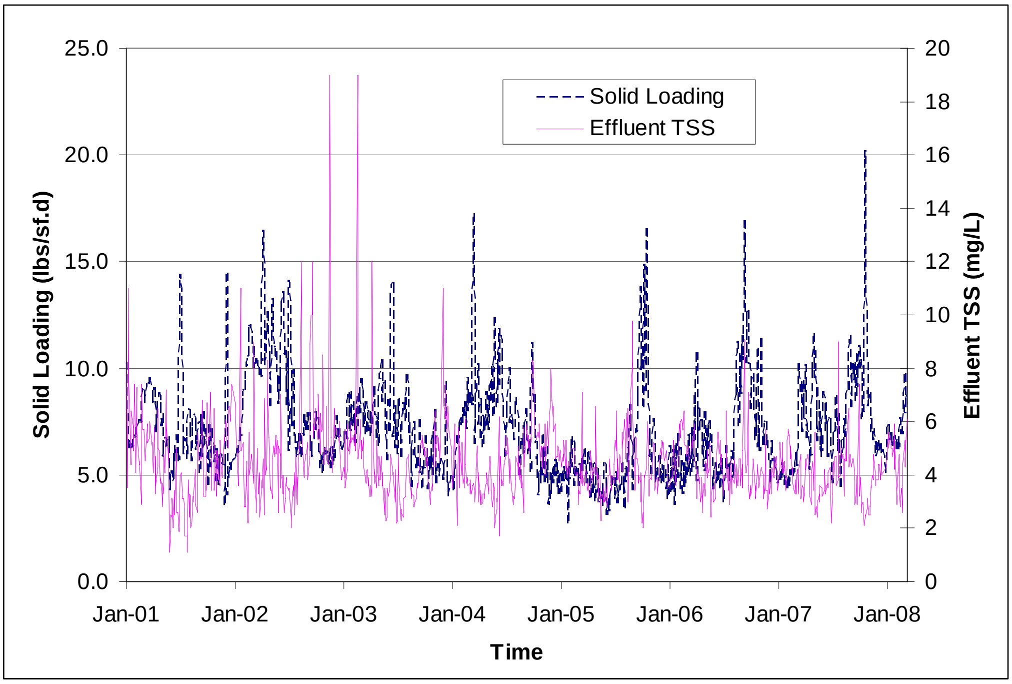


Figure 4-5. Plant 3 Secondary Clarifier Performance (2001-2008)

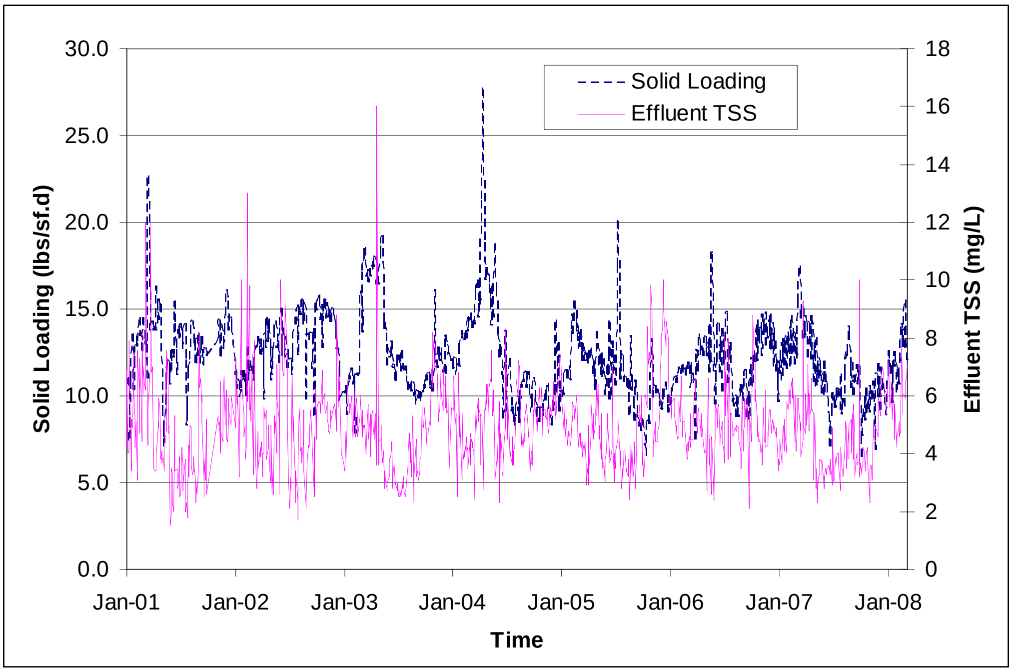
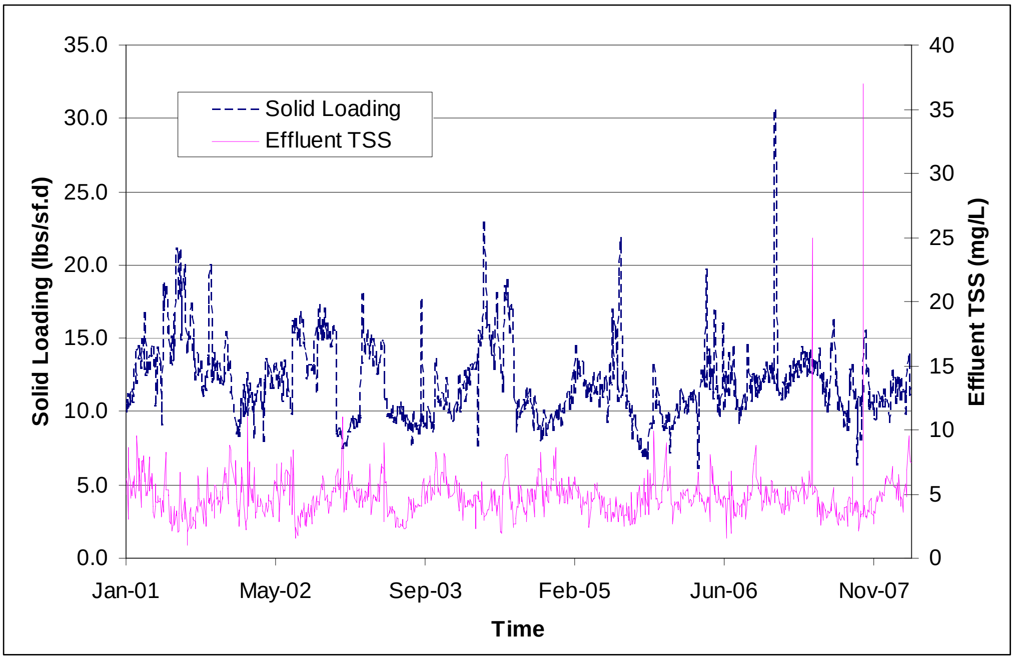


Figure 4-6. Plant 4 Secondary Clarifier Performance (2001-2008)



The method documented in a technical paper titled *The Relationship between SVI and Activated Sludge Settling Characteristics* by G. T. Daigger and R. E. Roper, Jr. was used to further estimate the maximum solids capacities of the existing secondary clarifiers. This method established the relationship between SVI data by activated sludge batch settling test and limiting flux for solids thickening in secondary clarifiers. The determined limiting flux based on activated sludge SVI data and clarifier underflow was then used to estimate the maximum solids capacities of the secondary clarifiers. The activated sludge SVI data for all 4 plants based upon operational data during January, 2001 and December, 2007 is presented in Table 4-22.

Table 4-22. Activated Sludge SVI Data

Parameter	Plant 1	Plant 2	Plant 3	Plant 4
Maximum SVI (mL/g)	124	134	123	141
98th Percentile SVI (mL/g)	120	115	110	134
95th Percentile SVI (mL/g)	115	110	103	121
Average SVI (mL/g)	92	83	81	87

According to the operational data, the aeration tank effluent exhibited very good settleability thanks to the implementation of bio-selectors and biological nutrient removal processes. The average activated sludge SVI for all 4 plants have been staying below 100 mL/g for the past 6 years. The method described in the referred paper was then used to estimate the maximum solids capacities of the existing secondary clarifiers. The analysis was based on the assumption that the activated sludge average SVI will stay below 100 mL/g with increased flow and solids loadings. The results are shown in Figure 4-7 and Table 4-23.

Figure 4-7. Secondary Clarifier Solids Loading Capacity Estimation

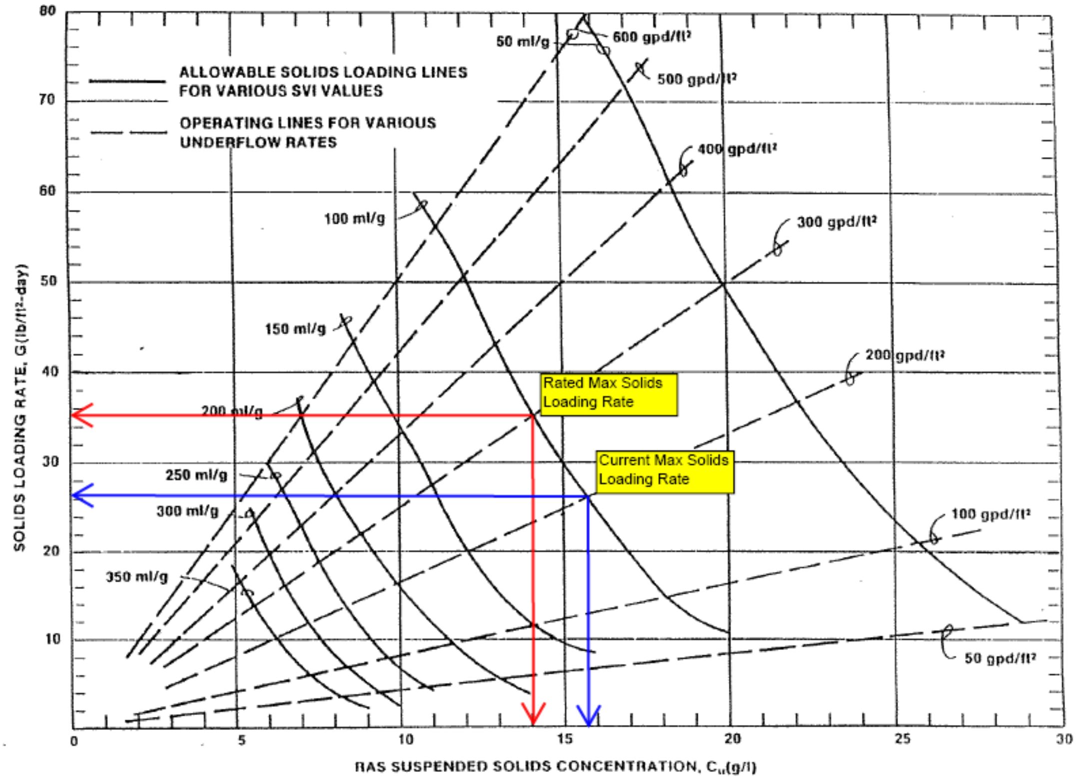


Table 4-23. Estimated Secondary Clarifier Solids Loading Capacities

Parameter	Underflow Rate (gpd/sf)			
	200	300	400	500
Underflow (MGD)	31.6	47.5	63.3	79.1
Maximum Solids Loading (lb/day/sf)	26.0	35.2	43.6	51.0
Maximum RAS TSS Concentration (mg/L)	15,600	14,100	13,050	12,200

According to the sedimentation flux theory, the clarifier limiting solids flux increases as underflow rises. Currently the plant operates at an average underflow of approximately 200 gpd/sf, which equals to 31.6 MGD assuming all secondary clarifiers are in service. Under this condition, the existing clarifiers have a solids capacity of 26.0 lb/day/sf or 4,113,616 lb/day with all clarifiers in service. More solids capacities can be obtained by increasing the underflow rate. However higher underflow rates require more recycled pumping and energy consumption. Also with increased flow and solids loadings, the activated sludge settleability (i.e. SVI) may change and affect the overall clarifier solids capacities. Selection of appropriate underflow and solids loading is based upon the analysis on the operational data and professional judgment. Based on the analysis, the maximum solids loading of the existing clarifiers was rated as 35.0 lb/day/sf. The rated loading should be verified when operational data under higher solids loadings becomes available in the future. Full scale solids capacity test is recommended to further determine the capacities of the existing secondary clarifiers. The recommended secondary clarifier capacities are presented in Table 4-24.

Table 4-24. Secondary Clarifier Recommended Capacity

Parameters	East Secondary Tanks		West Secondary Tanks	
	Plant 1	Plant 2	Plant 3	Plant 4
Recommend Maximum Hydraulic Loading (gal/sf/day)	1,200			
Recommended Maximum Hydraulic Capacity (MGD)	36.5	52.0	50.7	50.7
Current Utilization of Maximum Hydraulic Capacity @ Maximum Day	89%	63%	56%	58%
Recommended Firm Hydraulic Capacity (MGD)	29.7	41.6	38.0	38.0
Current Utilization of Firm Hydraulic Capacity @ Maximum Day	110%	79%	75%	77%
Combined Maximum Hydraulic Capacity (MGD)	190.0			
Combined Firm Hydraulic Capacity (MGD)	147.3			
Recommend Maximum Solids Loading (lbs/sf/day)	35.0			
Recommended Maximum Solids Capacity (lbs/day)	1,063,300	1,516,900	1,478,680	1,478,680
Current Utilization of Maximum Solid Capacity @ Maximum Day	89%	57%	79%	87%
Recommended Firm Solids Capacity (lbs/day)	864,850	1,212,400	1,109,010	1,109,010
Current Utilization of Firm Solid Capacity @ Maximum Day	110%	71%	105%	116%
Combined Maximum Solids Capacity (lbs/day)	5,537,560			
Combined Firm Solids Capacity (lbs/day)	4,295,270			

According to the evaluation presented in Table 4-24, the existing clarifiers have sufficient hydraulic and solids capacities for current loadings at maximum day condition. However Plant Nos. 1, 3, and 4 will be overloaded in terms of solids loadings at maximum day condition with one clarifier out of service at each plant.

Table 4-25 and 4-26 present detailed information on the existing RAS and WAS pumps at the East and West Plants.

Table 4-25. East Plant RAS/WAS Pumps Description

Parameters	Plant 1 RAS Pumps	Plant 2 RAS Pumps	Plant 1 WAS Pumps	Plant 1 WAS Pumps	Plant 2 WAS Pumps
Number of Units	3	3	2	1	2
Type	Centrifugal	Centrifugal	Centrifugal	Centrifugal w/ VFD	Centrifugal w/ VFD
Unit Capacity (gpm)	3,500	5,200	1,300	175	850
TDH (ft)	30	20.5	23	25	40

Table 4-26. West Plant RAS/WAS Pumps Description

Parameters	RAS Pumps	WAS Pumps
Number of Units	3	3
Type	Centrifugal	Centrifugal w/ VFD
Unit Capacity (gpm)	3,500	850
TDH (ft)	30	40

Table 4-27. RAS/WAS Pumps Capacity Evaluation

Parameters	RAS Pumps	WAS Pumps
Design Criteria	The RAS pumping capacity shall cover the range between 25 and 75 percent of the daily average flow.	The maximum sludge pumping rate shall be at least 200% of the volumetric sludge production rate.
Maximum Capacity	36,600 gpm	6,175 gpm
Firm Capacity	27,900 gpm	4,025 gpm
Required Pumping Capacity	21,198 gpm (75% of 40.7 MGD)	1,500 gpm
Current Utilization of Firm Capacity @ Daily Average Flow	77%	37%

Table 4-27 provides a capacity evaluation of the RAS and WAS sludge pumps. The existing pumps provide adequate pumping capacities to accommodate the current loadings.

UV Disinfection Facilities

UV disinfection facilities disinfect the effluent from the secondary clarifiers. The existing UV disinfection system is an open channel, low pressure mercury vapor type. There are a total of 7 channels, 5 of which are installed with UV disinfection equipment, one is reserved for future equipment, and the seventh channel is used as a by-pass channel when the UV system is out of service. Each of the five channels has two UV banks in series. Normally two to four channels are in service with one bank of lamps operational. During peak flow rates, additional UV channels are added to meet the flow demands. Channels are brought online and taken offline by automatically controlled motorized gates installed on the inlet of each channel.

Table 4-28 presents detailed information on the existing UV disinfection facilities.

Table 4-28. UV Disinfection Facility Description

Parameters	Values
Number of Operational Channels	5
Number of Bypass Channels	1
Number of Future Channels	1
Type	Open channel, low pressure mercury vapor
Number of Banks per Operational Channel	2
Number of Lamps per Bank	368
Total Number of Lamps	3,680
UV Transmission	65%

Table 4-29. UV Disinfection Facility Capacity Evaluation

Parameters	Values
Design Effluent TSS	20 mg/L
Ninth Addition Design Dosage	30,121 Microwatt-sec/cm^2
Effluent Fecal Coliform Count	400 counts per 100 ml
Unit Channel Design Capacity	20 MGD
Design Peak Capacity	100 MGD
Current Hydraulic Loading @ Max Day	95.0 MGD
Current Utilization of Capacity	95%

Table 4-29 provides a capacity evaluation of the existing UV disinfection facilities. At maximum day conditions, the existing facilities provide adequate capacity to accommodate the current hydraulic loading.

Plant Effluent Pumping Facilities

The existing effluent pumping facilities were constructed during the plant's Seventh Addition. The plant effluent is pumped to Badfish Creek through a 54" force main and to Badger Mill Creek through a 20" force main. The Badfish Creek effluent pumps consist of five horizontal split case centrifugal pumps, each with an 800 hp, 880 rpm motor. Three pumps are outfitted with 25.94" diameter impellers and two are equipped with impellers trimmed to 24" to save energy when lower flow rates are practicable. The Badger Mill Creek effluent pumps include two centrifugal pumps, each with a 200 hp, variable speed motor. Each pump has a capacity of 2,000 gpm at 190 psi. Tables 4-30 and 4-31 provide the detailed information on the Badfish Creek effluent pumps and Badger Mill Creek pumps.

Table 4-30. Badfish Creek Effluent Pump Description

Parameters	Large Pumps	Small Pumps
Number of Units	3	2
Type	Centrifugal	Centrifugal
Impeller Diameter	25.94 “	24”
Speed	880 rpm	880 rpm
Unit Capacity	25,000 gpm @ 100 psi	22,000 gpm @ 95 psi

Table 4-31. Badger Mill Creek Effluent Pump Description

Parameters	Values
Number of Units	2
Type	Centrifugal
Speed	Variable
Unit Capacity	2,000 gpm @ 190 psi

Table 4-32 presents an evaluation of the capacities of the existing effluent pumps. The existing effluent pumps do not have adequate capacity to accommodate current maximum day or maximum hour flows; however, effluent equalization facilities are available to mitigate this capacity issue.

Table 4-32. Plant Effluent Pump Capacity Evaluation

Parameters	Badfish Creek Pump	Badger Mill Creek Pump
One Small Pump Operating	22,000 gpm @ 95 psi	
One Large Pump Operating	25,000 gpm @ 100 psi	
One Large and One Small Pump Operating	39,558 gpm @ 100 psi	
Two Large Operating	43,722 gpm @ 133 psi	
One Small & Two Large	49,274 gpm @ 146 psi	
Three Large	52,744 gpm @ 155 psi	
Maximum Capacity	52,744 gpm (76.0 MGD)	4,000 gpm (5.76 MGD)
Total Maximum Capacity	81.7 MGD	
Current Effluent Flow @ Max Day	97.6 MGD	
Current Utilization of Capacity @ Max Day	119%	
Current Effluent Flow @ Max Hour	117.1 MGD	
Current Utilization of Capacity @ Max Hour	143%	

Effluent Storage Facilities

The plant has two effluent storage tanks and an effluent storage lagoon for plant effluent storage. The disinfected effluent beyond effluent pumping capacity and up to an estimated flow rate of 115 MGD overflows to effluent storage reservoirs. The effluent storage reservoirs, in turn, overflow to the effluent storage lagoon when their maximum storage capacities are reached. Flows in excess of 115 MGD (estimated) receive secondary treatment, but are not disinfected. This estimated flow rate is based on a flow split at the flow splitter of 45 percent to the east side of the plant and 55 percent to the west side of the plant. At a total flow of 115 MGD, the east plant flow would be 52 MGD which is the flow rate from the east side final clarifiers at which bypassing of secondary effluent was observed previously. The excess flows above 115 MGD are diverted to the effluent equalization facilities. The effluent equalization facilities have a volume of 66 MGD. When this volume is exceeded, an overflow structure diverts additional flows to the ditch on the north side of the lagoons. Flow in the ditch goes to Nine Springs Creek and then Lake Waubesa. Discharges to the equalization facilities are pumped back to the secondary process when the plant peak flow subsides. Effluent storage volume is reduced by 1.3 million gallons for each inch of precipitation. Table 4-33 provides the detailed information on the effluent storage facilities.

Table 4-33. Effluent Storage Facility Description

Parameters	Effluent Storage Reservoirs	Effluent Storage Lagoon
Number of Units	2	1
Unit Volume (gallons)	650,000	66,000,000
Total Volume (gallons)	1,300,000	66,000,000
Minimum Water Level (ft)	6.75	-1.50
Maximum Water Level (ft)	12.0	3.53
Total Equalization Volume (gallons)	650,000	66,000,000

5. Solids Handling and Disposal

The majority of the existing solids handling and disposal facilities were improved during the plant’s Tenth Addition. Primary sludge is thickened by two gravity thickeners, and WAS is thickened by two DAF thickeners. Thickened sludge is pumped to Digester No. 7 (acid digester), then pumped sequentially to three thermophilic digesters in the East Complex, and then pumped sequentially to three mesophilic digesters located in the West Complex. Digested sludge flows by gravity to two sludge storage tanks. From the sludge storage tanks the sludge is pumped to either the GBT building for thickening or the centrifuge building for dewatering. Thickened sludge is pumped to the Metrogro Storage Tanks prior to liquid land application. Dewatered sludge is conveyed to the Biosolids End-Use Production Facility for further processing.

Gravity Thickeners

Primary sludge is pumped into the gravity thickeners for thickening. The thickened sludge pumps operate continuously, typically one per thickener, to convey the thickened sludge to the anaerobic digester feed header, where it combines with thickened WAS. The combined stream is fed to the acid digester. Operators manually adjust pump speed to maintain appropriate sludge blanket levels in the thickeners and minimum sludge flows to the acid digester. Supernatant from the gravity sludge thickeners flows by gravity to the East Plant primary clarifiers. Table 5-1 presents detailed information on the existing gravity thickeners.

Table 5-1. Gravity Thickeners Equipment Description

Parameters	Gravity Thickeners
Number of Units	2
Type	Circular
Depth	10 feet sidewall, 15 feet center
Diameter	55 feet
Unit Surface Area	2,376 square feet
Total Surface Area	4,752 square feet

Table 5-2 provides an evaluation on the capacity of the existing gravity thickeners. With both units in service, the thickeners have sufficient hydraulic capacity to accommodate current loadings but inadequate solids capacity for the current maximum week condition. The plant staff indicated that the existing gravity thickeners have been working well in dewatering the primary sludge although the maximum week solids loading are slightly over the design capacity. This suggests that the existing gravity thickeners may have potential to accommodate solids loadings higher than the design values. Further tests are recommended to determine the maximum solids capacities of the existing gravity thickeners.

Table 5-2. Gravity Thickeners Capacity Evaluation

Parameters	Maximum Capacity	Firm Capacity
Design Solids Loading	25 lb/sf/day	
Design Hydraulic Loading	600 gpd/sf	
Design Solids Capacity	118,800 lb/day	59,400 lb/day
Design Hydraulic Capacity	1,980 gpm	990 gpm
Current Solids Loading @ Maximum Week	121,778 lb/day	
Current Hydraulic Loading @ Maximum Week	500 gpm	
Current Utilization of Solids Capacity @ Maximum Week	103%	206%
Current Utilization of Hydraulic Capacity @ Maximum Week	25%	50%
Current Solids Loading @ 98th Percentile Value	91,477 lb/day	
Current Utilization of Solids Capacity @ 98th Percentile Value	75%	150%
Current Solids Loading @ 95th Percentile Value	67,129 lb/day	
Current Utilization of Solids Capacity @ 95th Percentile Value	55%	110%

Table 5-3 describes the technical details of the gravity thickening sludge pumps.

Table 5-3. Gravity Thickening Sludge Pumps Equipment Description

Parameters	Sludge Pumps
Number of Units	4
Type	Positive Displacement
Rated Capacity per Pump	100 gpm
Pumps Serving Gravity Thickeners	2 pumps are dedicated to each gravity thickener

Table 5-4 provides a capacity evaluation of the equipment. At maximum week condition the thickened sludge pumps have adequate pumping capacity under the current loading conditions with both sludge pumps in operation. When one pump is out of service, the other pump will be overloaded at the maximum week condition.

Table 5-4. Gravity Thickening Sludge Pumps Capacity Evaluation

Parameters	Maximum Capacity	Firm Capacity
Design Criteria	Transfer expected range of thickened primary sludge to the digesters	
Pumping Capacity per Gravity Thickener	200 gpm	100 gpm
Current Required Pumping Capacity per Gravity Thickener at 6.0% Solids (maximum week)	169 gpm	
Current Utilization of Capacity	85%	169%

Dissolved Air Flotation Thickeners

Waste activated sludge is continuously pumped to the dissolved air flotation (DAF) thickeners by the WAS pumps. Primary and secondary scum is also periodically pumped to the flotation thickener by the scum pneumatic ejectors. The flotation thickener components such as recirculation pumps, the skimmers, and the air compressor system operate continuously. The thickened sludge is transported to the thickened sludge wet well, and periodically pumped out by the DAF thickened sludge pumps. The thickener subnatant flows by gravity to the DAF Building and then to the east primary clarifiers.

Table 5-5 provides information on the DAF thickeners.

Table 5-5. DAF Thickeners Equipment Description

Parameters	DAF Thickeners
Number of Units	2
Type	Circular
Diameter	55 feet
Total Surface Area	4,752 square feet

Table 5-6 provides a capacity evaluation of the DAF thickeners. The DAFs have adequate capacity at current loadings with both units in service under maximum week condition. When one thickener is out of service, the other one will be overloaded under the maximum week conditions.

Table 5-6. DAF Thickeners Capacity Evaluation

Parameters	Maximum Capacity	Firm Capacity
Design Solids Loading	17.5 lb/day/sf (no polymer used)	
Design Hydraulic Loading	0.65 gpm/sf (no polymer used)	
Recycle Flow per DAF	625 gpm	
Design Solids Capacity	83,160 lbs/day	41,580 lbs/day
Design Hydraulic Capacity	3,089 gpm	1,544 gpm
Current Solids Loading @ Max Week	59,580 lb/day	
Current Hydraulic Loading @ Max Week	1,480 gpm	
Current Utilization of Solids Capacity @ Max Week	72%	143%
Current Utilization of Hydraulic Capacity @ Max Week	88%	177%
Current Solids Loading @ 98th Percentile Value*	56,350 lb/day	
Current Utilization of Solids Capacity @ 98th Percentile Value	68%	136%
Current Solids Loading @ 95th Percentile Value*	54,354 lb/day	
Current Utilization of Solids Capacity @ 95th Percentile Value	65%	130%

Table 5-7 presents information on the existing DAF thickened sludge pumps.

Table 5-7. DAF Thickened Sludge Pumps Equipment Description

Parameters	Sludge Pumps
Number of Units	4
Type	Positive Displacement
Rated Capacity per Pump	100 gpm
Pumps Serving DAF Thickeners	2 pumps are dedicated to each DAF thickener

Table 5-8 provides an analysis on the capacity of the existing DAF thickened sludge pump capacity. The existing pumps have adequate capacity to accommodate current loading under maximum week conditions with both pumps in operation.

Table 5-8. DAF Thickened Sludge Pumps Capacity Evaluation

Parameters	Maximum Capacity	Firm Capacity
Design Criteria	Transfer expected range of thickened primary sludge to the anaerobic digesters	
Pumping Capacity per Gravity Thickener	200 gpm	100 gpm
Current Required Pumping Capacity per DAF Thickener at 4.2% Solids (maximum week)	110 gpm	
Current Percent of Capacity	55%	110%

Table 5-9 presents information on the existing DAF air supply equipment.

Table 5-9. DAF Air Supply Equipment Description

Parameters	Recirculation Pump	Air Compressor	Air Saturation Tank
Number of Units	3, one for each DAF, one standby	2, one work, one standby	2, one for each DAF
Type	Centrifugal	n/a	Vertical
Rated Unit Capacity	530 gpm @ 60 psi	75 cfm @ 200 psig 15 hp	200 gallons 200 psi

Table 5-10 provides an analysis on the capacity of the existing DAF air supply equipment capacity. The existing air compressors have adequate capacity to accommodate current loading under maximum week conditions while the recirculation pumps and air saturation tanks are overloaded.

Table 5-10. DAF Air Supply Equipment Capacity Evaluation

Parameters	Recirculation Pump	Air Compressor	Air Saturation Tank
Design Criteria	Air/Solids=0.02	Air/Solids=0.02	30-60 second retention time
Current Max Week Sludge Feed Rate	1,480 gpm		
Current Max Week Solids Loading	59,580 lb/day		
Current Required Capacity	2,666 gpm @ 60 psi	40 cfm @ 200 psig	740 gallons (30 second retention time)
Current Max Capacity	1,060 gpm @ 60 psi	150 cfm @ 200 psig	400 gallons
Current Percent of Max Capacity	252%	26%	185%

Anaerobic Digesters

The Temperature Phased Anaerobic Digestion (TPAD) system was implemented during the Tenth Addition. The digestion system was originally configured to consist of thermophilic digestion followed by mesophilic digestion. It was intended to operate with sequential batch feeding and withdrawal to produce Class A biosolids. Due to various operational problems, the digestion process is currently being reconfigured.

Anaerobic digestion at the plant occurs in two locations, the East Complex and the West Complex. The East Complex consists of three thermophilic digesters (Nos. 4, 5 and 6) and an acid digester (No. 7), among which the No. 6 digester was converted from an old sludge storage tank, and No. 7 digester was constructed during the Tenth Addition. The West Complex consists of three mesophilic digesters (No. 1, 2, and 3). Thickened primary sludge and WAS are continuously pumped into the acid digester, and then batched into the three thermophilic digesters. The three thermophilic digesters were designed to operate in a sequencing batch cycle including a fill, hold, and drawdown period. A cycle would vary between 12 hours and 24 hours. The thermophilic digesters are not currently operated in a batch mode. Alternate digester operations are being evaluated for producing a Class A product. Research is in progress to demonstrate that shorter batch times and lower thermophilic temperatures (~128 degrees F) can produce a Class A biosolids under alternative EPA regulations. Table 5-11 provides information on the existing anaerobic digesters.

Table 5-11. Anaerobic Digester Description

Parameters	East Digesters (No. 4, 5, 6 & 7)	West Digesters (No. 1, 2 & 3)
Number of Units	4	3
Type	Fixed concrete cover	Fixed concrete cover
Diameter	80	75
Side Water Depth	28	16
Cone Depth	2.67	12.5
Unit Volume	1,086,000 gallons	659,000 gallons
Total Volume	3,259,000 gallons (Thermophillic) 1,086,000 gallons (Acid Mesophilic)	1,976,000 gallons (Mesophilic)

Table 5-12 presents the analysis on the capacities of the existing anaerobic digesters. Although the digesters have adequate design capacities to accommodate current solids loadings, they have experienced a variety of operational problems including decreased methane production, foaming in the acid digester, clogging in the sludge piping and heat exchangers, etc.

Table 5-12. Anaerobic Digester Capacity Evaluation

Parameters	Design Criteria	Current Condition (Max Month)	Capacity Utilization
Sludge Flow (gpd)	389,000	315,100*	81%
Solid Feed (lb/day)	153,000	118,243*	77%
Thermophilic SRT – East (days)	7.4	10.3	--
Mesophilic (Acid) SRT – East (days)	1.9	2.3	65.6%
Mesophilic SRT – West (days)	5.1	6.3	--
Total SRT (days)	15.3	20.0	--
Solids Loading Rate (lb VSS/1,000 cf)	135	105	77%
Min. VSS Destroyed (%)	55	--	--
Max. VSS Destroyed (%)	63	--	--
Min. Gas Production (cf/day)	867,000	240,580	--
Max. Gas Production (cf/day)	993,000	947,782	--

* *Current loadings based upon maximum month conditions from operational data from January 2002 through December 2006.*

Acid to Thermo pumps transfer sludge from acid digester to three thermophilic digesters. Thermo to Meso pumps transfer sludge from thermophilic digesters to mesophilic digesters. Table 5-13 provides information on digester sludge transfer pumps.

Table 5-13. Digester Sludge Transfer Pump Description

Parameter	Thermo to Meso Pumps	Booster Pumps	Acid to Thermo Pump A	Acid to Thermo Pump B
No. of Units	2	1	2	1
Type	PD (2 stage)	Centrifugal	Centrifugal	PD (2 stage)
Unit Capacity	395 gpm	160-340 gpm	160-500 gpm	340 gpm
Rated Head	80 psi	40-65 ft	40-100 ft	80 psi
Sludge Concentration (%)	1.5 – 3.0	1.5 – 3.0	2.5 – 4.0	2.5 – 4.0
Speed (rpm)	250 (variable)	1,800	1,800 (Variable)	320 (variable)

Table 5-14 provides an analysis on the capacity of digester sludge transfer pumps. The existing sludge transfer pumps have adequate capacities under current conditions.

Table 5-14. Digester Sludge Transfer Pump Capacity Evaluation

Parameter	Thermo to Meso Pumps	Booster Pumps	Acid to Thermo Pump A	Acid to Thermo Pump B
Maximum Capacity	790 gpm	340 gpm	1,000 gpm	340 gpm
Firm Capacity	395 gpm	--	500 gpm	
Current Required Pumping Capacity	270 gpm	--	270 gpm	
Current Utilization of Firm Capacity	68%	--	68%	

The mixers in both the east and west digester complexes operate continuously. Table 5-15 provides information on the digester mixing equipment. Table 5-15 provides a capacity evaluation of the equipment. The east digester mixers have sufficient capacity for the current loading while the west digester mixers are slightly undersized based upon commonly used design criteria.

Table 5-15. Digester Mixing Facility Description

Parameters	East Digesters (No. 4, 5 & 6)	East Digester (No. 7)	West Digesters (No. 1, 2 & 3)
Number of Units	7 per digester	7 per digester	2 per digester
Type	Confined gas	Confined gas	Mechanical
Unit Pumping Capacity	4,200 gpm	4,200 gpm	10,000 gpm
Total Pumping Capacity	29,400 gpm	29,400 gpm	20,000 gpm
Total Gas Flow	305 scfm	--	--
Turnover Rate	45 minutes	45 minutes	33 minutes
Compressor Total Horsepower	40 hp	40 hp	--
Mixer Motor Total Horsepower	--	--	20 hp

Table 5-16. Digester Mixing Facility Capacity Evaluation

Parameters	East Digesters (No. 4, 5 & 6)	East Digester (No. 7)	West Digesters (No. 1, 2 & 3)
Design Criteria	0.25 hp per 1,000 cf of digester volume		
HP per Digester	40	40	20
Current Mixing Requirement	36.3	36.3	22.0
Current Percent Utilization	91%	91%	110%

Digester Gas Compression, Treatment and Storage

Digester gas from Acid Digester No. 7 passes through a foam separator and is compressed in the East Gas Control Room in Sludge Control Building No. 2. This gas is then used for mixing this digester with excess gas piped to Aeration Tank No. 17 where it is diffused through coarse bubble Sanitaire diffusers into the mixed liquor to strip the odorous compounds from the gas.

Digester gas is collected from the three thermophilic digesters nos. 4, 5, and 6, and piped to the gas control rooms in Sludge Control Building No. 2. The gas from each digester passes through a dedicated foam separator and a mixing compressor and is used to mix the contents of these three digesters. Excess gas is diverted to gas boosters that raise the gas pressure for transport to the gas treatment facilities outside of Sludge Control Building No. 2.

Gas treatment includes hydrogen sulfide removal using iron-impregnated wood chips, moisture removal using a condenser and chiller, and siloxane removal via carbon filtration. Gas from the treatment system is reheated to 80 degrees F and returned to the gas system at a pressure of 4 psi.

Treated gas is used as fuel for six hot water boilers (three in the Boiler Building and three in Sludge Control Building No. 2) and three gas engines. Two gas engines in Sludge Control Building No. 2 drive 475 KW induction generators. The third engine in the East Blower Building drives a positive displacement air blower. Surplus gas is flared through a waste gas flare, located near Sludge Control Building No.2.

When digester gas pressure and storage volume is low, a natural gas blending system is started and blended gas is used to supplement the digester gas. This gas is generally used to fuel the three boilers in the Boiler Building and the blower engine.

Gas storage is provided at the two sludge storage tanks at the West Complex. When gas pressure reaches approximately 7.5 inches water column (w.c.), the gas holder covers will begin to rise and store gas. When the cover exceeds 75 percent full, the pressure will begin to rise until it reaches 9.2 inches w.c. When the covers are full, the waste gas is flared, or the gas will release around the sides of the floating covers.

Table 5-17 provides information on the digester gas storage facilities.

Table 5-17. Digester Gas Storage Description

Parameters	Sludge Storage Tank
Number of Units	2
Type	Gas holder cover
Unit Gas Storage Volume	32,200 cf
Total Gas Storage Volume	64,400 cf

Table 5-18 provides a capacity evaluation of the digester gas storage facilities.

Table 5-18. Digester Gas Storage Capacity Evaluation

Parameters	Discussion
Design Criteria	Provide capacity to equalize daily variation in digester gas production
Available Digester Gas Equalization	64,400 cf
Current Daily Digester Gas Production	n/a
Current Daily Digester Gas Production Variability	n/a
Storage Percent of Current Variability	n/a

Sludge Heating System

Digester heating for the East Complex is provided by heat recovered from engine generators located in Sludge Control Building No. 2 and an engine driven blower located in the East Blower Building. Heat recovered from the engine generators is the primary heat source. Three boilers located in the Sludge Control Building No. 2 provide supplemental heating for the process hot water system. Each boiler has a rated capacity of 5.9 MMBtu per hour, or 50% of the total process heating requirements with one of the boilers used as a standby. Digester heating requirements for the West Zone is minimal as a result of being operated as mesophilic digestors. The existing process heating system is reserved as a backup in case there is a future need for heating Digesters No.1, 2, and 3. The three boilers in the Boiler Building have a rated unit capacity of 4.3 MMBtu per hour and will be sufficient for meeting the heating requirements for West Zone.

Table 5-19 provides information on the digester heating system at the East Digester Complex. Table 5-20 provides an analysis of the capacities of the existing heat exchangers and recirculation pumps at the East Digestion Complex.

Table 5-19. East Digester Heating System Description

Parameters	Heat Exchangers HEX-4, 5 & 6	Heat Exchangers HEX-7	Heat Exchangers HEX-8, 9 & 10	Heat Exchangers HEX-11 & 12	Digester Recirculation Pumps
Number of Units	3	1	3 (1 standby)	2 (1 standby)	3
Type	Spiral	Spiral	Tube and Shell	Tube and Shell	Centrifugal
Unit Capacity	1.65 MMBTU/Hr	0.5 MMBTU/Hr	5.4 MMBTU/Hr	6.1 MMBTU/Hr	n/a
Sludge Flow Rate		270 gpm	162-270 gpm	162-270 gpm	500 gpm
Function	No. 4, 5, & 6 digester recirculation	No. 7 digester recirculation	Raw sludge preheating	Raw sludge supplemental heating	No. 4, 5 & 6 digester recirculation

Table 5-20. East Digester Heating System Capacity Evaluation

Parameters	Heat Exchangers HEX-4, 5 & 6	Heat Exchangers HEX-7	Heat Exchangers HEX-8, 9 & 10	Heat Exchangers HEX-11 & 12	Digester Recirculation Pumps
Function	Maintain thermophilic temperature in Digester No. 4, 5 & 6	Maintain thermophilic temperature in Digester No. 7	Preheat raw sludge	Raise sludge to thermophilic temperature	Maintain thermophilic temperature in Digester No. 4, 5 & 6
Current Sludge Rate (max month)	1.65 MBTU/hr	n/a	269 gpm	269 gpm	500 gpm
Current Utilization	~67%	n/a	99%	99%	100%

Table 5-21 provides information on the existing heat exchangers and recirculation pumps at the West Digestion Complex. Table 5-22 provides information on the existing boilers for biosolids process heating.

Table 5-21. West Digester Heating System Description

Parameters	Heat Exchangers HEX-1, 2 & 3	Recirculation Pumps
Number of Units	3	3
Type	Spiral	Centrifugal
Unit Capacity	1.53 MMBTU/Hr	500 gpm
Function	Digester 1, 2 & 3 recirculation	Digester 1, 2 & 3 recirculation

Table 5-22. East and West Zone Boiler Description

Parameters	East Zone (SCB No. 2)	West Zone (Boiler Bldg)
Number of Units	3	3
Type	Flex Tube	Fire Tube
Unit Capacity	5.9 MMBTU/Hr	4.3 MMBTU/Hr

Digested Sludge Storage Tanks

The digested storage tanks provide a reservoir for digested sludge and digester gas to facilitate downstream sludge dewatering and gas utilization operation. Biosolids flow by gravity from the mesophilic digesters to the storage tanks. Digester gas is stored in the floating gas holder covers.

Table 5-23 provides information on the existing digested sludge storage tanks.

Table 5-23. Sludge Storage Tank Description

Parameters	Values
Number of Units	2
Type	Floating gas holder cover
Diameter	70 ft
Side Wall Depth	12 ft
Unit Volume	450,000 gallons
Total Volume	900,000 gallons

Table 5-24 provides a capacity evaluation of the existing digested sludge storage tanks. The existing sludge storage tanks provide adequate equalization volume.

Table 5-24. Digested Sludge Storage Tank Capacity Evaluation

Parameters	Values
Design Criteria	Provide capacity to equalize daily variation in biosolids production
Available Biosolids Equalization	300,000 gallons
Current Daily Biosolids Volume Variability	Negligible, primary and WAS pumps operate constantly
Current Percent of Capacity	Negligible

Gravity Belt Thickeners (GBT)

The GBT feed pumps pump sludge from either sludge storage tank to GBTs for thickening. One of the GBTs also serves as a backup to the DAF thickeners in the event one of the DAF thickeners is out of service. The thickened sludge is transferred to the Metrogro Storage Tanks. GBT filtrate and belt washwater flow by gravity to the recycle wet well. The GBT recycle pumps pump recycled water to the plant flow splitter.

The GBT polymer system consists of both a dry polymer feed process and a liquid feed process and is located in the GBT Building and the GBT Polymer Building. The polymer system is sized for two GBTs operating at their peak capacity. When one of the DAF thickeners is out of service, polymer can be pumped to the operating DAF thickener to assist in thickening of the WAS.

Table 5-25 presents information on the existing GBT system.

Table 5-25. GBT System Description

Parameters	GBT	GBT Pumps	GBT Cake Pumps	GBT Recycle Pumps
Number of Units	2	3 (1 stand-by)	2 (1 stand-by)	2 (1 stand-by)
Type	Gravity Belt	Progressing Cavity	Progressing Cavity	1 Centrifugal constant speed 1 Centrifugal variable speed
Belt Width	2 meters	--	--	--
Speed	--	2 @ 275 rpm, adjustable speed 1 @ 270 rpm adjustable speed	275 rpm, adjustable speed	--
Rated Unit Capacity	250 gpm 3,300 lb/hr	2 @ 160 gpm @ 275 rpm 1 @ 325 gpm @ 270 rpm	160 gpm per pump	2,269 gpm at full speed

Table 5-26 provides a capacity evaluation of the existing GBT system. The system has sufficient capacities for the current maximum month loadings with both units in operation.

Table 5-26. GBT System Capacity Evaluation

Parameters	GBT	GBT Feed Pump Capacity	GBT Cake Pump Capacity	GBT Recycle Pump Capacity
Max Design Capacity	500 gpm 6,600 lb/hr	480 gpm	400 gpm	2,269 gpm
Firm Design Capacity	250 gpm 3,300 lb/hr	320 gpm	200 gpm	2,269 gpm
Current Loadings (max month)	493 gpm (12/07)* 2,939 lb/hr (07/00)*	493 gpm*	115 gpm*	--
Current Utilization of Max Design Capacity	99% (Hydraulic) 45% (Solid)	103%	29%	--
Current Utilization of Firm Design Capacity	198% (Hydraulic) 90% (Solid)	206%	58%	--

** Assume GBT operated 24 hours a day under maximum loading conditions.*

Centrifuge

The existing centrifuge was installed during the plant's Tenth Addition and is in good condition. The centrifuge dewaters digested sludge and then transports it to the Biosolids End-Use Production Building on a belt conveyor. There is one unit installed with provisions for a second unit. The centrifuge polymer system is sized to provide polymer for two centrifuges.

Table 5-27 presents information on the existing centrifuge system.

Table 5-27. Centrifuge System Description

Parameters	Centrifuge	Shaftless Screw Conveyor	Belt Conveyor
Number of Units	1	1	1
Unit Design Capacity	150 gpm 2,000 lb/hr	4 wet ton/hr	8 wet ton/hr
Dewatered Solids Concentration	23-25%	--	--

Table 5-28 presents an evaluation of the existing centrifuge system capacity. The centrifuge system is being operated intermittently mainly due to the problematic anaerobic digestion operation.

Table 5-28. Centrifuge System Capacity Evaluation

Parameters	Centrifuge	Shaftless Screw Conveyor	Belt Conveyor
Number of Units	1	1	1
Unit Capacity	150 gpm 1,250 lb/hr	4 wet ton/hr	8 wet ton/hr
Dewatered Solids Concentration	21%	--	--
Weekly Operation	104 hours	104 hours	104 hours
Weekly Capacity	936,000 gallons 130,000 lbs	416 wet tons	832 wet tons

Metrogro Storage Tanks

The plant has two types of biosolids end-products: Metrogro liquid biosolids and a soil-like end product generated by Biosolids End-Use Production Facility. Metrogro biosolids are stored in the three existing Metrogro storage tanks with a total volume of 19.4 million gallons. Each tank is covered with an aluminum dome to collect odorous air and is equipped with six 15-horsepower submersible propeller mixers to provide a uniform feed for the Metrogro land application program.

Table 5-29 presents information on the existing Metrogro storage tanks.

Table 5-29. Metrogro Storage Tanks Description

Parameters	Metrogro Storage Tanks	Metrogro Storage Tank Mixers
Number of Units	3	18 total, 6 per tank
Type	Precast Concrete	Submersible Propeller
Diameter	160 feet	--
Depth	39 feet SWD	--
Volume per Tank (includes cone bottom)	6,467,747 gallons	--
Total Volume	19,403,232 gallons	--
Size	--	15 hp

Table 5-30 presents an analysis on the capacity of the existing Metrogro storage tanks. The three existing tanks provide sufficient storage capacity under the current condition.

Table 5-30. Metrogro Storage Tanks Capacity Evaluation

Parameters	Values
Design Criteria (WPDES permit requirements)	Provide 180 days storage
Storage Capacity	19,500,000 gallons
Current Maximum Biosolids Production Over 180 Days	18,800,000 gallons
Current Utilization of Capacity	94%

Biosolids End-Use Production Facility

The Tenth Addition Facility Plan called for approximately 10-25% of the dewatered biosolids to be mixed with amendment materials to produce a “soil-like” end product. The end-product is designed to compliment but not compete with the Metrogro Liquid Land Application Program. Space is provided in the facility to store dewatered cake, amendment materials, and final product. The facility consists of a partially covered asphalt pad. The covered area serves to protect stored materials from the rain. The uncovered area is used as a working area or for air drying biosolids.

Table 5-31 presents information on the existing Biosolids End-Use Production Facilities.

Table 5-31. Biosolids End-Use Production Facilities Description

Parameters	Values
Number of Units	1
Type	Pre-fabricated metal building
Total Covered Area	27,000 sf
Total Uncovered Area	9,000 sf

Table 5-32 provides an evaluation on the existing facilities.

Table 5-32. Biosolids End-Use Production Facilities Capacity Evaluation

Parameters	Values
Total Covered Area Storage Capacity	n/a
Total Uncovered Area Storage Capacity	n/a
Current Required Storage Capacity	n/a
Current Utilization of Capacity	n/a

Plant Water System

The existing plant water system was installed during the plant's Tenth Addition to provide non-potable water use for the treatment processes. The plant water system is equipped with booster pumps, automatic strainers, and a disinfection system. The plant water system serves gravity belt thickeners, centrifuge, digester/storage tank cleaning, liquid ring gas compressors for digester confined gas mixing system, headworks facility, odor beds, and general washdown. Based on the operational data between November 2005 and March 2008, the peak day plant water usage accounts for approximately 63% of the plant water system capacity. The existing system has sufficient capacities to accommodate the current plant water usage. Table 5-33 provides the information on the existing plant water system.

Table 5-33. Plant Water System Description

Parameters	Booster Pump	Strainer
Number of Units	3	2
Type	Centrifugal	Automatic, self cleaning
Unit Capacity	600 gpm	1,200 gpm
Motor	20 hp, variable speed	--

Table 5-34 provides an analysis of the existing plant water system. Based upon the operational data between November 2005 and March 2008, the existing system has sufficient capacity to accommodate maximum day plant water usage.

Table 5-34. Plant Water System Capacity Evaluation

Parameters	Booster Pump	Strainer
Unit Capacity	600 gpm	1,200 gpm
Firm Capacity	1,200 gpm	1,200 gpm
Current Max Required Capacity*	759 gpm	759 gpm
Current Capacity Utilization**	63%	63%

**Current capacity is determined based on operational data between November 2005 and March 2008.*
***Assume plant water pumps work continuously.*

6. Plant Hydraulics

Background Information

The plant consists of four main sections – headworks, West Plant treatment train, East Plant treatment train, and ultraviolet (UV) disinfection and effluent pumping. The headworks contain fine band screens, grit basins, and a weir flow splitter to control the flow distribution between the East Plant and West Plant. Each treatment train contains rectangular primary clarifiers and aeration basins followed by circular secondary clarifiers. The UV disinfection receives flow from both the East and West Plant treatment trains and discharges final treated effluent by pumps to receiving water bodies. Gravity flow and a series of weirs govern the hydraulics of NSWWTP with the final effluent pumps and return activated sludge pumps being the only pumps affecting the overall plant hydraulics of the liquid stream.

Evaluation Method

A plant hydraulics spreadsheet was developed to determine the overall hydraulic capacities and to identify potential hydraulic bottlenecks at the NSWWTP. The spreadsheet is programmed to allow the user to examine each section of the plant separately. Figure 6-1 shows the schematic diagram of the NSWWTP hydraulics as modeled in this analysis.

Both the East and West Plant clarifiers and aeration basins operate in parallel groups with the exact flow split through each individual clarifier or basin generally unknown. It is assumed, based on the equal size of the individual components, that the flow through the West Plant is split equally to its aeration basins and final clarifiers, such that each of the eight West Plant final clarifiers treats one eighth of the entire flow through the West Plant.

The 11 East Plant final clarifiers, 18 aeration basins and 14 primary clarifiers are operated in three groups. The assumption of equal distribution cannot be made for the flow splits through the East Plant due to the different sizes and flow paths of individual components. The unknown, variable flow splits are solved by varying the split of flow through each group of clarifiers or aeration basins until each possible flow path creates same amount of headloss.

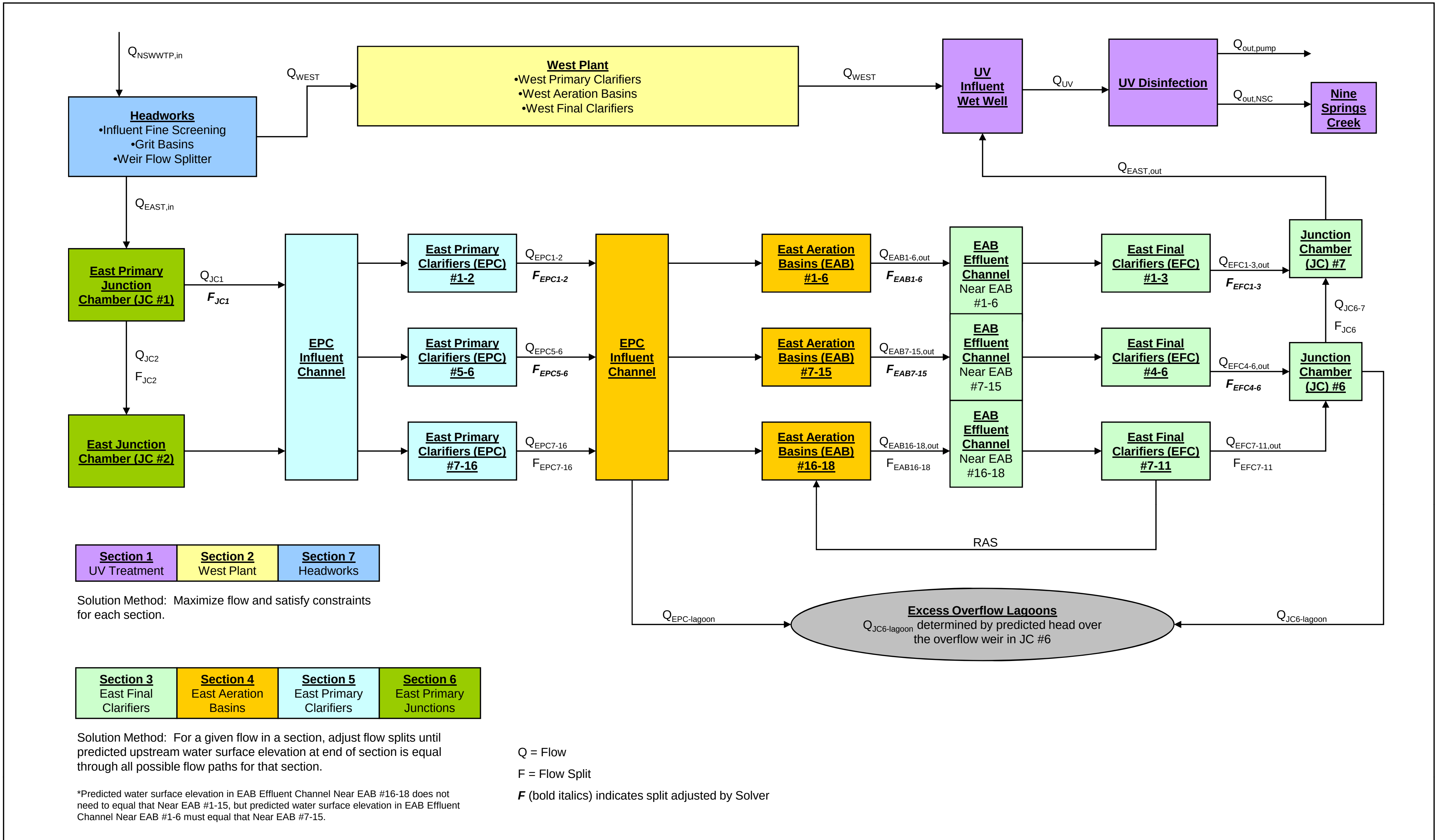
The hydraulic spreadsheet utilized the Microsoft Excel add-in Solver to determine the maximum flow rate through the UV disinfection and West Plant directly and to solve for the unknown flow splits of the East Plant. The maximum flows are determined based upon user definable constraints, such as minimum freeboard or maximum overflow rates at clarifiers.

Hydraulic Analysis

The hydraulic spreadsheet was utilized to estimate maximum flows under different conditions. The following four scenarios were analyzed to determine the maximum hydraulic capacities of the plant:

- Scenario No. 1 – Status quo without constrains

  No modification was made to the existing facilities. The structure minimum freeboard was set to be zero. No limit was set to the primary and secondary clarifier overflow rates.



MALCOLM PIRNIE

Flow Schematic for Nine Springs Wastewater Treatment Plant Hydraulic Modeling

Figure 6-1

- Scenario No. 2 – Status quo with constrains on clarifier overflow rates

  No modification was made to the existing facilities. The primary and secondary clarifier overflow rate upper limits were set to the recommended values presented in Chapter 4.

- Scenario No. 3 – Status quo with constrains on the freeboard heights.

  No modification was made to the existing facilities. The structure minimum free board was set to be 1.0 ft. No limit was set to the primary and secondary clarifier overflow rates.

- Scenario No. 4 – Diversion from splitter structure to lagoons

  In this scenario, a 72" excess diversion flow pipe was added from the flow splitter structure to excess flow storage lagoons. Excess flow during peak flows will be diverted to lagoons. The structure minimum freeboard was set to be zero. No limit was set to the primary and secondary clarifier overflow rates.

Table 6-1 presents the hydraulic analysis result for scenario #1. With no constraints other than preventing wastewater from overflowing the treatment structures, the plant can accommodate a peak flow of 169 MGD with approximately 88 MGD being discharged to either effluent storage lagoons or Nine Springs Creek.

Table 6-1. Hydraulic Analysis Summary - Scenario #1

Parameters	Values
Maximum Total Plant Flow (MGD)	169
East Plant Flow (MGD)	91
West Plant Flow (MGD)	78
Effluent Building Flow (MGD)	125
Disinfected Effluent Overflow (MGD)	44
Secondary Treatment Effluent Overflow (MGD)	44
Effluent Return Pump Pumping Rate (MGD)	82
East Primary Clarifier Overflow Flow (gal/sf/day)	4,600
West Primary Clarifier Overflow Flow (gal/sf/day)	3,890
East Secondary Clarifier Overflow Flow (gal/sf/day)	1,605
West Secondary Clarifier Overflow Flow (gal/sf/day)	915

Table 6-2 presents the hydraulic analysis results for scenario #2. In this scenario, the maximum overflow rates for the East and West primary clarifiers were set to be 1,500 and 2,000 gallons per square foot per day respectively. The maximum hydraulic capacity for this scenario is 76 MGD with no excess flow being discharged. The East primary clarifier No. 1 and No. 2 create bottlenecks in this scenario. The model analysis shows that the flow splitting among the 14 clarifiers in East Plant are not proportional to their surface areas, hence some clarifiers reach the preset maximum overflow rate before the others. When clarifier No. 1 and No. 2 reach 1,500 gal/sf/d, clarifier No. 7 to No. 16 are running under 800 gal/sf/d. Modifications to the existing flow splitting structures and conduits will improve the flow splitting among all clarifiers and increase the overall hydraulic capacity of the East primary clarifiers.

Table 6-2. Hydraulic Analysis Summary - Scenario #2

Parameters	Values
Maximum Total Plant Flow (MGD)	76
East Plant Flow (MGD)	36
West Plant Flow (MGD)	40
Effluent Building Flow (MGD)	76
Disinfected Effluent Overflow (MGD)	0
Secondary Treatment Effluent Overflow (MGD)	0
Effluent Return Pump Pumping Rate (MGD)	76
East Primary Clarifier Overflow Flow (gal/sf/day)	1,500
West Primary Clarifier Overflow Flow (gal/sf/day)	2,000
East Secondary Clarifier Overflow Flow (gal/sf/day)	753
West Secondary Clarifier Overflow Flow (gal/sf/day)	471

Table 6-3 presents the hydraulic analysis result for scenario #3. In this scenario, the minimum freeboard height was set to 1.0 foot throughout the entire plant. The maximum flow capacity for this scenario is 140 MGD with approximate 58 MGD of excess flow being discharged to either effluent storage lagoons or Nine Springs Creek. Both East and West Plant clarifiers are heavily overloaded in terms of hydraulic loadings under this scenario.

Table 6-3. Hydraulic Analysis Summary - Scenario #3

Parameters	Values
Maximum Total Plant Flow (MGD)	140
East Plant Flow (MGD)	71
West Plant Flow (MGD)	68
Effluent Building Flow (MGD)	116
Disinfected Effluent Overflow (MGD)	34
Secondary Treatment Effluent Overflow (MGD)	24
Effluent Return Pump Pumping Rate (MGD)	82
East Primary Clarifier Overflow Flow (gal/sf/day)	3,439
West Primary Clarifier Overflow Flow (gal/sf/day)	3,425
East Secondary Clarifier Overflow Flow (gal/sf/day)	1,423
West Secondary Clarifier Overflow Flow (gal/sf/day)	806

Table 6-4 presents the hydraulic analysis result for scenario #4. This scenario assumes 80 MGD of excess flow being diverted to lagoons from Flow Splitter. It raises the plant overall hydraulic capacity to 249 MGD. The total secondary treated excess flow will be 88 MGD. The lagoons will be filled in approximately 23 hours at this flow rate assuming all treated excess flow is discharged to Nine Springs Creek.

Table 6-4. Hydraulic Analysis Summary - Scenario #4

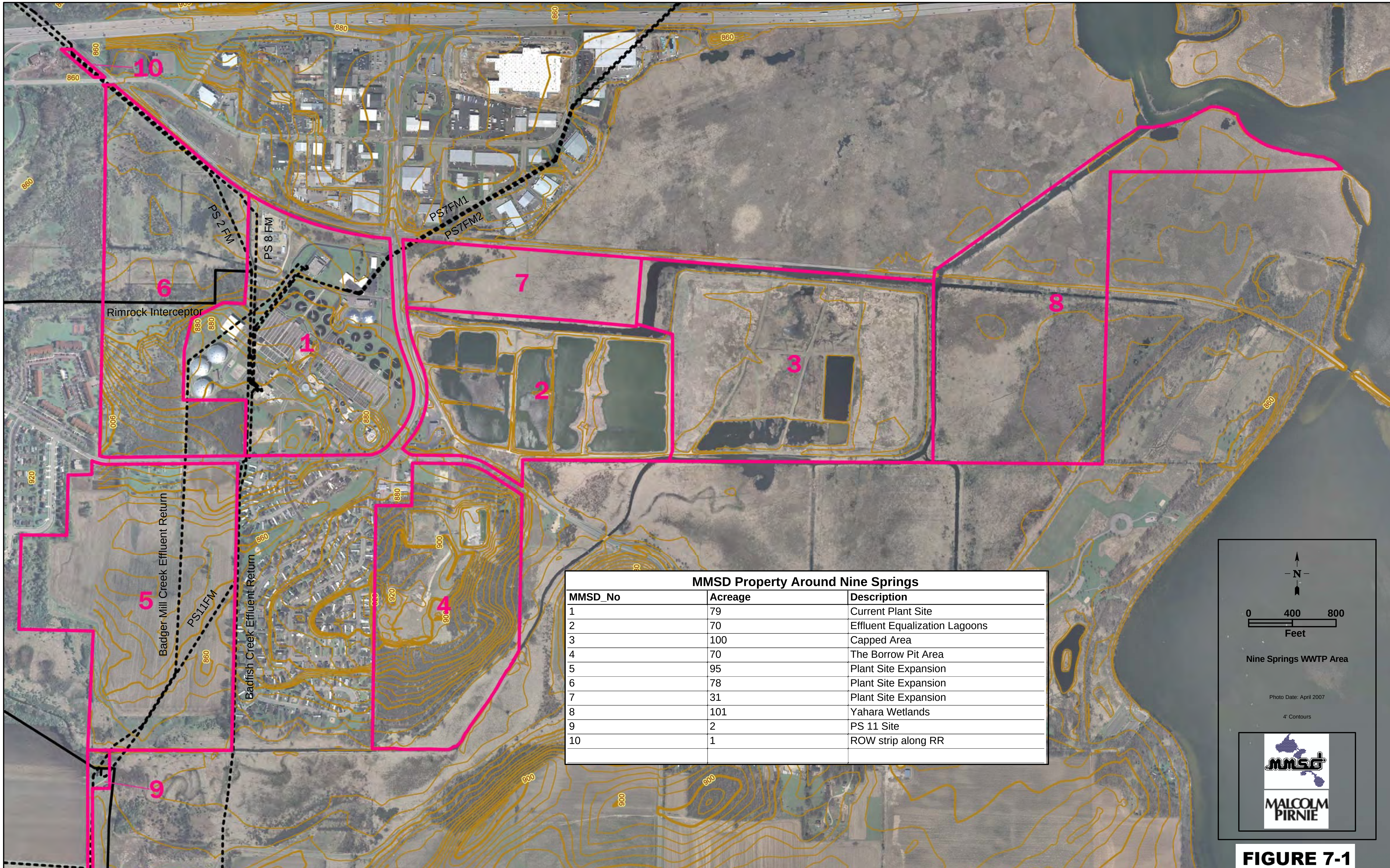
Parameters	Values
Maximum Total Plant Flow (MGD)	249
East Plant Flow (MGD)	91
West Plant Flow (MGD)	78
Excess Flow Diverted from Flow Splitter to Lagoons	80
Effluent Building Flow (MGD)	125
Disinfected Effluent Overflow (MGD)	44
Secondary Treatment Effluent Overflow (MGD)	44
Effluent Return Pump Pumping Rate (MGD)	82
East Primary Clarifier Overflow Flow (gal/sf/day)	4,600
West Primary Clarifier Overflow Flow (gal/sf/day)	3,890
East Secondary Clarifier Overflow Flow (gal/sf/day)	1,605
West Secondary Clarifier Overflow Flow (gal/sf/day)	915

7. Site Considerations

Land availability at the vicinity of the NSWWTP is a major concern in the master planning. One viable planning alternative is to expand the existing plant and continue the current centralized operation. Lack of sufficient reserved land will create restraints for applying this alternative and decrease the District's flexibility in implementing wastewater treatment and biosolids disposal options. The existing facilities at the NSWWTP will have to be expanded under the following two conditions or the combination of them:

- More stringent effluent discharge or reuse limits and biosolids disposal requirements are applied by the regulatory agencies. This will make it necessary to implement more advanced wastewater treatment and biosolids disposal processes to meet the new limits.
- Centralized wastewater treatment and biosolids disposal operation at the NSWWTP is continued. Therefore additional treatment facilities will need to be constructed to accommodate increased flows and loadings to the plant during the planning period.

This chapter documents the District-owned properties in the vicinity of the NSWWTP. Figure 7-1 presents the detailed information of these properties. The parcel Nos. 5, 6 and 7 have the potential to be used for future plant improvement and expansion. The roles of these properties in the plant's future expansion and improvement will be further studied when the future alternatives are better defined in the later phases of master planning.



MMSD Property Around Nine Springs

MMSD_No	Acreage	Description
1	79	Current Plant Site
2	70	Effluent Equalization Lagoons
3	100	Capped Area
4	70	The Borrow Pit Area
5	95	Plant Site Expansion
6	78	Plant Site Expansion
7	31	Plant Site Expansion
8	101	Yahara Wetlands
9	2	PS 11 Site
10	1	ROW strip along RR

FIGURE 7-1

8. Electrical Distribution Systems

The plant's electrical distribution system has been evaluated from both capacity and from remaining life perspectives.

Capacity

Madison Gas & Electric (MG&E) furnishes the electrical power requirements for the plant at 13.8kV. On site generators provide a portion of the plant power at 480 volts. The electrical power that MG&E supplies is provided to the plant through two 13.8 kV electrical feeders, and then transformed to 4.16 kV by two MMSD-owned 10/14 MVA transformers. The transformers feed the entire plant through two 2,000 Amp buses in Switchgear S1. To provide redundancy, all areas of the plant may be fed from either switchgear bus. Several larger loads utilize the 4.16 kV voltage directly (effluent pumps and blowers) while unit substations with step-down transformers provide lower utilization voltage for smaller loads.

The plant's two engine generators are installed in Sludge Control Building No. 2. The generators are induction type and each has a maximum rated capacity of 475 kW, 480 volts. The power generated is subsequently stepped up by transformers in Sludge Control Building No. 2 and connected into the main switch gear. The electricity produced by these generators is used exclusively within the plant and not conveyed to the utility grid. Table 8-1 presents information on the electricity purchased from MG&E from 2001 to 2007. Due to the lack of operational data, a conservative power factor value of 0.7 was used for converting power demand from kW to kVA.

Table 8-1. Monthly Plant Electrical Consumption (2001-2007)

Parameters	Average	Maximum	Minimum
MG&E On-Peak Demand (kW/kVA)	3,268/4,669	4,250/6,071	2,638/3,769
MG& E Off-Peak Demand (kW/kVA)	3,350/4,786	4,113/5,876	2,776/3,966
Total Electricity by MG & E (kW-hr)	1,828,356	2,642,000	1,530,000

Table 8-2 provides an evaluation of the capacity of the existing 13.8 kV transformers. It shows that the existing transformers have adequate capacity for the current electricity usage and for considerable future expansion.

Table 8-2. Main Transformer Capacity Analysis

Parameters	Value
No. of Unit	2
Transformer Unit Capacity (kVA)	10,000
Transformer Firm Capacity (KVA)	10,000
MG&E On-Peak Demand @ Max Month (KVA)	6,071
MG& E Off-Peak Demand @ Max Month (KVA)	5,876
Firm Capacity Utilization for On-Peak Demand	61%
Firm Capacity Utilization for Off-Peak Demand	59%

Remaining Life

The remaining life of the electrical equipment is generally dependent on three criteria:

- Capacity to provide power to present and anticipated future loads
- Acceptable physical and electrical condition of the equipment such that neither catastrophic nor frequent maintenance problems are experienced or anticipated
- Availability of spare parts

From the capacity review above, the plant electrical system appears to have sufficient capacity for both present and considerable additional future loads. From a physical condition perspective, maintenance staff has not reported any corrosion, overheating, or frequent maintenance problems that would tend to suggest physical deterioration. From a spare parts perspective, it is noted that the electrical equipment installed in the 1980's may be nearing the end of its useful life. Generally, electrical equipment, other than transformers, is considered to have a useful life of 25-30 years, although this depends on the availability of spare parts and physical condition. Transformer life is dependent on the percentage of rated loading (the lower the load, the longer the life), and for liquid filled transformers, on the condition of the dielectric fluid. Liquid filled transformer life can extend to 100 years or more under low to medium load conditions where regular maintenance has been performed. For the 50-year long range plan, electrical distribution equipment should be evaluated for replacement based on the criteria above, and should be scheduled for replacement at the appropriate time.

9. Operation and Maintenance Facilities

The purpose of this chapter is to document the information regarding the staffing and existing operation/maintenance facilities at the NSWWTP, and set the benchmark relative to the facilities required to adequately accommodate current and future operational activities at the plant. The deficiencies on the existing facilities identified by plant staff were also listed.

General Staffing

The District currently has 83 employees, most personnel work 8 hours per day, Monday through Friday. The plant operators work two 12-hour shifts including weekends. During the spring and fall hauling seasons, the Metrogro Crew works 10 to 12 hours per day and may work during weekends. The total staff by group is presented in Table 9-1.

Table 9-1. District Staff Composition by Group

Group	No. of Staff	Men	Women
Admin/Administration/Accounting/Clerical	6	2	4
Training	1	1	0
Information Systems	4	2	2
Special Projects/Pretreatment	2	2	0
Laboratory	8	5	3
Engineering	10	10	0
Purchasing	2	2	0
Monitoring Services/Sewer Maintenance	5	5	0
Metrogro Biosolids Reuse Program	5	5	0
Mechanical Maintenance	8	8	0
Electrical Maintenance	8	8	0
Operations	13	12	1
Building and Grounds	11	11	0
Total	**83**	**73**	**10**

Personnel Facilities

Locker/shower and lunch facilities are provided at the Operations Building, Maintenance Shop No.1 and Vehicle Loading Building. The detailed facility information is presented in Tables 9-2 and 9-3.

Table 9-2. Locker/Shower Facilities Description

Facility	No. of Men's Lockers	No. of Women's Lockers	Men's Shower Facilities	Woman's Shower Facilities	Laundry Service	Boot Wash Area
Operations Building	84	10	One group shower	One group shower	Yes	One w/ 22 lockers
Maintenance Shop No. 1	20	n/a	Two individual showers	n/a	Yes	n/a
Vehicle Loading Building	15	4	Two individual showers	One individual showers	Yes	n/a

Table 9-3. Lunch Facilities Description

Facility	Refrigerator	Microwave	Sink	Dishwasher	Cabinet
Operations Building – Multipurpose Room	X	X	X		X
Operations Building – Second Floor Break Area	X	X	X	X	X
Operations Building – Laboratory Library	X	X	X		X
Maintenance Shop No. 1	X	X	X		X
Vehicle Loading Building	X	X	X		X

The plant maintenance staff has identified the following items to be addressed and improved for the personnel facilities:

- Laundry area
- Lunchroom
- Locker room facilities with showers
- Restrooms

Office and Support Facilities

Most offices are located in the Operations Building. Workgroup supervisors have offices in the Maintenance Shop No. 1 and the Vehicle Loading Building. Table 9-4 presents the support facility information.

Table 9-4. Support Facility Description

Facility	Function	Area (sf)
Commission Room	Meetings	420
Engineering Conference Room	Meetings	250
Administration Library	Meetings	180
Laboratory	Analyses	9,140
First Aid Room	First Aid	66
Process Control Room	Process monitoring/control	275
Multipurpose Room	Meetings, tours, lunch area	1,300
Print Room	Creating prints	350
File Storage	Storage of records and drawings	340
Server Room	Computer servers	860
Training area	Training , Training Library, Training Manager's office, Media Room	700

The plant maintenance staff has identified the following items to be addressed and improved for the office and support facilities:

- Office area for supervisors
- Office area for purchasers
- Purchasing section is close to the Maintenance Shop and induction heater, which makes a lot of noise.
- Library area – O&M manuals, plans, vendor manuals, etc.
- Computer work space/desks
- Parking area
- Wireless network

Maintenance Facilities

The District maintenance facilities include Maintenance Shop Nos. 1 and 2; Storage Building Nos. 1, 2 and 3; Service Building, and Vehicle Loading Building. Service provided at each facility and corresponding square footage are presented in Table 9-5.

Table 9-5. Maintenance Facilities Description

Facility	Function	Area (sf)
Maintenance Shop No. 1	Mechanical Maintenance/vehicle parking	680
	Meeting, locker area	195
	Electrical Maintenance/vehicle parking	680
	Electrical maintenance office area	400
	Purchasing office	440
	Inventory storage – first floor	790
	Inventory storage – basement	570
	Monitoring Services sampler repair	420
	Excess equipment/parts storage	4,100
	Maintenance supervisors' office area	590
Maintenance Shop No. 2	Machining and welding areas	2,150
	Vehicle maintenance	1,700
Storage Building No. 1	Vehicle parking, inventory, storage	7,280
Storage Building No. 2	Vehicle parking, B&G shop area, pump and hose storage, loading dock	5,000
Storage Building No. 3	Lubricant Storage	1,320
Service Building	Woodworking shop	700
	Sewer Maintenance inventory	500
Vehicle Loading Building	Fleet Maintenance	1,670
	Lubricant Storage	250
	Inventory Area	610

The plant maintenance staff has identified the following items to be addressed and improved for the maintenance facilities:

- Work space for mechanics and electricians
- Work space for Monitoring Services/Sewer Maintenance
- Work space for Building and Grounds crew
- Welding and machining areas
- Vehicle maintenance area
- Loading dock and drive up delivery area
- Sandblasting and pump washing areas
- Painting room
- Drive-through vehicle parking areas

Storage Facilities

Storage facilities are provided in multiple locations in the plant. Existing Storage facilities are presented in Table 9-6.

Table 9-6. Storage Facilities Description

Equipment/ Shelving	Shop No.1	Shop No.2	Storage Building No.1	Storage Building No. 2	Storage Building No. 3	Operations Building	Vehicle Loading Building
Workbench	x	x		x	x		
Storage cabinet	x	x		x			
Tool Storage cabinet	x	x		x			
Flammable storage cabinet				x			
Open shelving, light duty	x			x			
Open shelving, heavy duty	x		x	x	x		
Closed shelving, light duty	x			x			
Closed shelving, heavy duty	x	x					
Paint cabinet		x					
Acid cabinet						x	
Pipe stock shelving, horizontal			x				
Small bin storage	x						
Large bin storage	x		x				
Small circular bin	x						
Large circular bin	x						
Metal stock storage, horizontal		x					
Metal stock storage, vert.		x					
Hose storage rack				x			
Barrel racks					x		
Lubricant Storage							x
Inventory Area							x

The plant maintenance staff has identified the following items to be addressed and improved for the storage facilities:

- Inventory area and un-inventoried parts storage area
- Tool and equipment storage for mechanics
- Tool and equipment storage for electricians
- Tool and equipment storage for Monitoring Services/Sewer Maintenance
- Tool and equipment storage for Building and Grounds crew
- Vehicle storage
- Portable pump storage
- Large parts storage area for spares, mixers, maci pumps, heads, old breakers, etc.

Appendix A
Peaking Factors (1996-2007)

Table A. 1996 Raw Influent Data

	Flow (MGD)	TSS (lbs/day)	BOD (lbs/day)	TKN (lbs/day)	NH_3-N (lbs/day)	T-P (lbs/day)
Average Annual	38.18	68,116	69,918	10,020	6,345	2,150
30 Day Maximum	41.39	80,448	83,816	11,181	6,979	2,514
7 Day Maximum	43.02	85,779	89,378	11,795	7,281	2,648
1 Day Maximum	46.18	89,487	92,714	11,963	7,410	2,683

Table B. 1997 Raw Influent Data

	Flow (MGD)	TSS (lbs/day)	BOD (lbs/day)	TKN (lbs/day)	NH_3-N (lbs/day)	T-P (lbs/day)
Average Annual	36.92	65,162	69,954	9,967	6,248	2,036
30 Day Maximum	39.42	75,985	83,183	10,992	6,963	2,355
7 Day Maximum	42.24	81,095	87,248	11,531	7,242	2,522
1 Day Maximum	53.42	83,787	90,255	11,677	7,474	2,581

Table C. 1998 Raw Influent Data

	Flow (MGD)	TSS (lbs/day)	BOD (lbs/day)	TKN (lbs/day)	NH_3-N (lbs/day)	T-P (lbs/day)
Average Annual	41.12	69,414	71,424	10,569	6,509	2,180
30 Day Maximum	45.98	79,922	83,282	11,698	7,358	2,481
7 Day Maximum	51.31	86,420	89,002	12,120	7,735	2,578
1 Day Maximum	73.23	91,850	93,229	12,538	8,009	2,700

Table D. 1999 Raw Influent Data

	Flow (MGD)	TSS (lbs/day)	BOD (lbs/day)	TKN (lbs/day)	NH_3-N (lbs/day)	T-P (lbs/day)
Average Annual	41.59	70,843	71,481	10,741	6,551	2,109
30 Day Maximum	46.20	83,006	82,446	11,947	7,235	2,417
7 Day Maximum	49.22	90,119	87,004	12,448	7,495	2,527
1 Day Maximum	78.52	93,311	91,119	12,744	7,679	2,600

Table E. 2000 Raw Influent Data

	Flow (MGD)	TSS (lbs/day)	BOD (lbs/day)	TKN (lbs/day)	NH_3-N (lbs/day)	T-P (lbs/day)
Average Annual	42.10	78,127	75,424	11,045	6,617	2,102
30 Day Maximum	48.93	95,921	88,194	12,291	7,337	2,409
7 Day Maximum	60.10	106,054	91,811	12,885	7,563	2,516
1 Day Maximum	89.19	114,282	95,012	13,107	7,820	2,621

Table F. 2001 Raw Influent Data

	Flow (MGD)	TSS (lbs/day)	BOD (lbs/day)	TKN (lbs/day)	NH_3-N (lbs/day)	T-P (lbs/day)
Average Annual	41.76	76,269	74,933	11,162	6,793	2,045
30 Day Maximum	45.30	94,321	88,331	12,612	7,566	2,343
7 Day Maximum	50.18	104,950	92,529	13,281	7,891	2,438
1 Day Maximum	81.83	121,514	94,659	13,595	8,077	2,508

Table G. 2002 Raw Influent Data

	Flow (MGD)	TSS (lbs/day)	BOD (lbs/day)	TKN (lbs/day)	NH_3-N (lbs/day)	T-P (lbs/day)
Average Annual	40.14	81,509	75,107	11,204	7,084	2,039
30 Day Maximum	43.05	101,612	88,399	13,141	7,991	2,393
7 Day Maximum	44.92	110,346	93,053	13,740	8,514	2,504
1 Day Maximum	52.29	112,985	95,753	14,065	8,761	2,557

Table H. 2003 Raw Influent Data

	Flow (MGD)	TSS (lbs/day)	BOD (lbs/day)	TKN (lbs/day)	NH_3-N (lbs/day)	T-P (lbs/day)
Average Annual	38.56	83,769	78,115	11,342	7,267	2,087
30 Day Maximum	40.98	96,269	90,609	13,154	8,201	2,402
7 Day Maximum	45.84	101,424	95,735	13,814	8,515	2,530
1 Day Maximum	63.65	105,242	98,482	14,023	8,756	2,598

Table I. 2004 Raw Influent Data

	Flow (MGD)	TSS (lbs/day)	BOD (lbs/day)	TKN (lbs/day)	NH_3-N (lbs/day)	T-P (lbs/day)
Average Annual	41.93	86,915	80,860	11,915	7,429	2,186
30 Day Maximum	47.60	105,334	95,947	13,356	8,308	2,521
7 Day Maximum	52.27	117,457	100,411	13,893	8,613	2,652
1 Day Maximum	95.13	123,729	103,691	14,206	8,776	2,725

Table J. 2005 Raw Influent Data

	Flow (MGD)	TSS (lbs/day)	BOD (lbs/day)	TKN (lbs/day)	NH_3-N (lbs/day)	T-P (lbs/day)
Average Annual	39.37	80,197	81,648	12,439	7,355	2,132
30 Day Maximum	42.65	92,266	94,357	13,941	8,433	2,414
7 Day Maximum	45.51	97,591	99,709	14,430	8,899	2,571
1 Day Maximum	50.78	100,503	104,333	15,278	9,119	2,631

Table K. 2006 Raw Influent Data

	Flow (MGD)	TSS (lbs/day)	BOD (lbs/day)	TKN (lbs/day)	NH_3-N (lbs/day)	T-P (lbs/day)
Average Annual	41.22	78,214	83,722	13,185	8,102	2,165
30 Day Maximum	45.01	93,182	100,086	14,761	9,141	2,494
7 Day Maximum	48.00	98,027	106,615	15,220	9,530	2,608
1 Day Maximum	53.36	101,520	109,152	15,604	9,655	2,656

Table L. 2007 Raw Influent Data

	Flow (MGD)	TSS (lbs/day)	BOD (lbs/day)	TKN (lbs/day)	NH_3-N (lbs/day)	T-P (lbs/day)
Average Annual	42.88	75,624	84,422	12,959	8,092	2,126
30 Day Maximum	48.75	89,424	97,357	14,346	9,092	2,332
7 Day Maximum	59.18	96,224	103,897	14,861	9,511	2,434
1 Day Maximum	75.31	101,233	108,792	15,127	9,734	2,490

Table M. 1996 Peaking Factors

	Flow	TSS	BOD	TKN	NH_3-N	T-P
30 Day Maximum	1.08	1.18	1.20	1.12	1.10	1.17
7 Day Maximum	1.13	1.26	1.28	1.18	1.15	1.23
1 Day Maximum	1.21	1.31	1.33	1.19	1.17	1.25

Table N. 1997 Peaking Factors

	Flow	TSS	BOD	TKN	NH_3-N	T-P
30 Day Maximum	1.07	1.17	1.19	1.10	1.11	1.16
7 Day Maximum	1.14	1.24	1.25	1.16	1.16	1.24
1 Day Maximum	1.45	1.29	1.29	1.17	1.20	1.27

Table O. 1998 Peaking Factors

	Flow	TSS	BOD	TKN	NH_3-N	T-P
30 Day Maximum	1.12	1.15	1.17	1.11	1.13	1.14
7 Day Maximum	1.25	1.24	1.25	1.15	1.19	1.18
1 Day Maximum	1.78	1.32	1.31	1.19	1.23	1.24

Table P. 1999 Peaking Factors

	Flow	TSS	BOD	TKN	NH_3-N	T-P
30 Day Maximum	1.11	1.17	1.15	1.11	1.10	1.15
7 Day Maximum	1.18	1.27	1.22	1.16	1.14	1.20
1 Day Maximum	1.89	1.32	1.27	1.19	1.17	1.23

Table Q. 2000 Peaking Factors

	Flow	TSS	BOD	TKN	NH_3-N	T-P
30 Day Maximum	1.16	1.23	1.17	1.11	1.11	1.15
7 Day Maximum	1.43	1.36	1.22	1.17	1.14	1.20
1 Day Maximum	2.12	1.46	1.26	1.19	1.18	1.25

Table R. 2001 Peaking Factors

	Flow	TSS	BOD	TKN	NH_3-N	T-P
30 Day Maximum	1.08	1.24	1.18	1.13	1.11	1.15
7 Day Maximum	1.20	1.38	1.23	1.19	1.16	1.19
1 Day Maximum	1.96	1.59	1.26	1.22	1.19	1.23

Table S. 2002 Peaking Factors

	Flow	TSS	BOD	TKN	NH_3-N	T-P
30 Day Maximum	1.07	1.25	1.18	1.17	1.13	1.17
7 Day Maximum	1.12	1.35	1.24	1.23	1.20	1.23
1 Day Maximum	1.30	1.39	1.27	1.26	1.24	1.25

Table T. 2003 Peaking Factors

	Flow	TSS	BOD	TKN	NH_3-N	T-P
30 Day Maximum	1.06	1.15	1.16	1.16	1.13	1.15
7 Day Maximum	1.19	1.21	1.23	1.22	1.17	1.21
1 Day Maximum	1.65	1.26	1.26	1.24	1.20	1.25

Table U. 2004 Peaking Factors

	Flow	TSS	BOD	TKN	NH_3-N	T-P
30 Day Maximum	1.14	1.21	1.19	1.12	1.12	1.15
7 Day Maximum	1.25	1.35	1.24	1.17	1.16	1.21
1 Day Maximum	2.27	1.42	1.28	1.19	1.18	1.25

Table V. 2005 Peaking Factors

	Flow	TSS	BOD	TKN	NH_3-N	T-P
30 Day Maximum	1.08	1.15	1.16	1.12	1.15	1.13
7 Day Maximum	1.16	1.22	1.22	1.16	1.21	1.21
1 Day Maximum	1.29	1.25	1.28	1.23	1.24	1.23

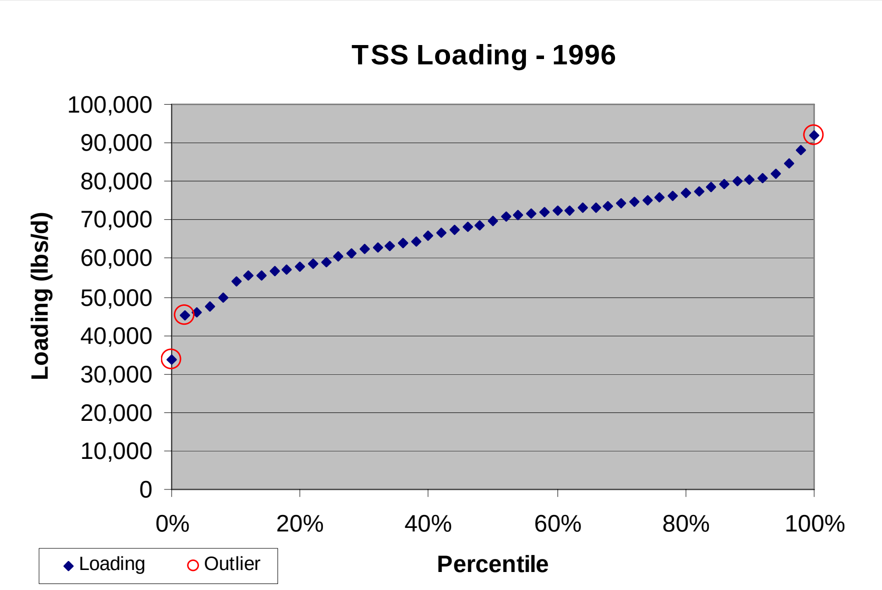
Table W. 2006 Peaking Factors

	Flow	TSS	BOD	TKN	NH_3-N	T-P
30 Day Maximum	1.09	1.19	1.20	1.12	1.13	1.15
7 Day Maximum	1.16	1.25	1.27	1.15	1.18	1.20
1 Day Maximum	1.29	1.30	1.30	1.18	1.19	1.23

Table X. 2007 Peaking Factors

	Flow	TSS	BOD	TKN	NH_3-N	T-P
30 Day Maximum	1.14	1.18	1.15	1.11	1.12	1.10
7 Day Maximum	1.38	1.27	1.23	1.15	1.18	1.14
1 Day Maximum	1.76	1.34	1.29	1.17	1.20	1.17

Appendix B
1996-2007 Percentile Plots
for Outlier Determination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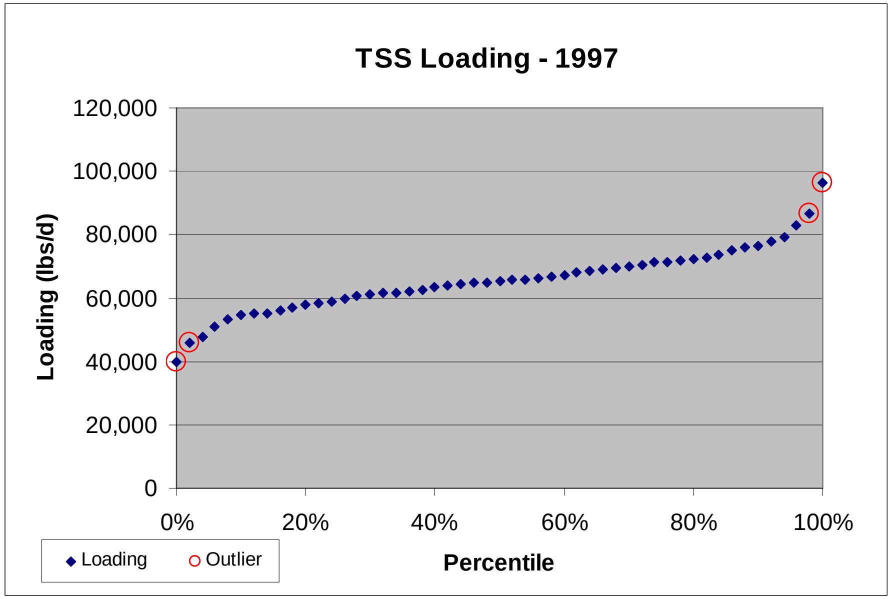
Loading
Outlier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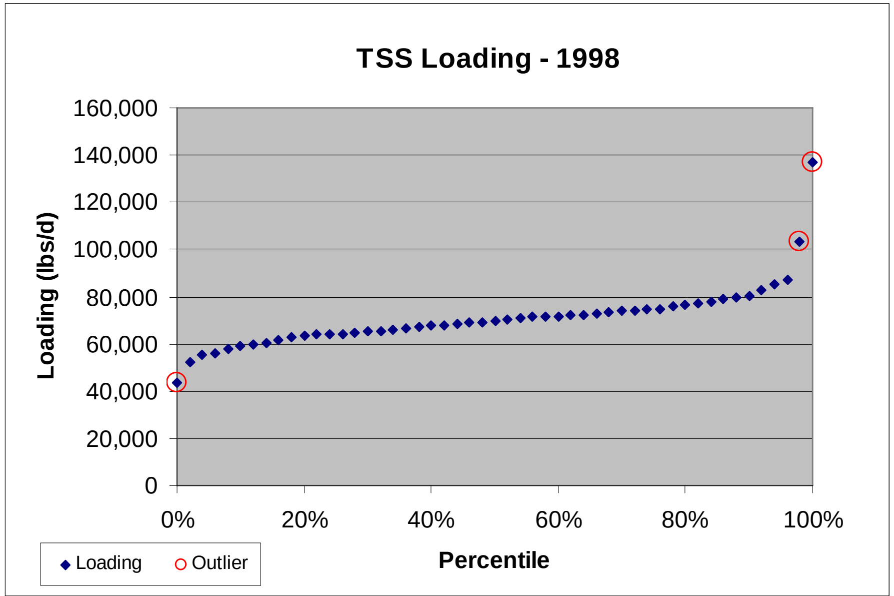
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Percentile
Loading
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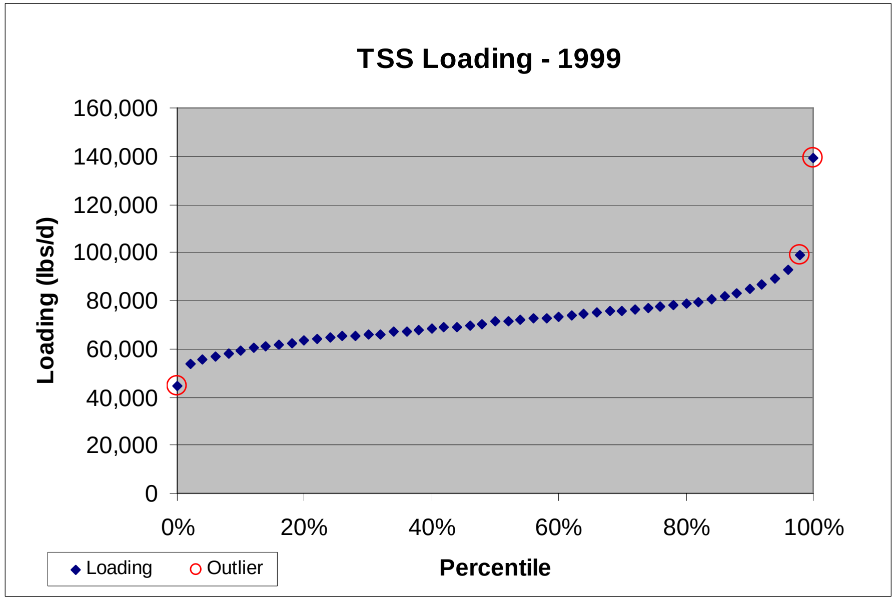
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Percentile
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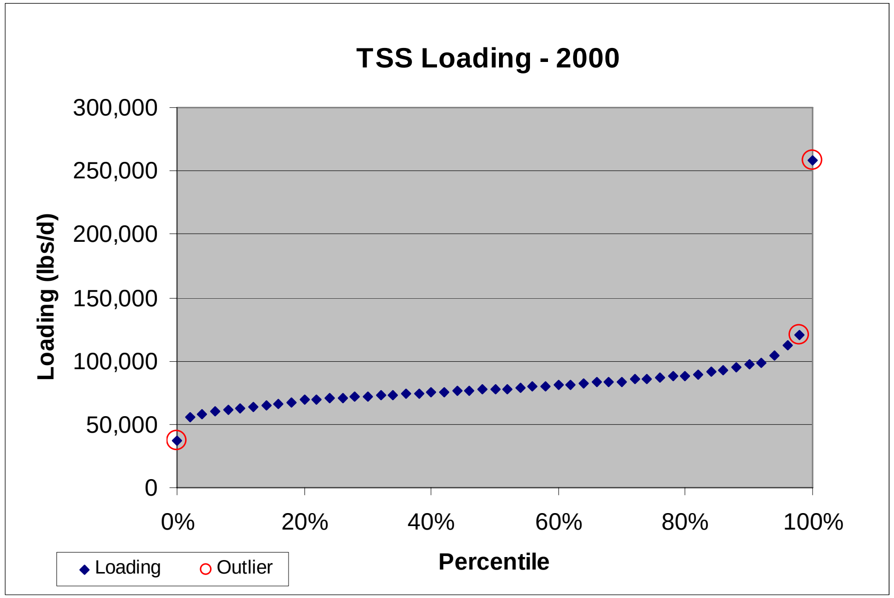
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Percentile
Loading
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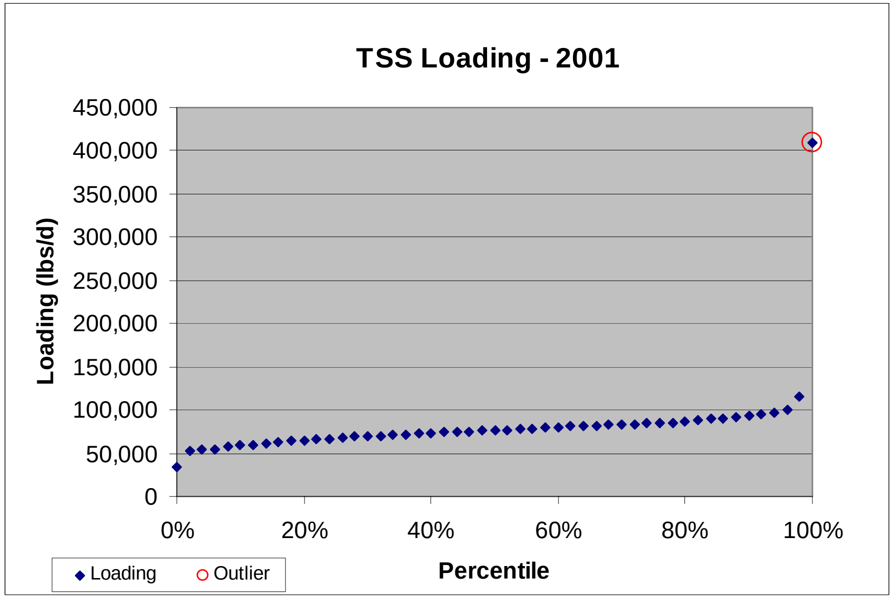
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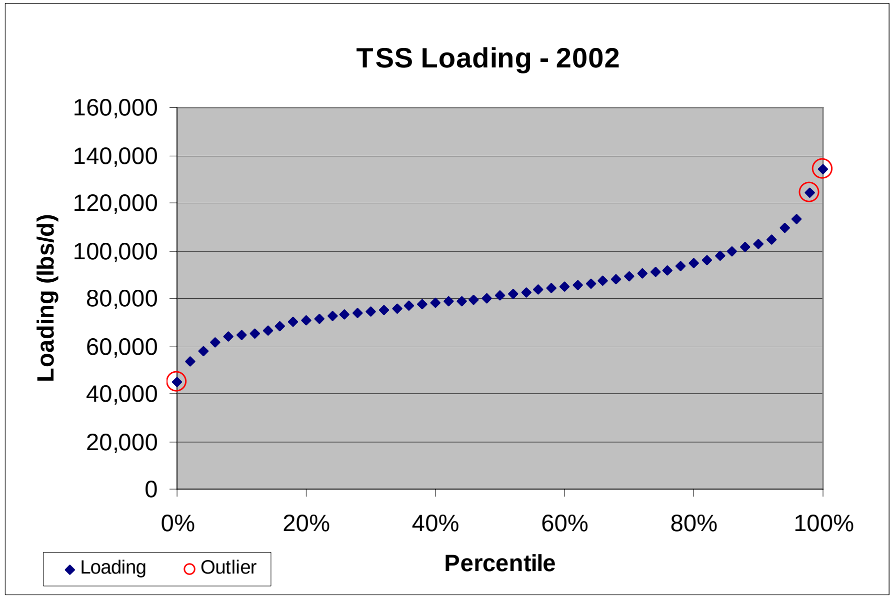
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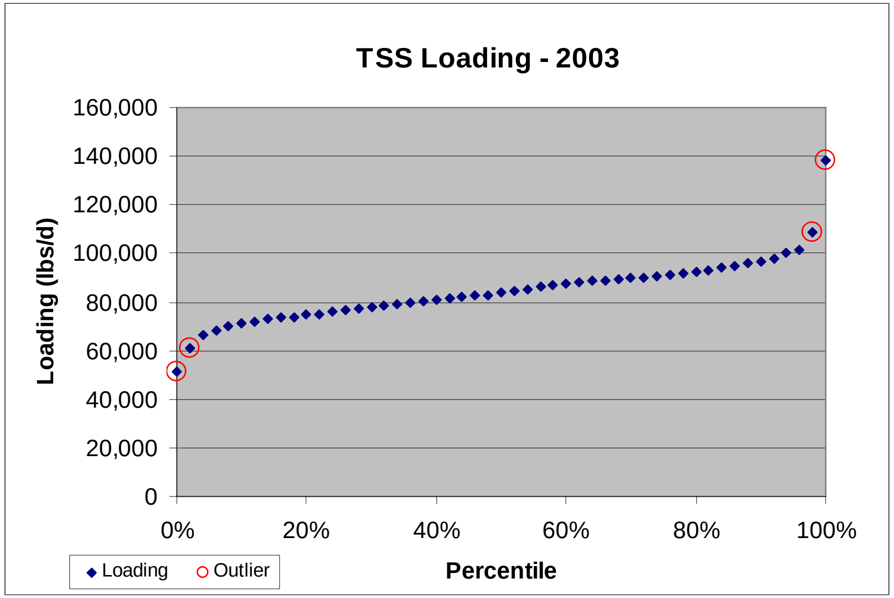
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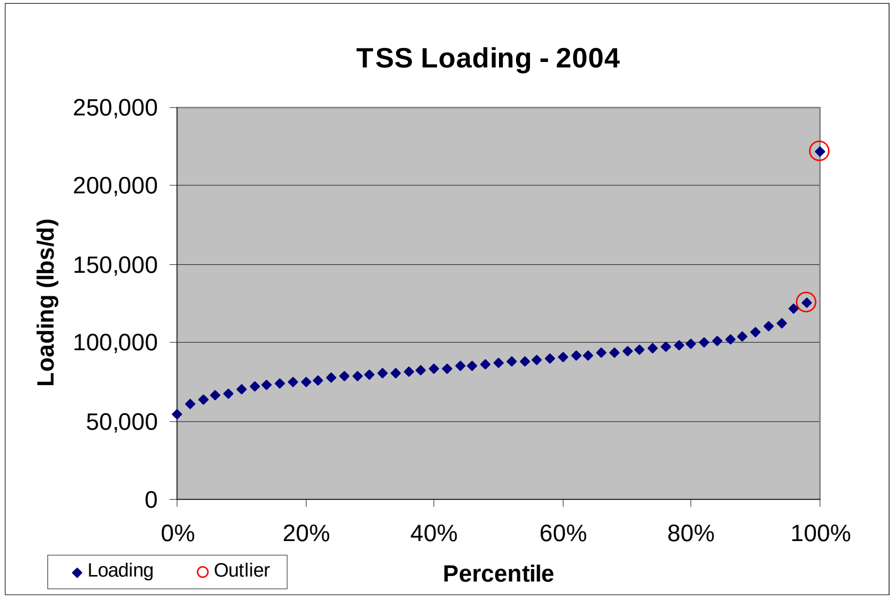
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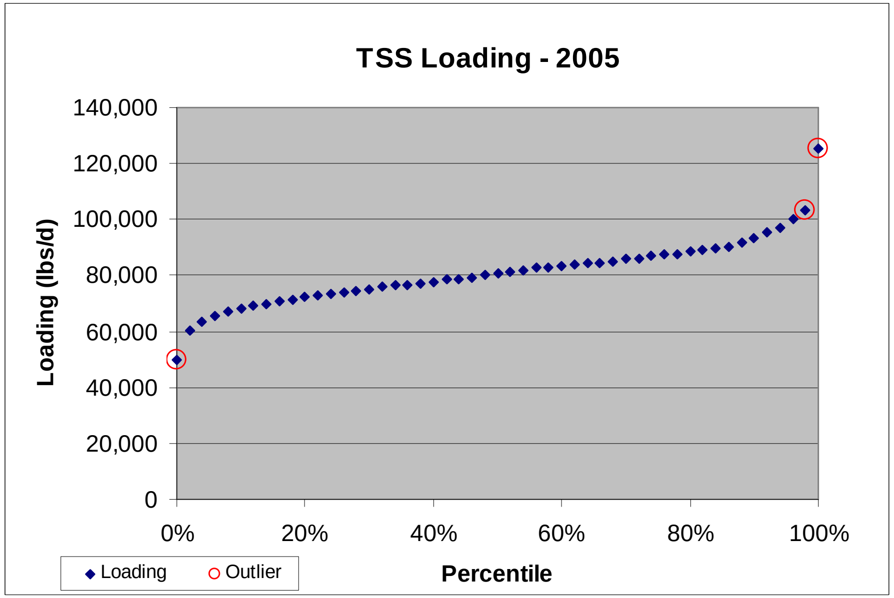
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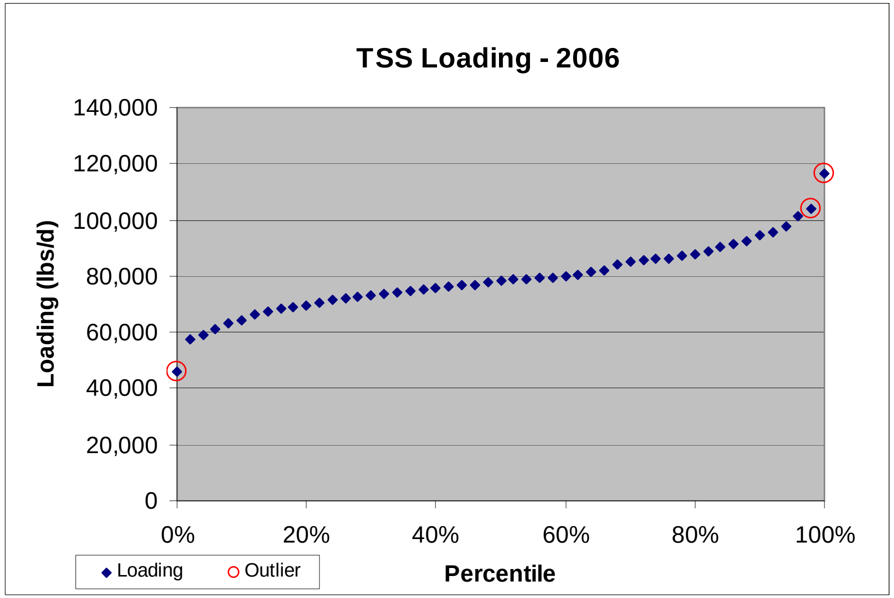
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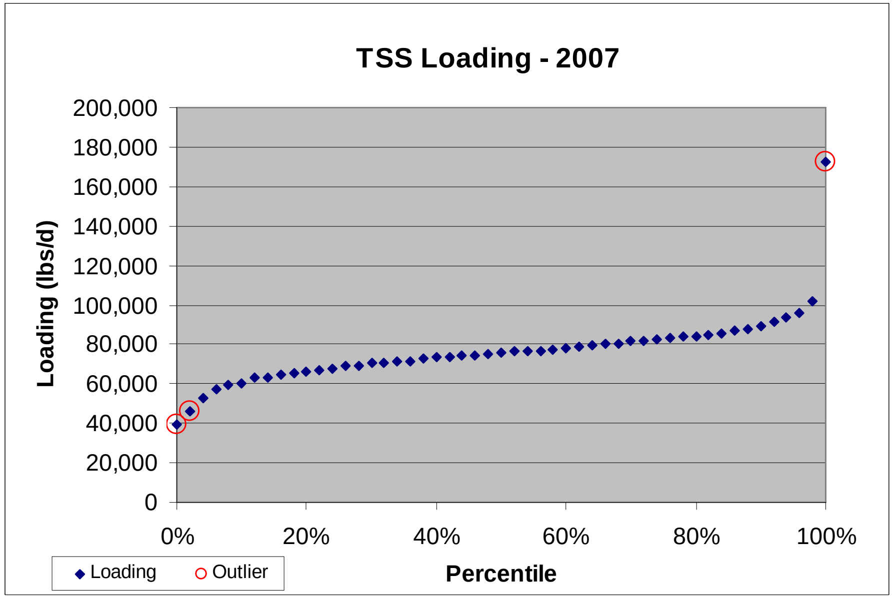
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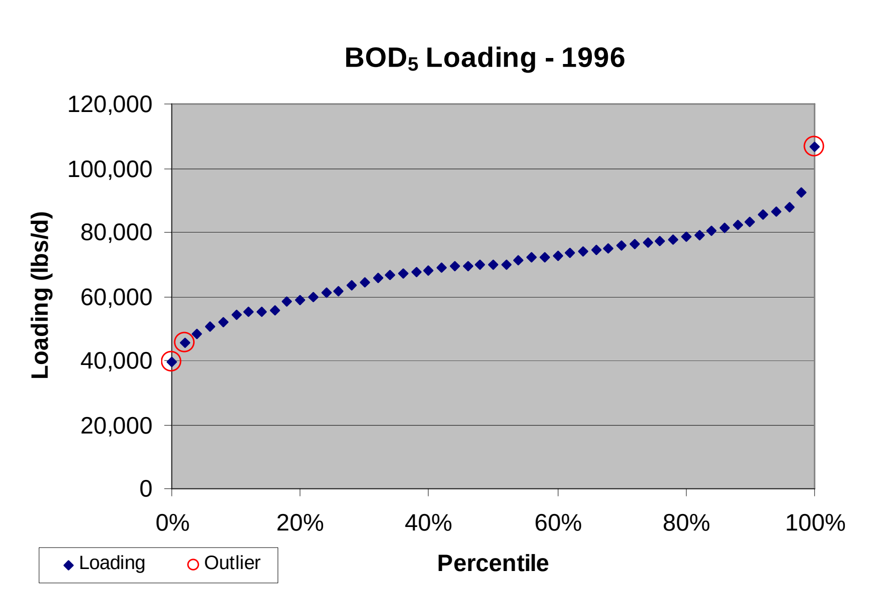
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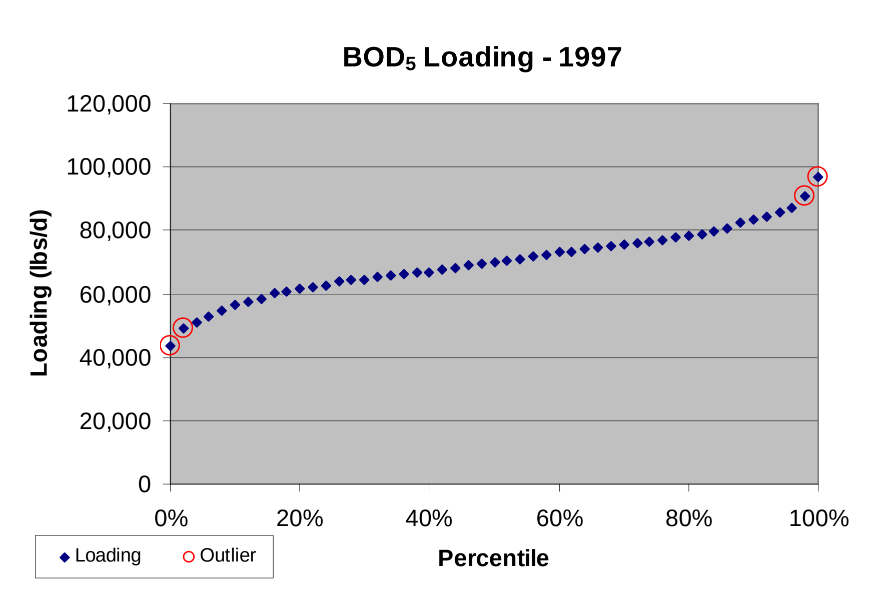
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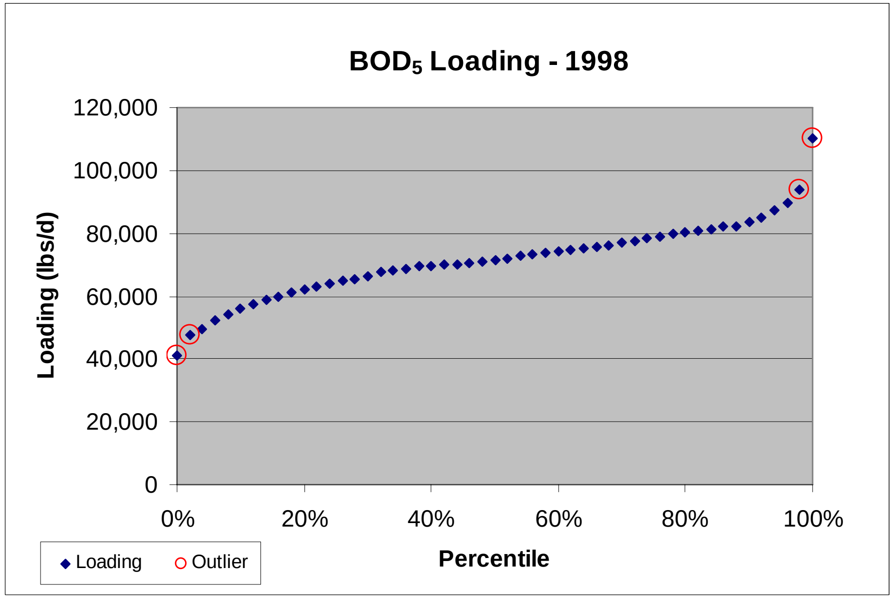
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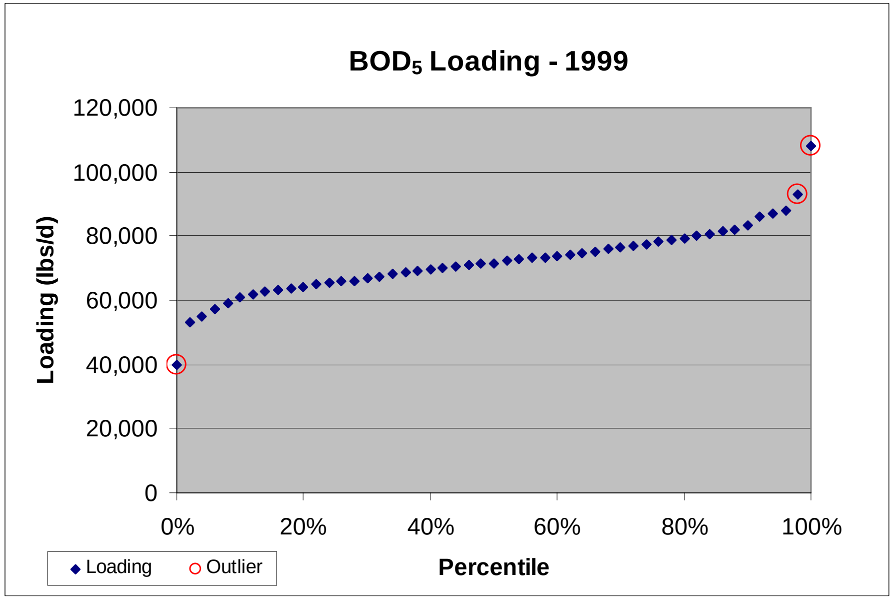
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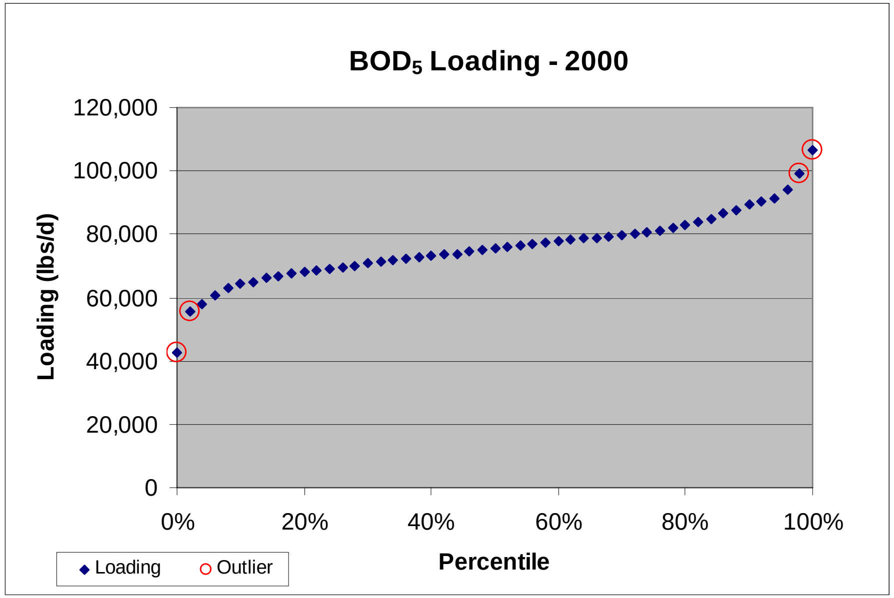
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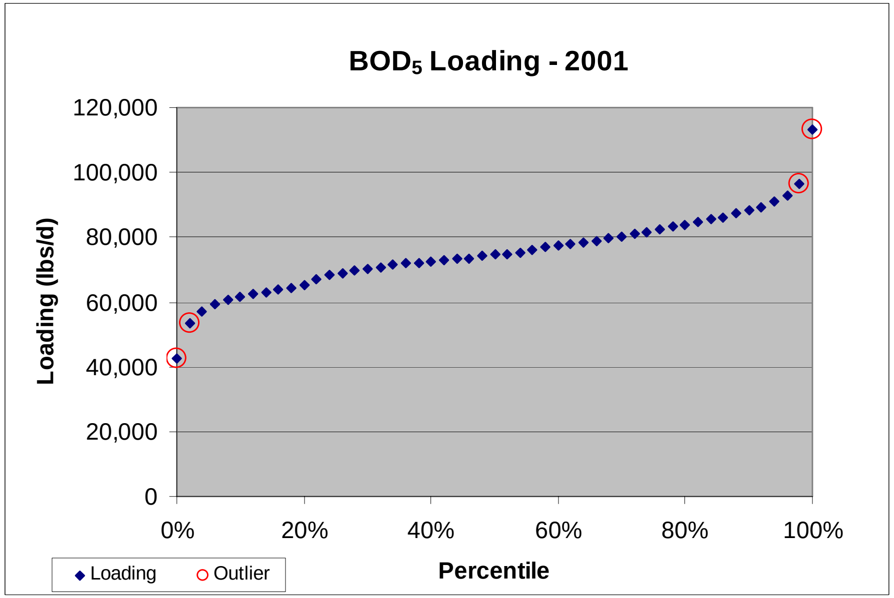
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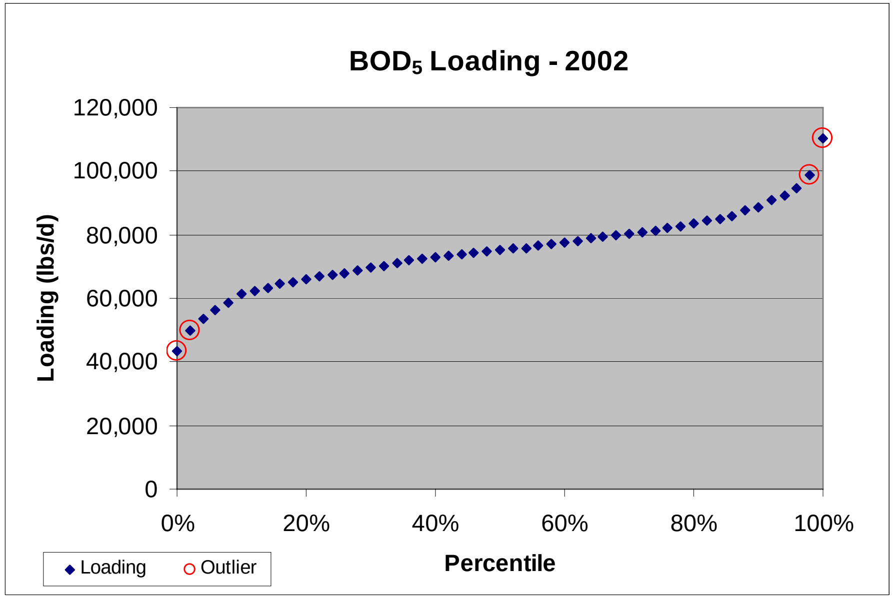
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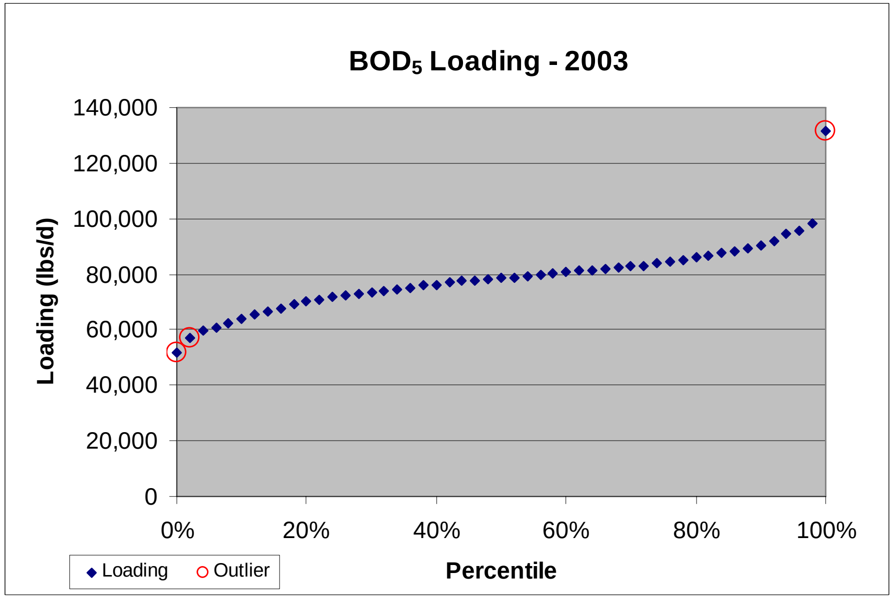
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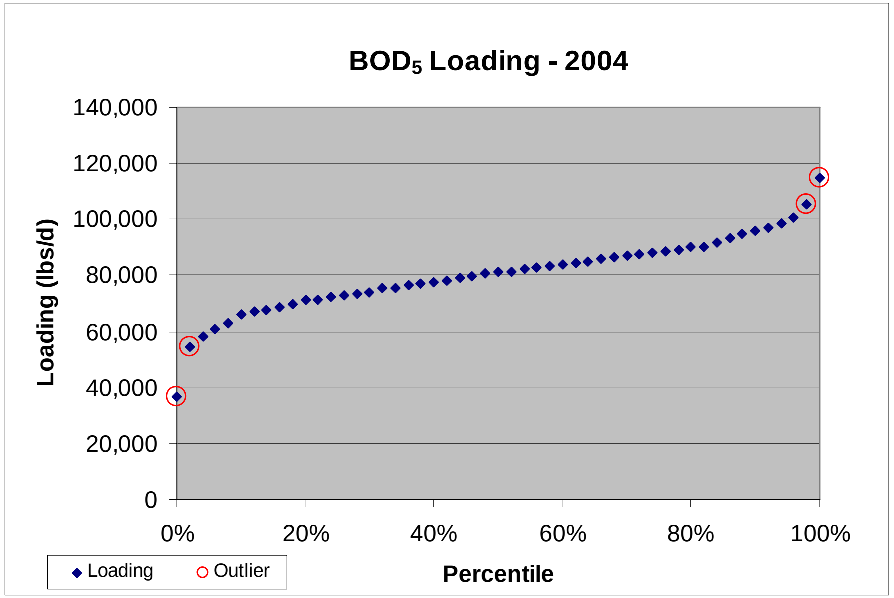
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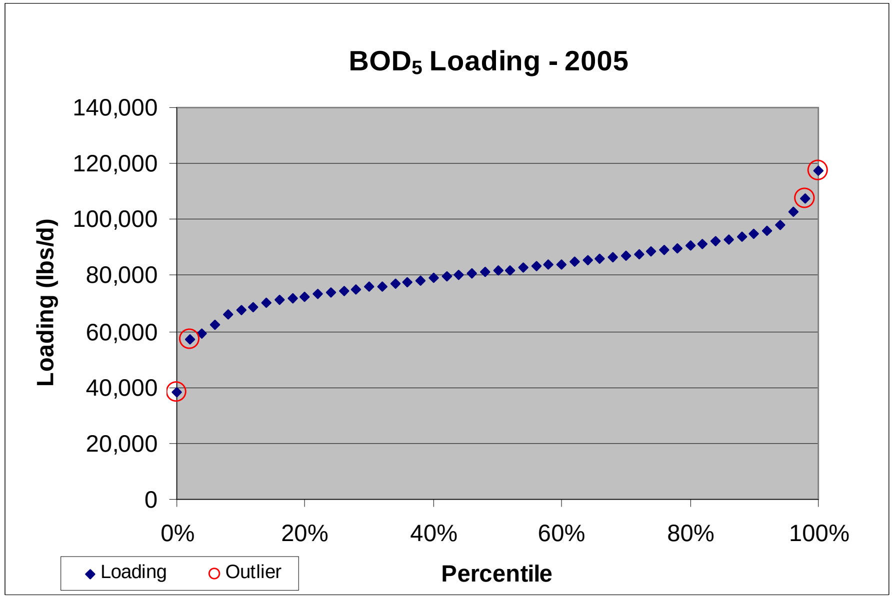
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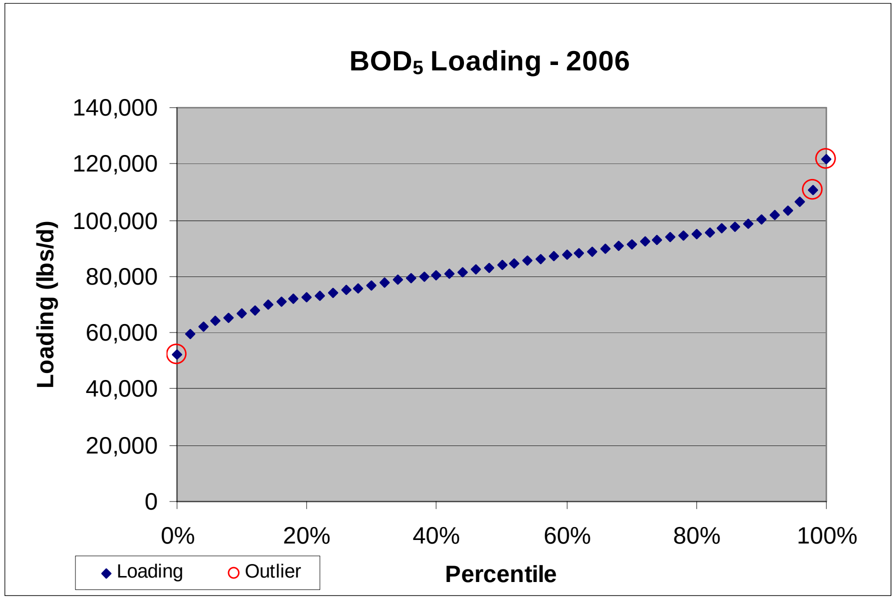
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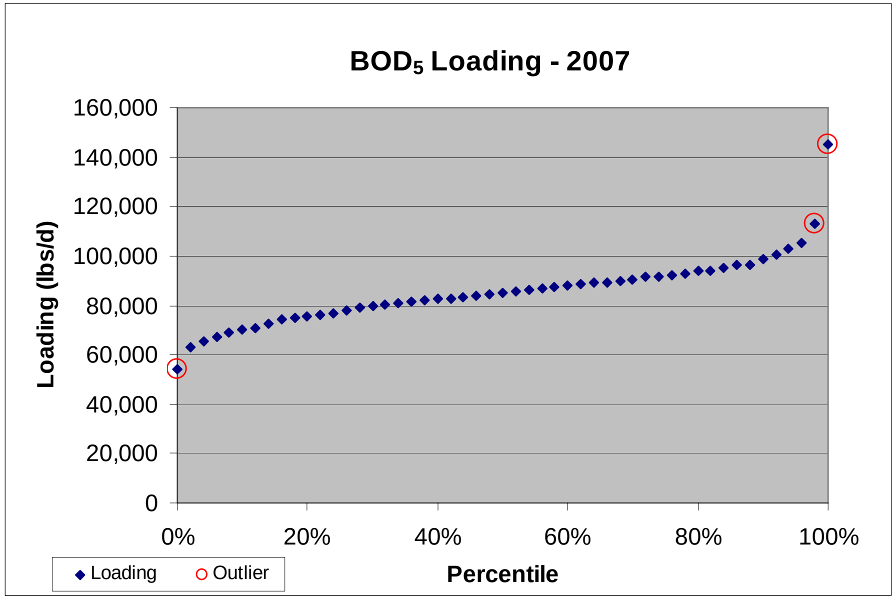
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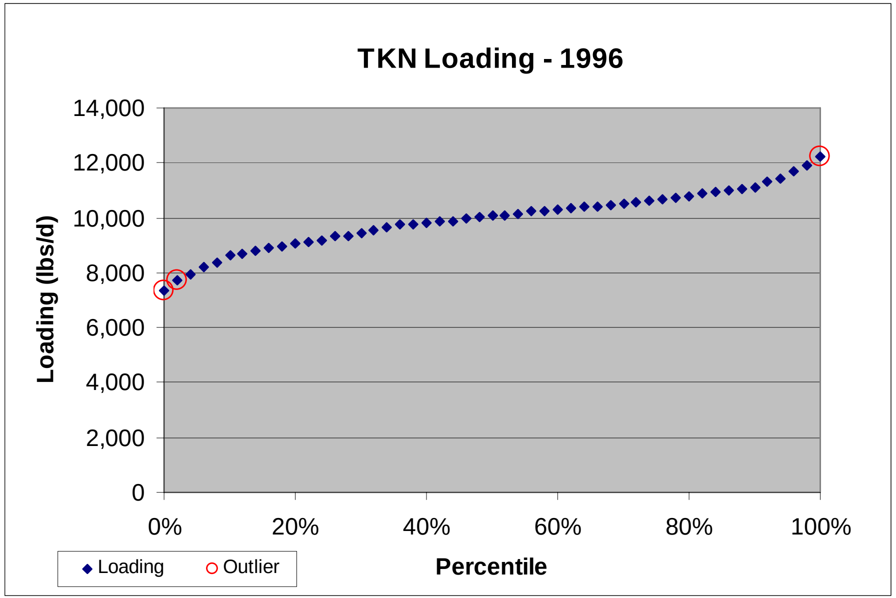
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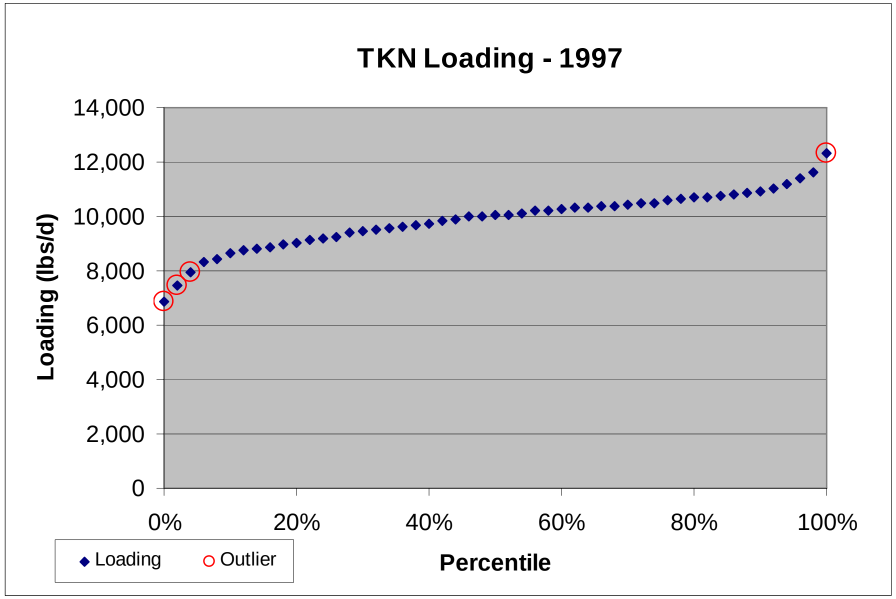
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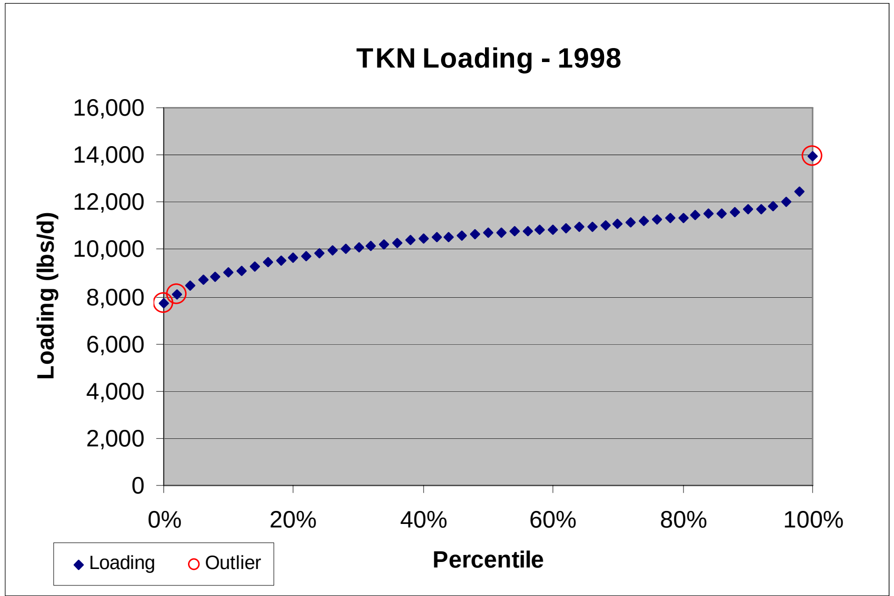
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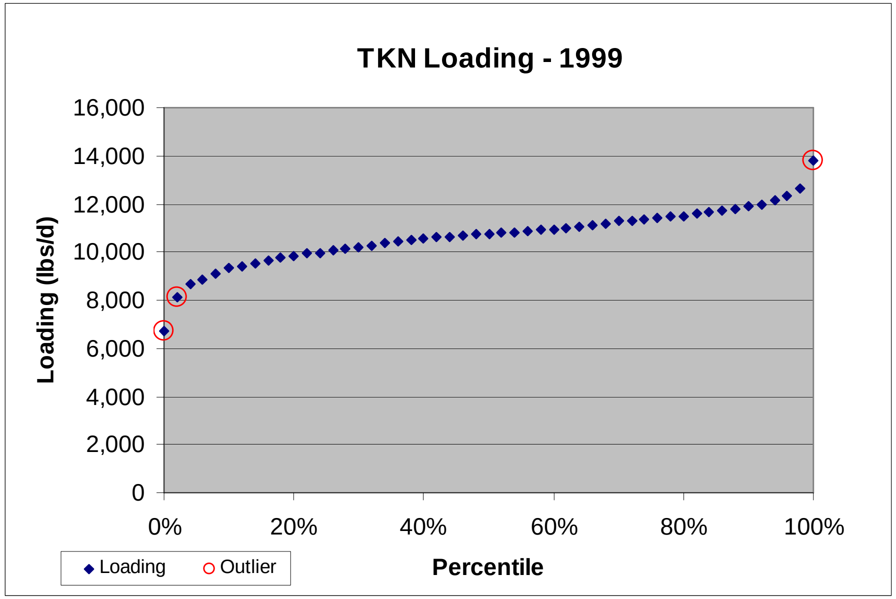
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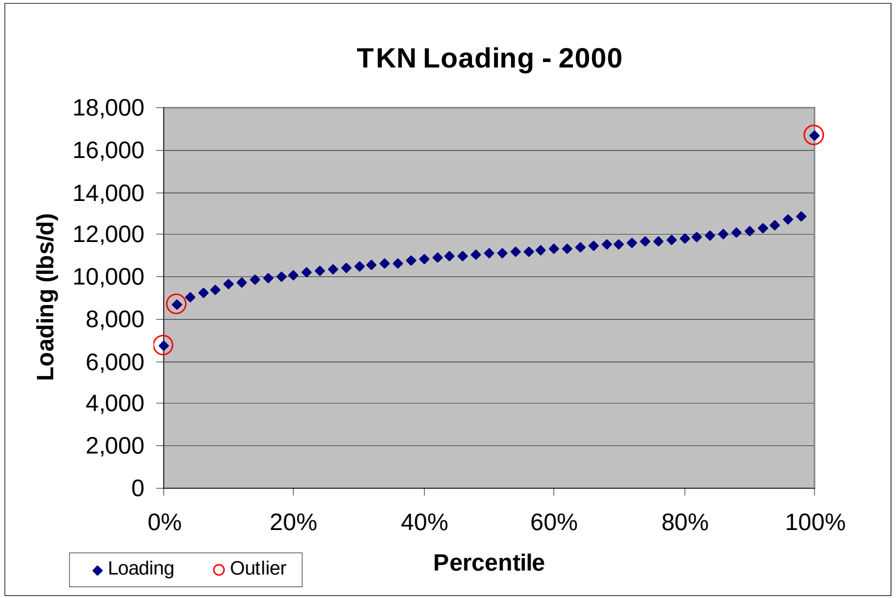
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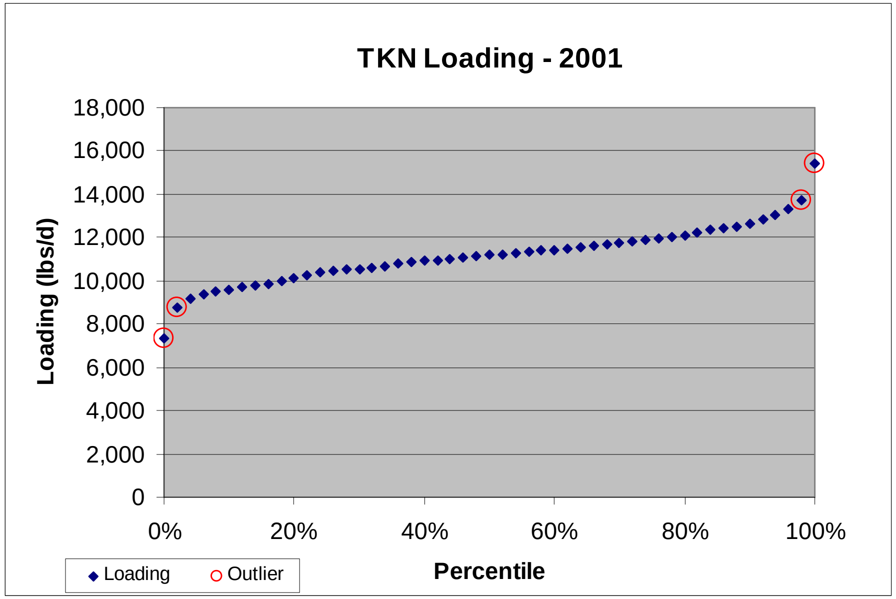
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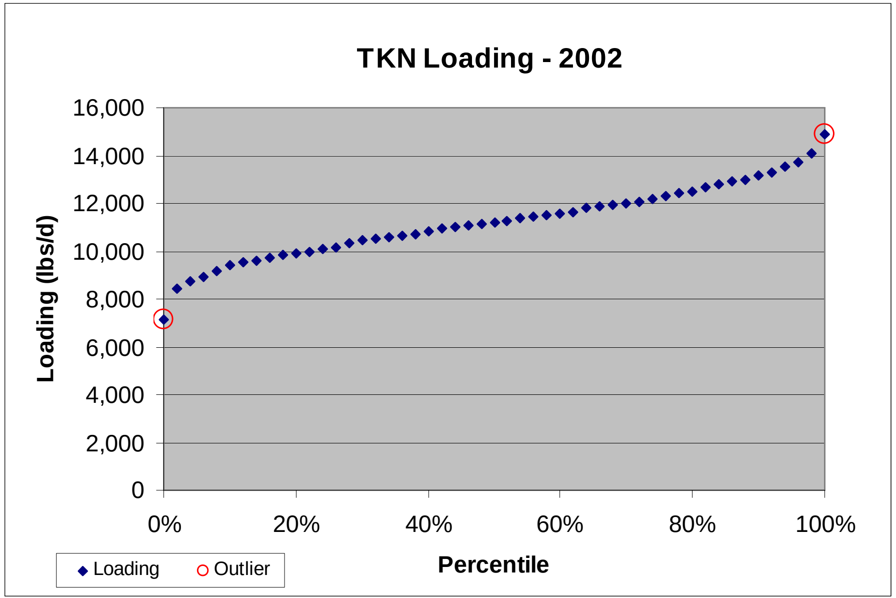
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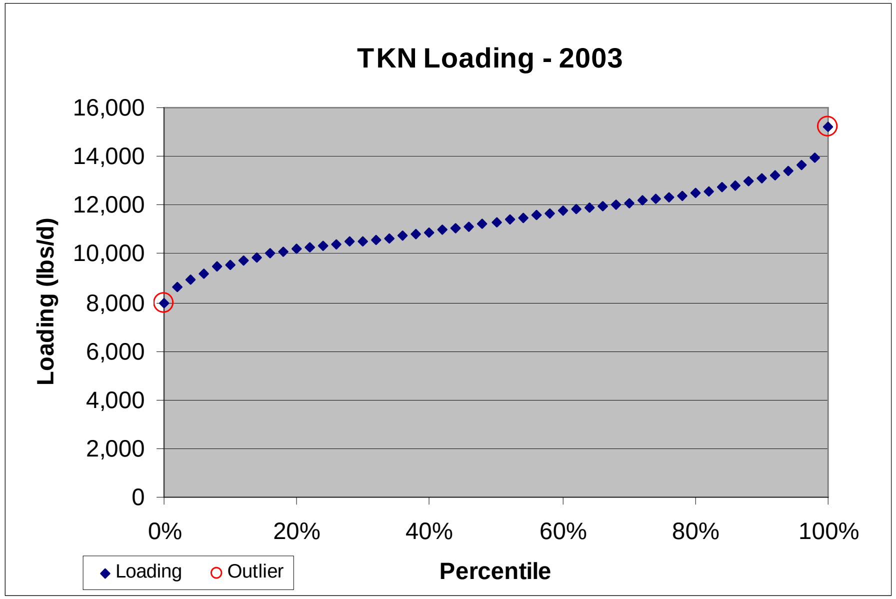
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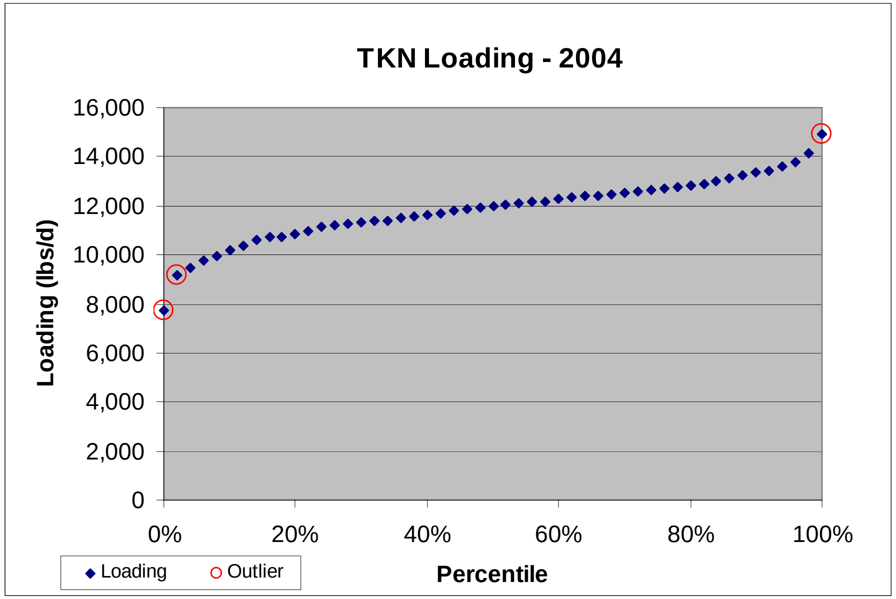
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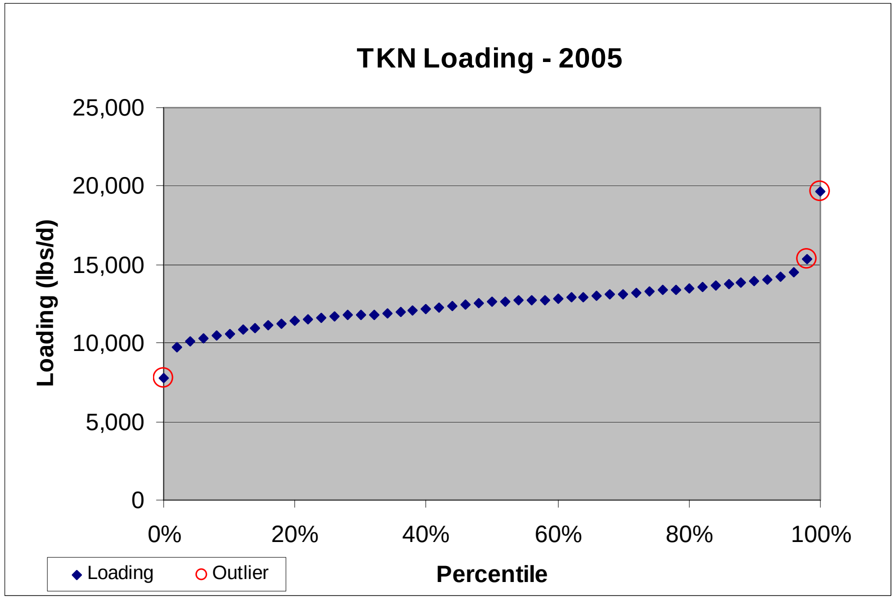
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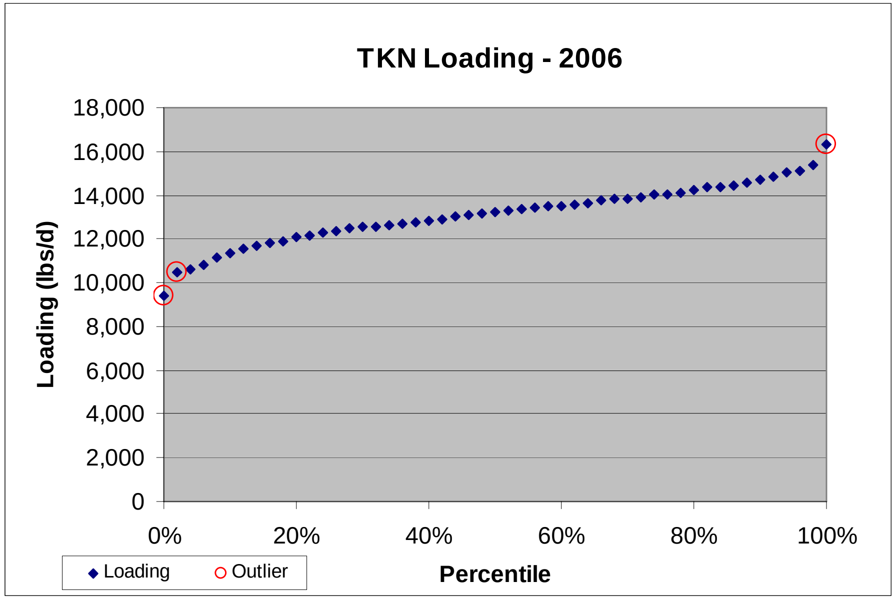
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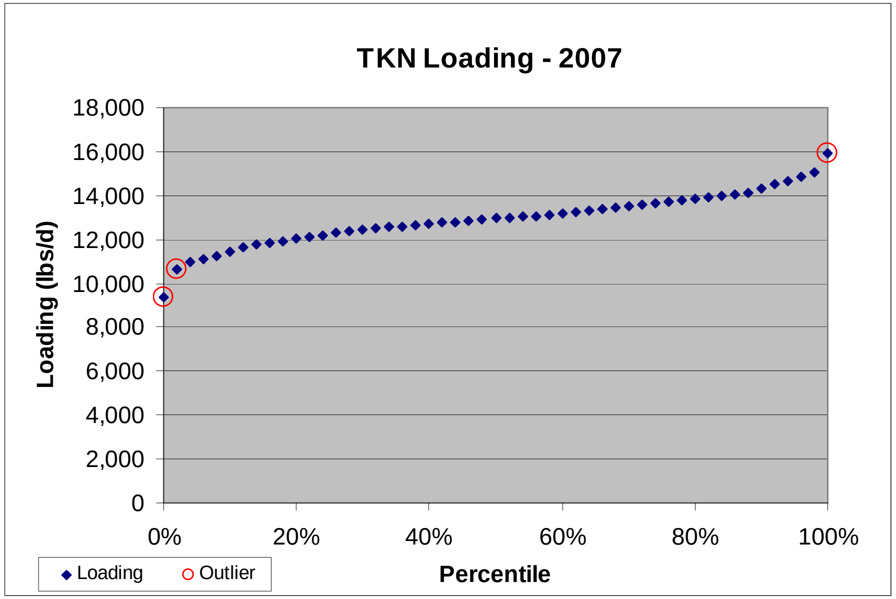
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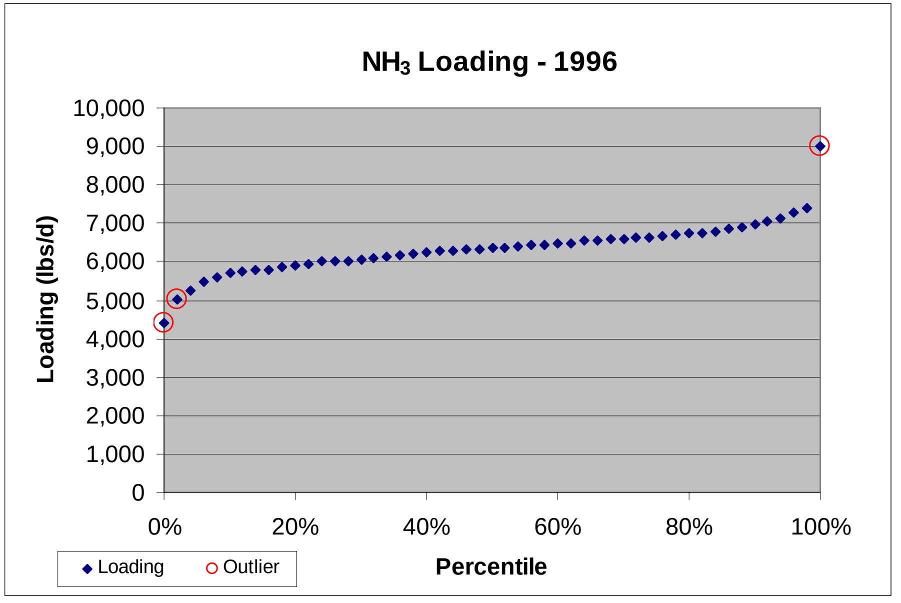
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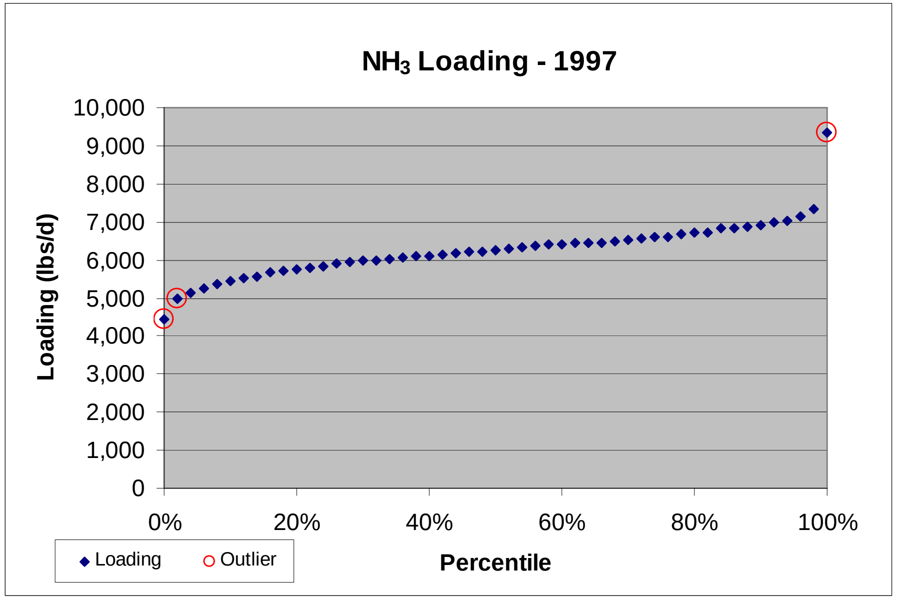
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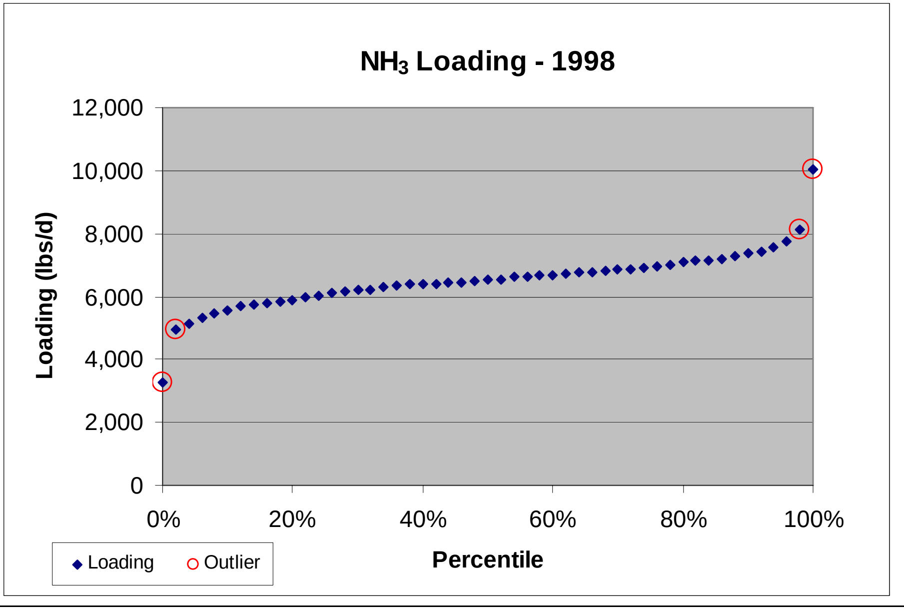
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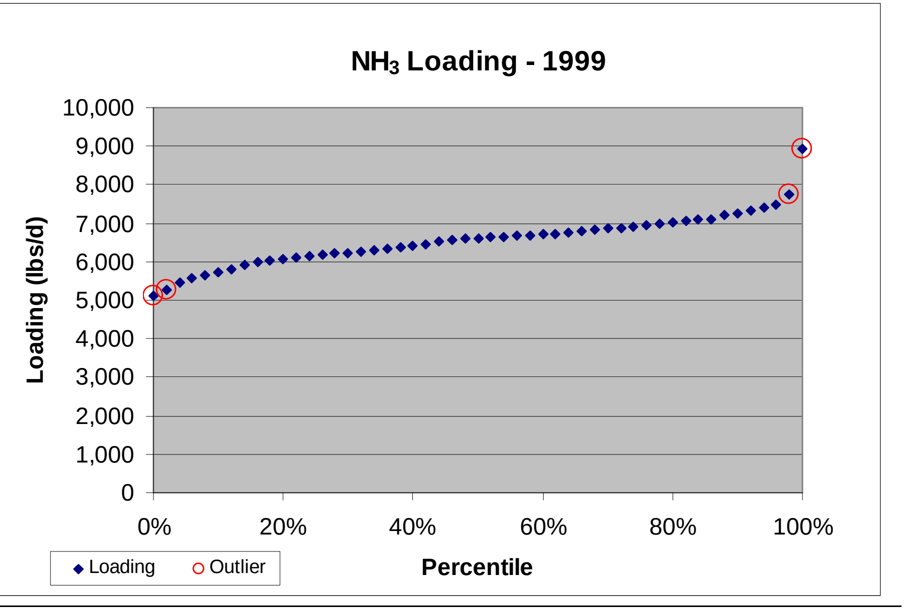
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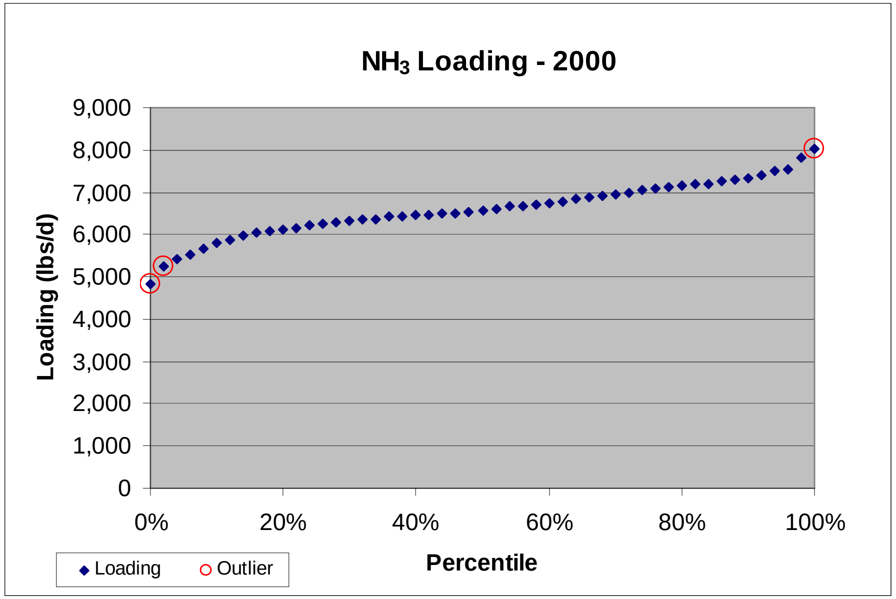
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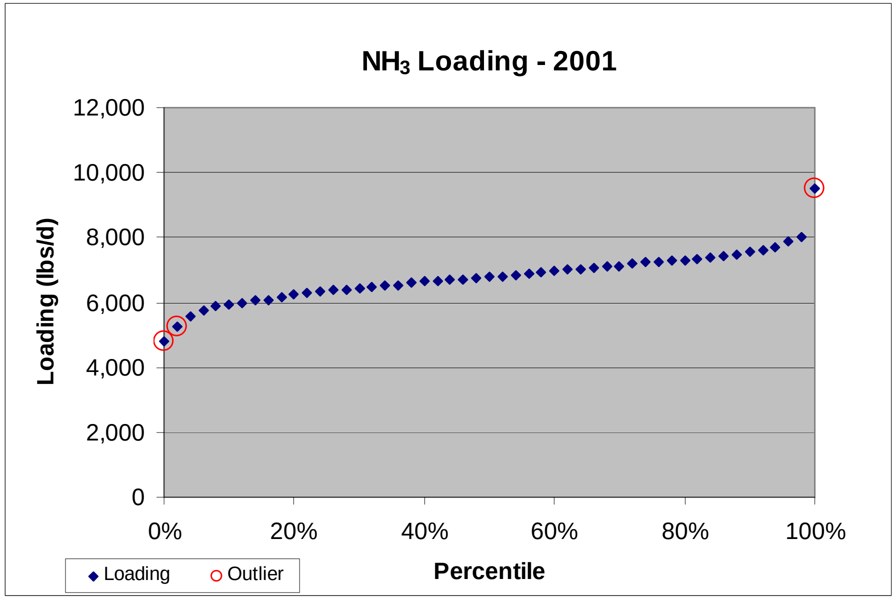
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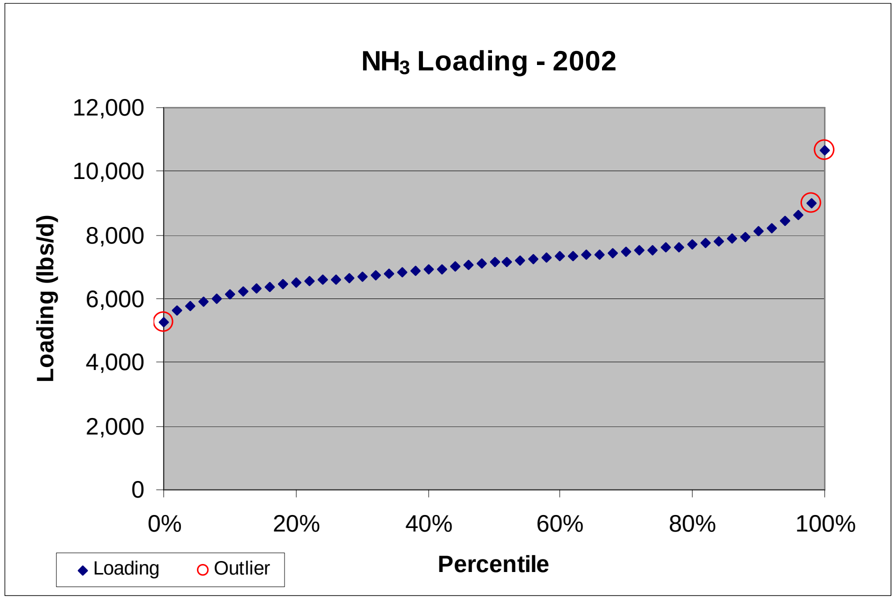
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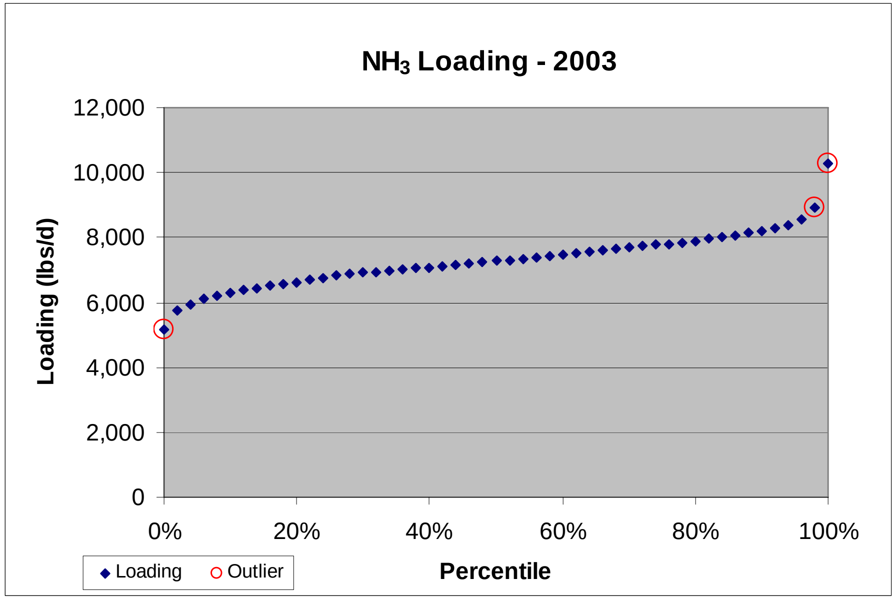
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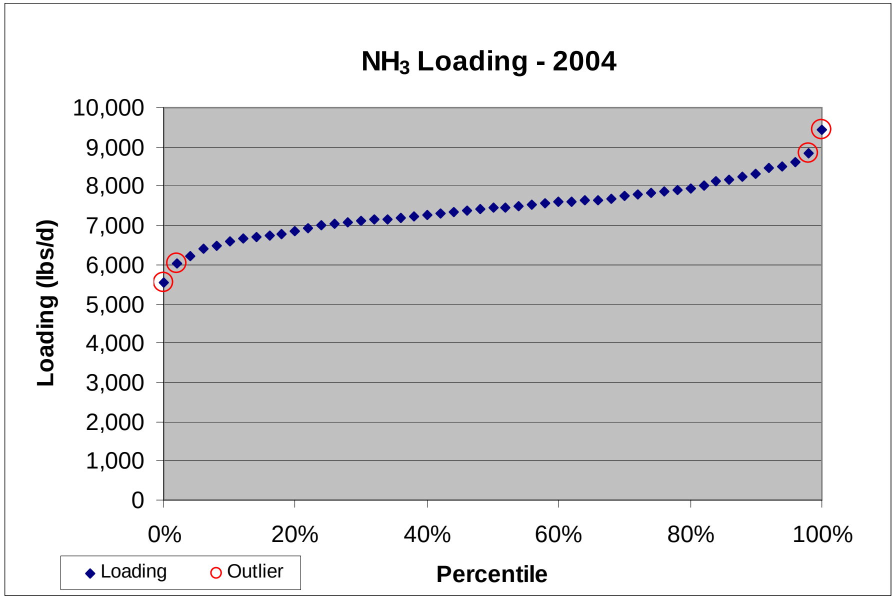
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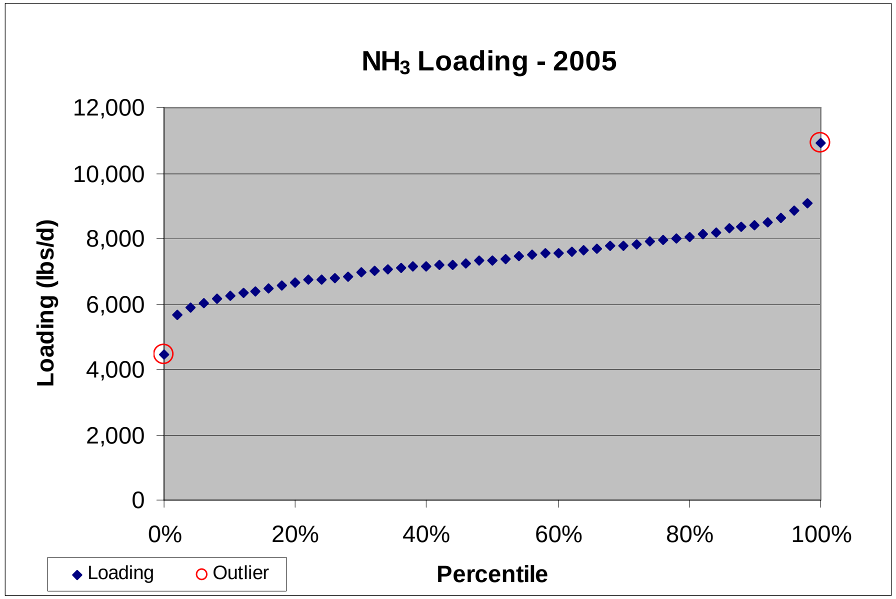
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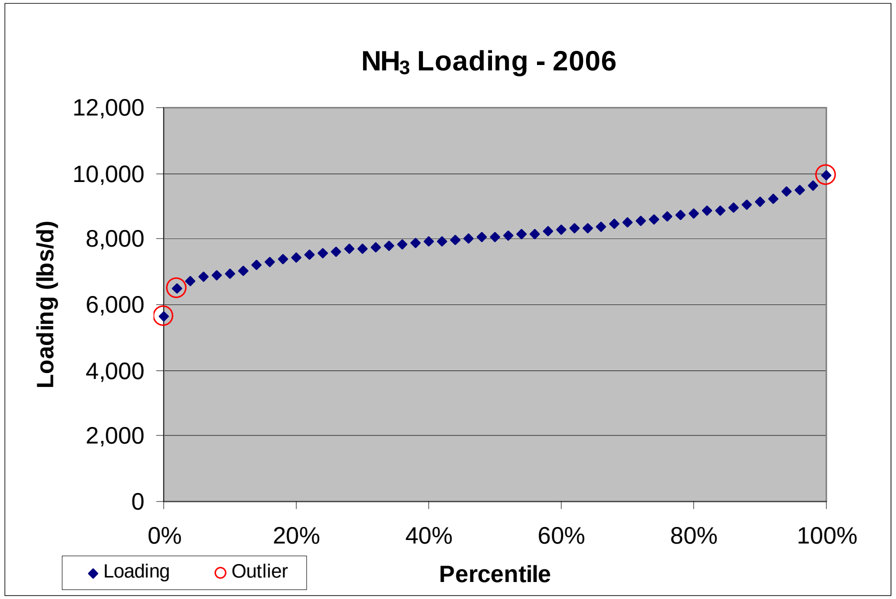
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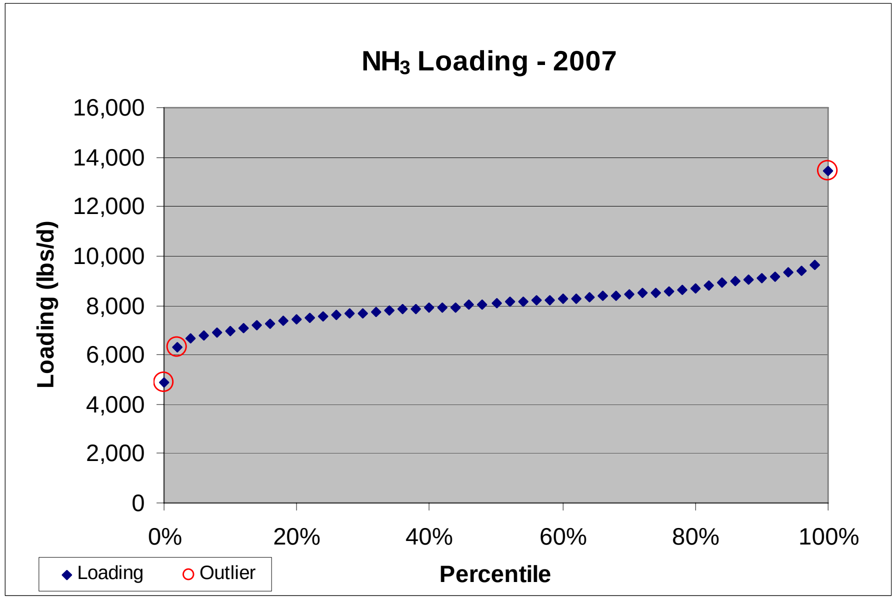
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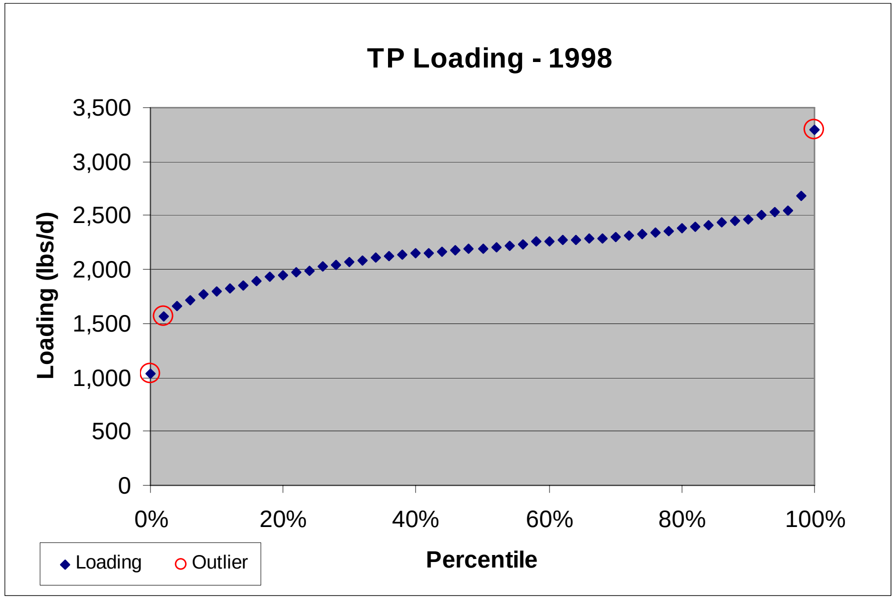
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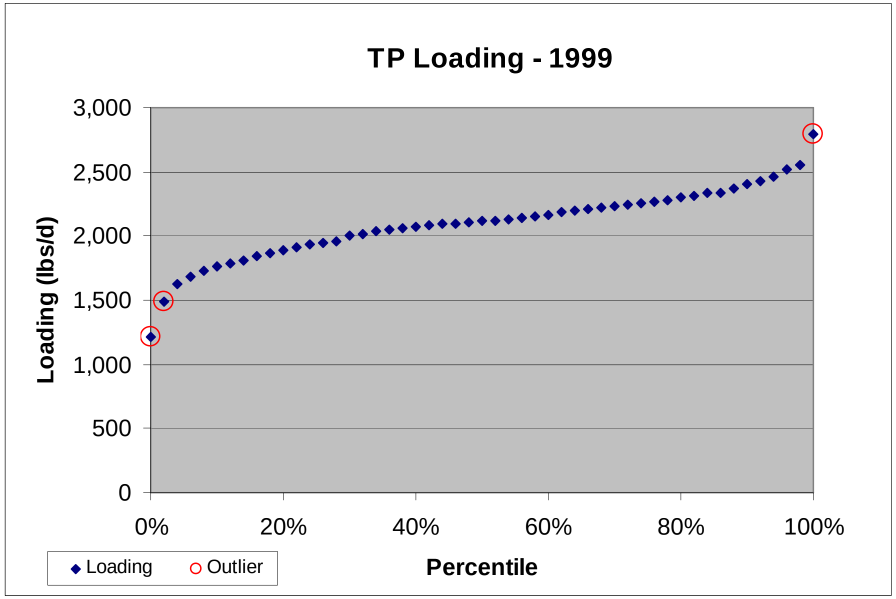
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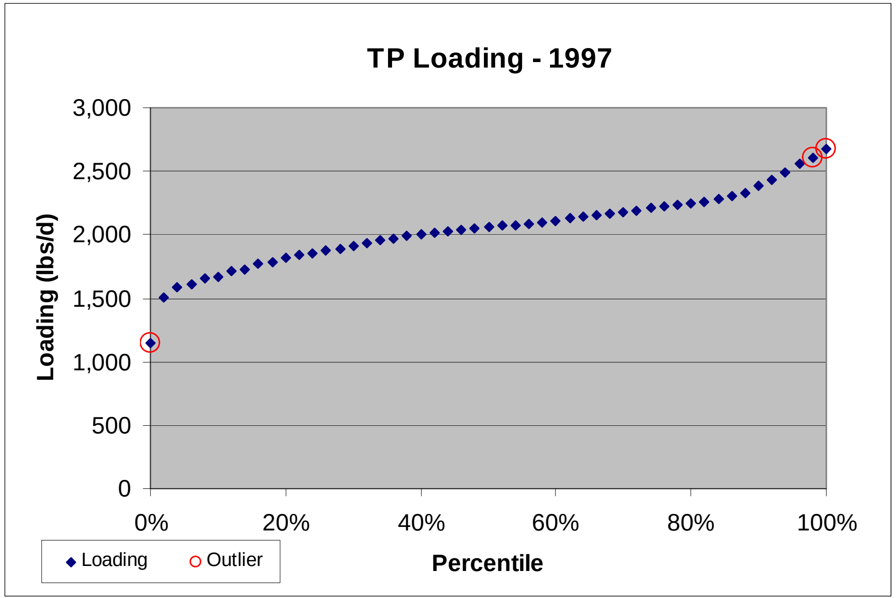
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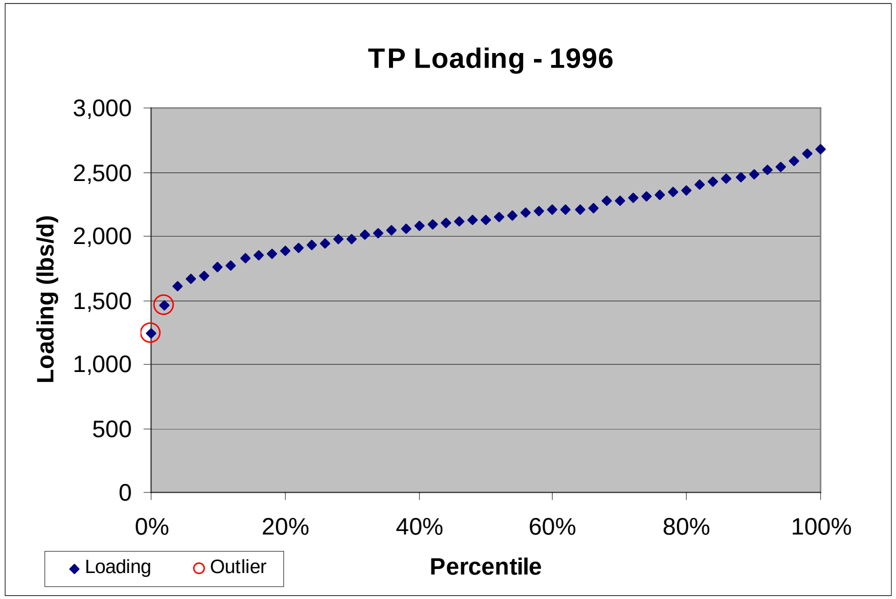
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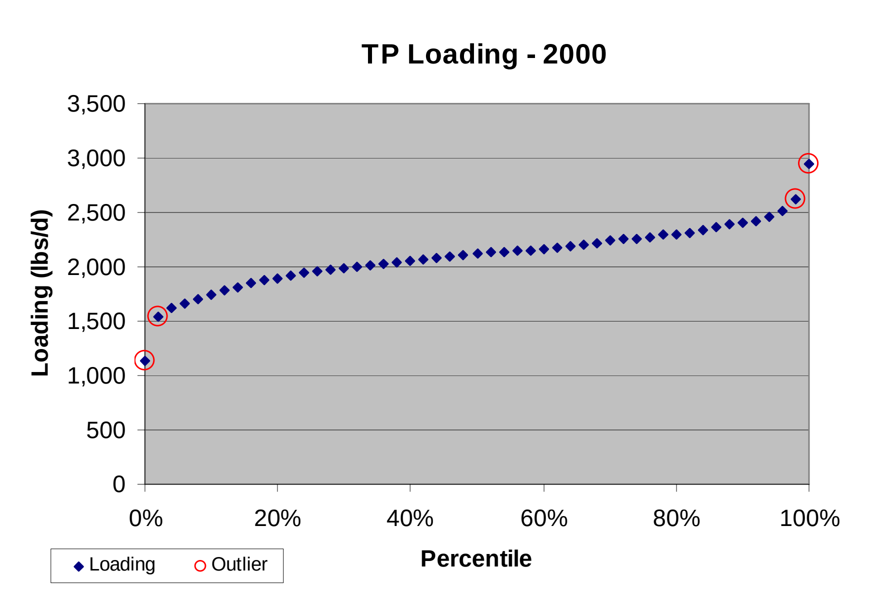
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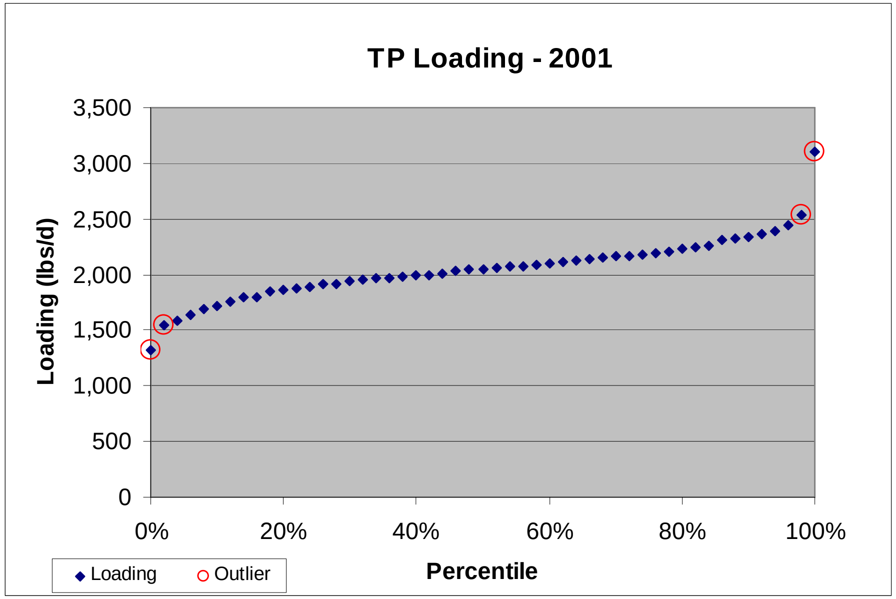
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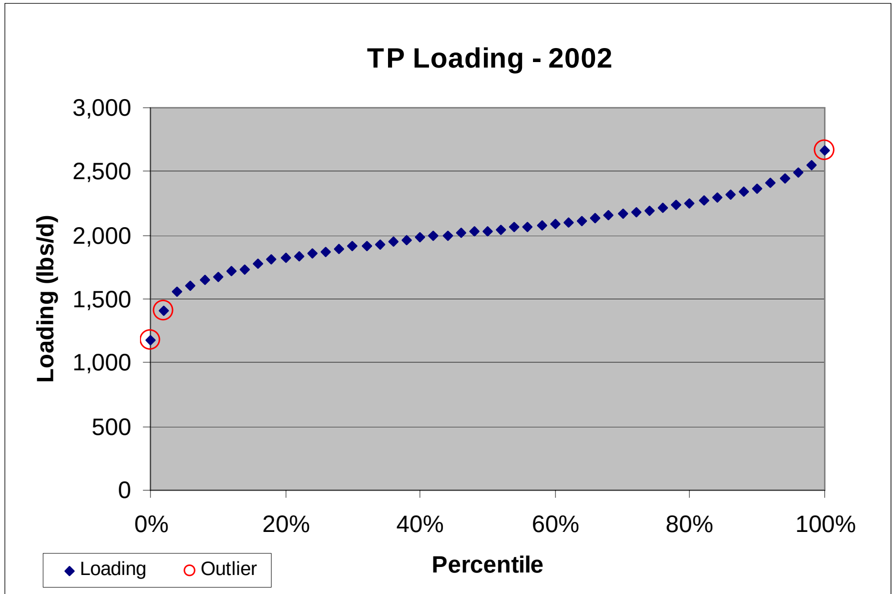
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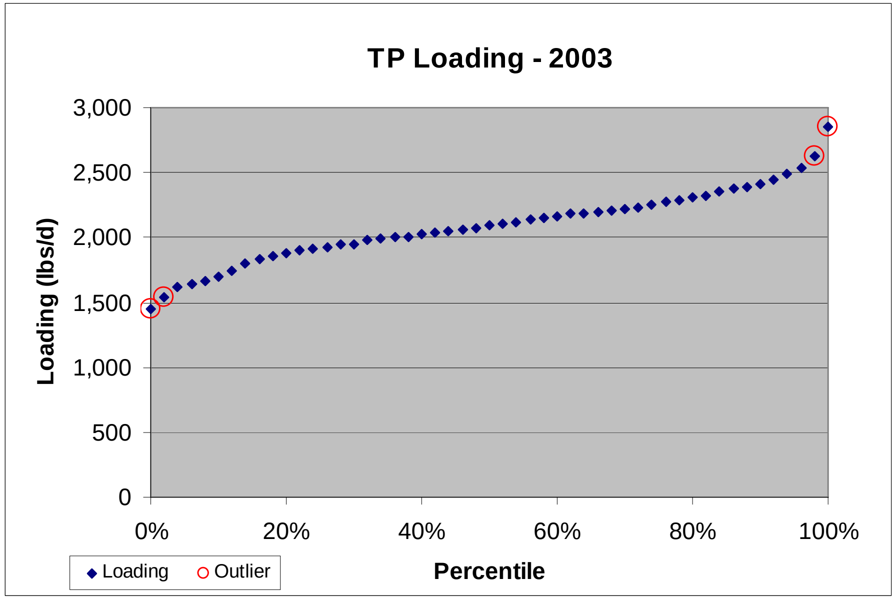
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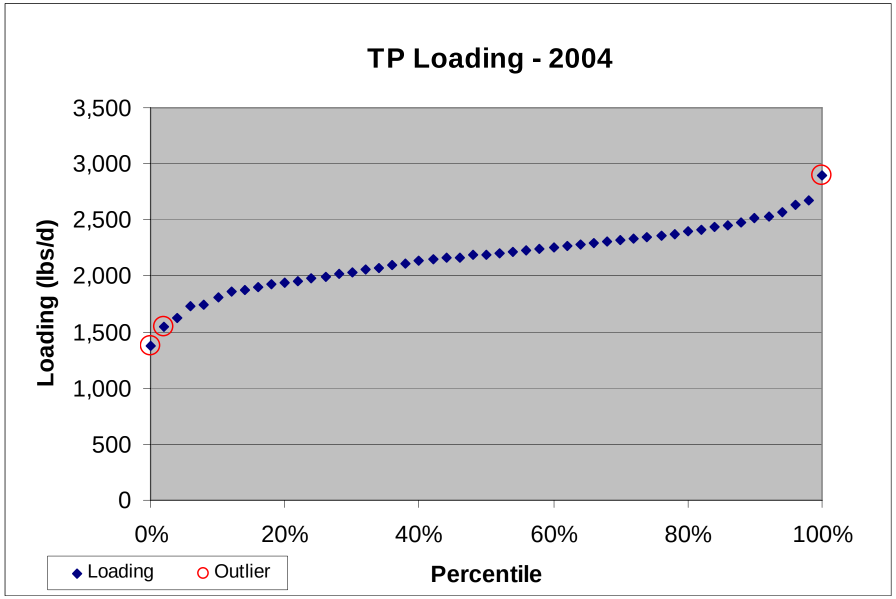
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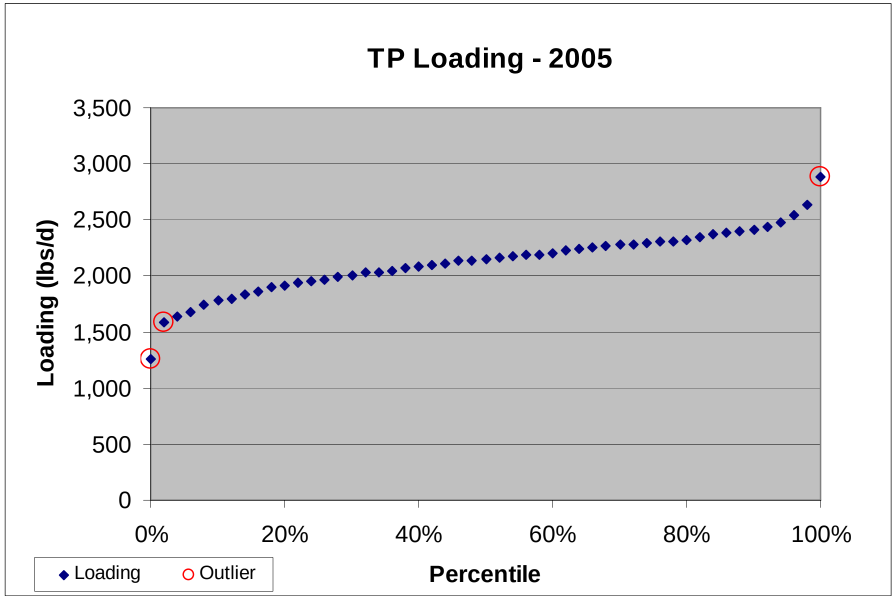
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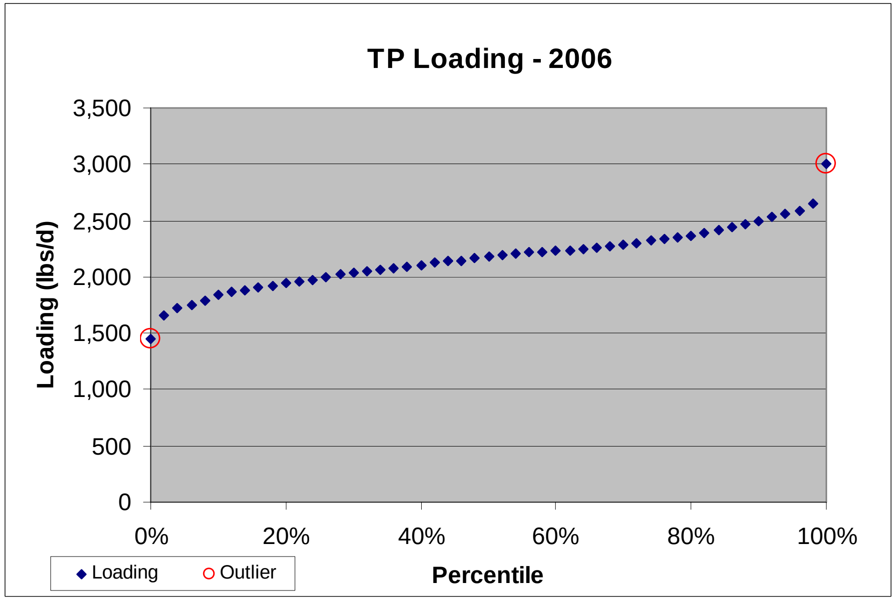
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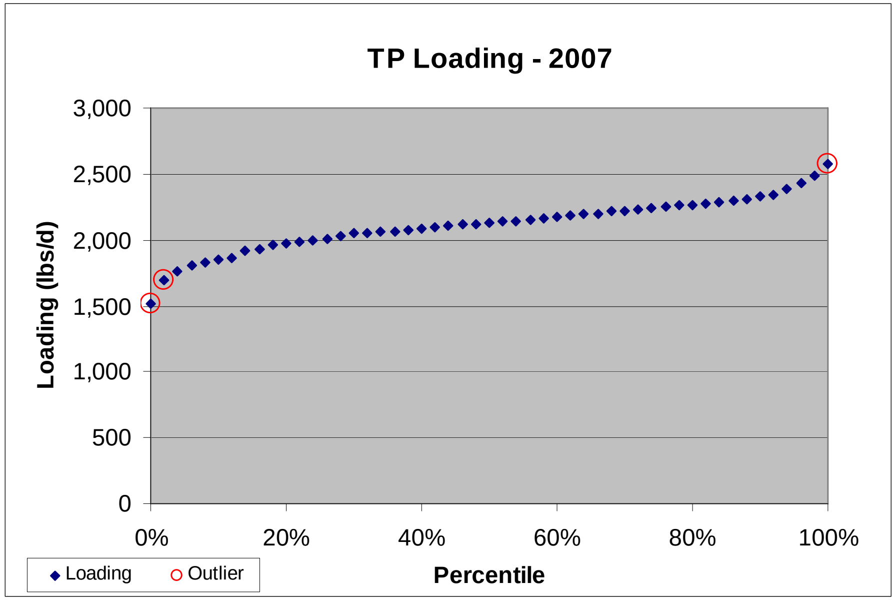
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Technical Memorandum No.2

Date: May 12, 2009

To: Madison Metropolitan Sewerage District

From: Steve McGowan, P.E., BCEE
Project Manager, Malcolm Pirnie, Inc.
Eric Wang, P.E.
Project Engineer, Malcolm Pirnie Inc

Subject: 50-Year Master Plan
TM-2: Flow and Loading Projections (Final)

Project No.: MMSD No. 8425001
MPI No. 6100-001

1. Introduction

This memorandum documents projected flows and loadings for the 50-year planning period. The memorandum presents information regarding the plant influent flow and loading projections, and internal loadings which will result from the projected influent flows and loadings. The projected internal flows and loadings were compared with the rated unit process capacities determined in Technical Memorandum 1 – Review of Existing Treatment Facilities. The comparison results provide information to be used for identifying the system needs for the planning period.

2. Influent Flows

Influent flows to the plant consist of raw wastewater delivered from the District service area via four force mains and of septage, holding tank, landfill leachate and other wastes that are trucked to the plant.

Raw Wastewater

The Capital Area Regional Planning Commission (CARPC) developed population and raw wastewater flow projections as part of the master planning for the District's sewerage collection system. The population and flow projections that CARPC developed are summarized in Table 2-1.

Table 2-1. Population and Flow Projection Summary

Year	Population	Average Raw Wastewater Flow (MGD)
2030*	431,000	50
2030**	555,000	60
2060*	555,000	60
2060**	605,000	70

** Population and flow based upon the projections developed by CARPC and represent the low estimates.*
***Population and flow based upon the projections developed by CARPC and represent the high estimates.*

Two estimates were generated for each planning year: the low estimate and the high estimate. The high estimates are based on the full build-out of each community's service area based on the community's Comprehensive Plan, while the low estimates are based on the Wisconsin Department of Administration (DOA) projected population data for Dane County.

The high estimate flow projections are used for the pumping station structures and conveyance system capacity evaluation since the service lives of these facilities are 75 to 100 years, and it is uncertain where higher growth will occur in the District's service area. The low estimate flow projections are used for the Nine Springs Wastewater Treatment Plant (NSWTP) facility capacity evaluation because the DOA-based estimates reflect the more probable total population and flow data scenarios when analyzing future design conditions at the NSWTP.

Septage, Holding Tank, Landfill Leachate and Other Wastes

Wastes from septic tanks, holding tanks, landfill leachate and commercial establishments (grease traps, settling basins, and portable toilets) are trucked to the plant for treatment. Table 2-2 presents daily average flow of wastes from septage, holding tanks, grease traps and settling basins at a given year and the flow as percentage of the average daily flow of raw wastewater in that year.

Table 2-2. Septage, Holding Tank, Grease Trap and Settling Basin

Year	Daily Average Flow (gpd)	Daily Average Flow (%)
1996	24,793	0.06
1997	23,975	0.06
1998	27,480	0.07
1999	25,806	0.06
2000	26,362	0.06
2001	31,242	0.07

2002	30,031	0.07
2003	31,197	0.08
2004	37,726	0.09
2005	35,929	0.09
2006	41,268	0.10
2007	45,962	0.11
Average	31,814	0.08

The monitoring records of landfill leachate and other wastes were only studied for January 2005 through December 2007 to determine the flows and loadings. This period was selected for analysis because this period represents the plant's current operation. The District terminated receiving whey from Bancroft Dairy Company in 2005 and the hauled-in wastes flow has been significantly reduced since then. Table 2-3 presents daily average flow of landfill leachate and other wastes at a given year and the flow as percentage of the daily average flow of raw wastewater in that year.

Table 2-3. Leachate and Other Wastes

Year	Daily Average Flow (gpd)	Daily Average Flow (%)
2005	2,647	0.01
2006	2,906	0.01
2007	2,534	0.01
Average	2,696	0.01

The combined hauled-in waste flow accounts for approximately 0.1% of the raw wastewater daily average flow for a given year. The projected hauled-in waste flows and total plant flows are presented in Table 2-4.

Table 2-4. Plant Flow Projections

Year	Raw Wastewater (MGD)	Hauled-in Wastes (MGD)	Total (MGD)
2030*	50	0.054	50
2030**	60	0.066	60
2060*	60	0.064	60
2060**	70	0.074	70

** Flow based upon the projections developed by CARPC and represents the low estimates.*
***Flow based upon the projections developed by CARPC and represents the high estimates.*

3. Influent Loadings

Influent loadings to the plant consist of raw wastewater delivered from the District service area via four force mains and of septage, holding tank, landfill leachate and other wastes that are trucked to the plant.

Raw Wastewater

In the previous facilities planning studies by the District, the following three methods have been used in projecting future loadings for 5-day biochemical oxygen demand (BOD_5), total suspended solids (TSS), total Kjeldahl nitrogen (TKN), and total phosphorus (TP):

- **Method based on per capita loading factors and projected future population.** This method determined the current per capita loading factors by dividing current average loadings by the current population of the District service area. The resulting per capita loading factors were then multiplied by projected future populations to project future loadings.
- **Method based on current waste-load strength and projected future raw wastewater flow rates.** This method involved determining the current average concentrations of the parameters of interest and then multiplying the concentrations by projected future flow rates to project future loadings.
- **Method based on historic loading trends.** This method involved plotting historical raw wastewater loadings and then projecting future loadings based on linear regression of the historical loading trends.

In the previous facilities planning studies by the District, the results generated by these three methods were compared and discussed. The method based on historic loading trends was selected as the most appropriate method for projecting future loads because it reflected actual consistent increasing trends in wastewater loading at the Nine Springs Wastewater Treatment Plant (NSWTP). Table 3-1 presents the comparison between the actual recorded loadings at the plant and the values projected in the Tenth Addition facilities planning.

Table 3-1. Loading Projection Comparison

Parameter	Year 2000		Year 2005	
	10th Addition Projections	Actual	10th Addition Projections*	Actual
BOD_5 (lbs/d)	73,000	75,424	81,500	81,648
TSS (lbs/d)	71,000	78,127	78,000	80,197
TKN (lbs/d)	10,700	11,045	11,750	12,439
TP (lbs/d)	2,210	2,102	2,430	2,132

** Projected loadings for Year 2005 were calculated assuming linear growth between Year 2000 and 2010.*

The results show that the actual plant BOD_5, TSS and TKN loadings match fairly well with the projected values, while TP loadings do not show consistent growth trend as projected. The inaccurate TP projections may result from lack of long term monitoring data for plant influent TP loadings when the study was conducted. Overall, the results generated by linear projection based upon historical loading trends method provided fairly accurate and reasonable future loading projections. Therefore, this method was also used in projecting future loadings to the plant in this study. Figures 3-1 through 3-4 show historic and projected loadings for BOD_5, TSS, TKN, and TP, respectively. Each figure shows the historical monthly average loadings and a linear regression of the monthly average values projected out through the planning period.

Figure 3-1. Raw Wastewater BOD_5 Loading Projection

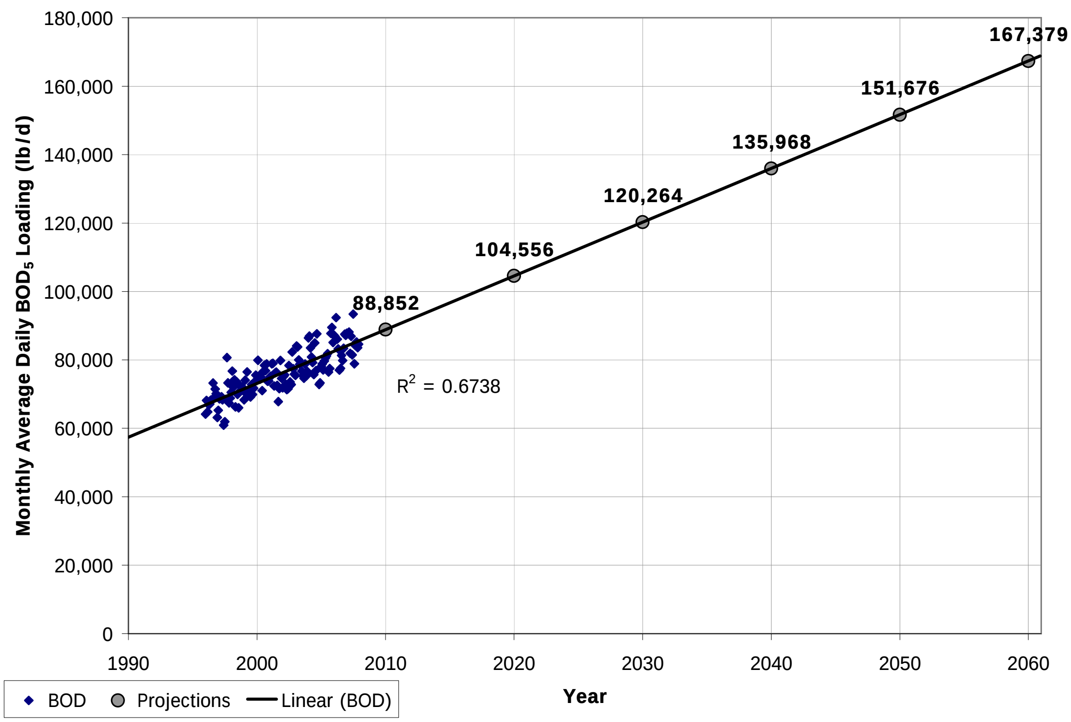


Figure 3-2. Raw Wastewater TSS Loading Projection

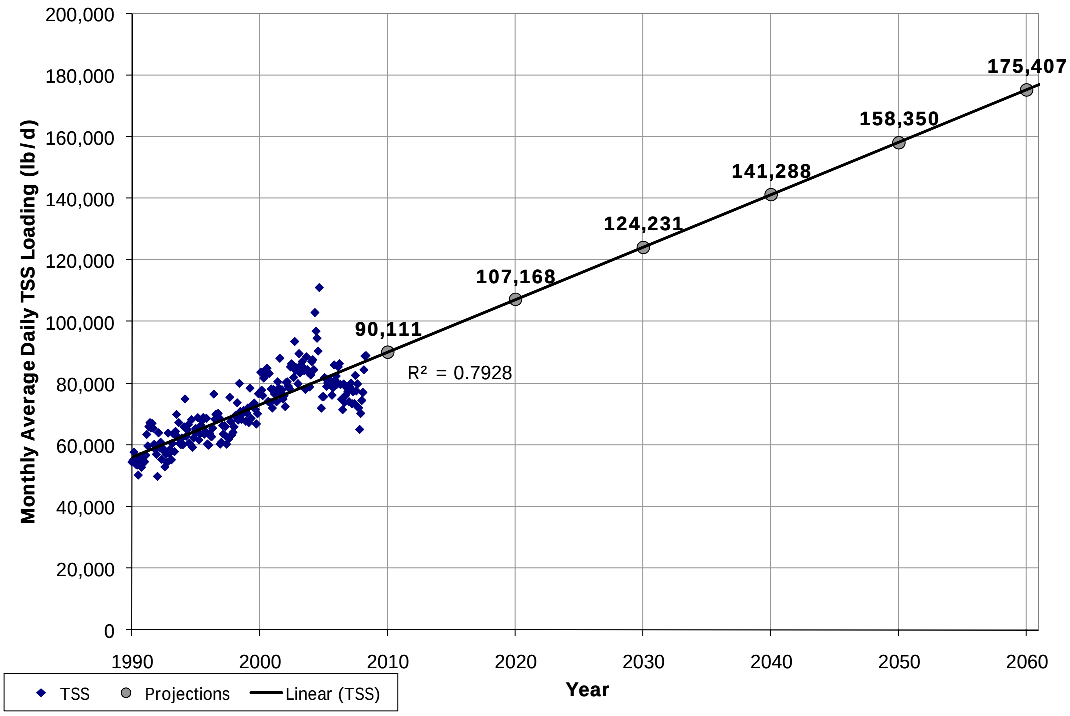


Figure 3-3. Raw Wastewater TKN Loading Projection

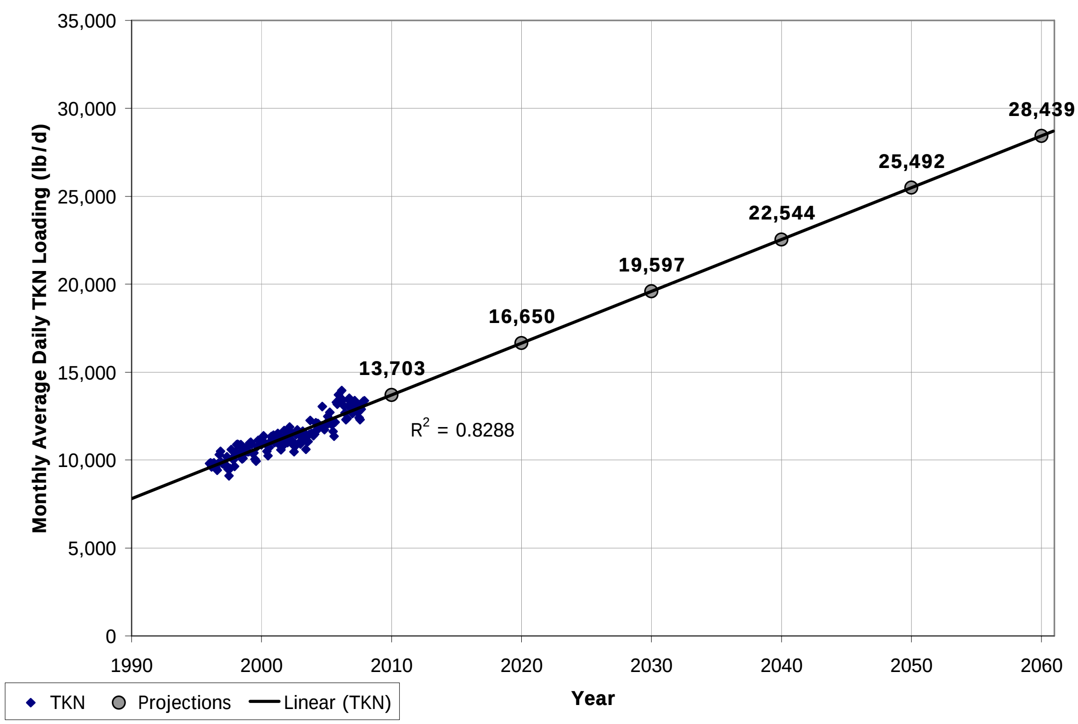


Figure 3-4. Raw Wastewater TP Loading Projection

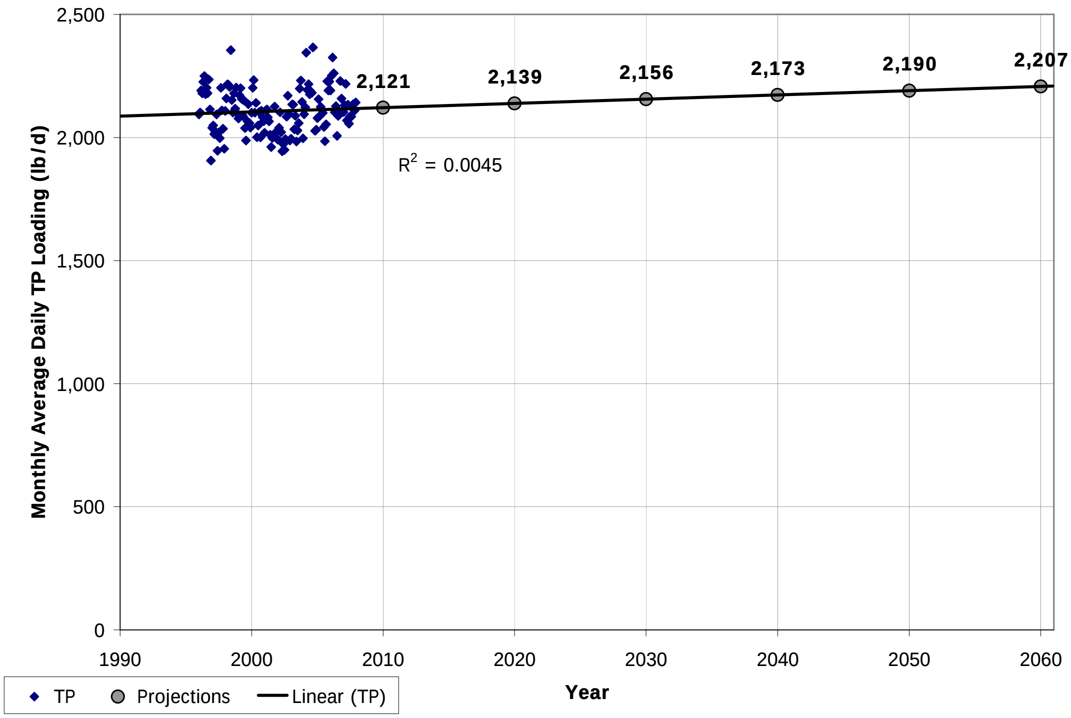


Table 3-2 presents the summary of projected raw wastewater loadings based upon historic loading trends.

Table 3-2. Loading Projections Based on Historic Loading Trends

Parameter	Year 2030	Year 2060
BOD_5 (lbs/d)	120,000	170,000
TSS (lbs/d)	124,000	175,000
TKN (lbs/d)	20,000	28,000
TP (lbs/d)	2,150	2,200

Based on previous facilities planning studies by the District, linear projection based on historical loading trends has proven to be a reasonable and reliable method in predicting the BOD_5, TSS, and TKN loadings to the plant. However it is not appropriate in predicting TP loadings due to lack of long term influent TP monitoring data. The average TP concentrations of the raw influent were found to be fairly stable for the past 12 years, ranging from 5.9 – 6.8 mg/L. The results are presented in Table 3-3.

Table 3-3. Historical Raw Influent TP Concentrations

Year	Average TP Loading (lbs/day)	Average Daily Flow (MGD)	Average TP Concentration (mg/L)
1996	2,150	38.2	6.8
1997	2,036	36.9	6.6
1998	2,180	41.1	6.4
1999	2,109	41.6	6.1
2000	2,102	42.1	6.0
2001	2,045	41.8	5.9
2002	2,039	40.1	6.1
2003	2,087	38.6	6.5
2004	2,186	41.9	6.3
2005	2,132	39.4	6.5
2006	2,165	41.2	6.3
2007	2,125	43.0	5.9
Average	2,113	40.5	6.3

Because of the stability of the influent TP concentration and unclear TP loading trends, the method based on waste-load strength was used to project TP loadings in the planning period. Table 3-4 lists projected TP loadings for 2030 and 2060. Projected loadings for the planning period were determined by multiplying the average TP concentration by the CARPC projected raw wastewater flow rates.

Table 3-4. TP Loading Projection Based on Waste-Load Strength

Parameter	Value
Average TP Concentration	6.3 mg/L
Year 2030 Flow*	50 MGD
Year 2030 TP Loading	2,700 lbs/day
Year 2060 Flow*	60 MGD
Year 2060 TP Loading	3,200 lbs/day

**Flow based upon the projections developed by CARPC and represents the low estimates.*

Table 3-5 summarizes the raw wastewater loading projections for the planning period.

Table 3-5. Raw Wastewater Loading Projections

Parameter	Year 2030	Year 2060
BOD_5 (lbs/d)	120,000	170,000
TSS (lbs/d)	124,000	175,000
TKN (lbs/d)	20,000	28,000
TP (lbs/d)	2,700	3,200

Septage, Holding Tank, Landfill Leachate and Other Wastes

Table 3-6 presents daily average loadings of septage, holding tank, grease trap and settling basin wastes at a given year as percentage of the raw wastewater average daily loadings of that year.

Table 3-6. Septage, Holding Tank, Grease Trap and Settling Basin Waste Loadings

Year	BOD_5 Daily Average Loading (%)	TSS Daily Average Loading (%)	TKN Daily Average Loading (%)	TP Daily Average Loading (%)
1996	-	1.77	-	-
1997	0.56	1.72	0.38	0.44
1998	0.65	2.39	0.78	0.73
1999	0.68	2.97	0.62	0.76
2000	0.81	2.09	0.50	0.57
2001	0.63	2.47	0.59	0.68
2002	0.51	1.53	0.51	0.54
2003	0.76	1.73	0.56	0.62
2004	1.15	2.26	0.63	0.78
2005	0.75	1.88	0.54	0.70
2006	0.84	2.41	0.66	0.65
2007	0.92	2.03	0.69	0.80
Average	0.74	2.10	0.57	0.64

Table 3-7 presents daily average loadings of the landfill leachate and other wastes at a given year as percentage of the raw wastewater average daily loadings of that year. The combined hauled-in wastes account for approximately 1.52% of raw wastewater BOD_5 loadings; 2.29% of TSS loadings; 0.77% of TKN loadings, and 0.83% of TP loadings. The summary of the projected combined hauled-in waste loadings for the planning period are presented in Table 3-8.

Table 3-7. Leachate and Other Wastes Loadings

Year	BOD_5 Loading (%)	TSS Loading (%)	TKN Loading (%)	TP Loading (%)
2005	0.79	0.18	0.21	0.19
2006	0.84	0.20	0.21	0.18
2007	0.73	0.18	0.19	0.19
Average	0.78	0.19	0.20	0.19

Table 3-8. Total Hauled-in Waste Loading Projections

Parameter	Year 2030	Year 2060
BOD_5 (lbs/d)	1,800	2,600
TSS (lbs/d)	2,600	3,700
TKN (lbs/d)	150	220
TP (lbs/d)	22	27

Table 3-9 presents the projected loadings to the plant combining both raw wastewater and hauled-in wastes. These projected flows and loadings are used in developing projected internal loadings for each unit process at the NSWWTP.

Table 3-9. Total Plant Loading Projection

Parameter	Year 2030	Year 2060
BOD_5 (lbs/d)	122,000	173,000
TSS (lbs/d)	127,000	179,000
TKN (lbs/d)	20,000	28,000
TP (lbs/d)	2,700	3,200

4. Internal Flows and Loadings

A mass balance spreadsheet developed in the Tenth Addition facilities planning was used to determine internal loadings to each unit process at the plant. The spreadsheet is configured to follow both the liquid and solids treatment trains at the plant. According to the Tenth Addition documents, the following assumptions were made in developing the mass balance spreadsheet:

- No new facilities have been constructed to handle future loads.
- Biosolids thickening performance would not be degraded with overloading of the existing facilities.

Table 4-1 and 4-2 list the results from this analysis, including internal loadings under the projected loadings in year 2020, 2030 and 2060. The results of internal flows and loadings were then compared with the rated plant unit process capacities determined in Technical Memorandum 1 – Review of Existing Treatment Facilities. The comparison results were presented in Table 4-3 and Figure 4-1 through 4-5.

Table 4-3. Unit Process Rated Capacity and Projected Utilizations

Parameter	Unit	Rated Max Capacity	Year 2020		Year 2030		Year 2060	
			Flow & Loading	Capacity Utilization	Flow & Loading	Capacity Utilization	Flow & Loading	Capacity Utilization
Influent Flowmeter Max Hour Flow	MGD	163.3	129	79%	135	83%	173	106%
Fine Screening System Max Hour Flow	MGD	180	129	72%	135	75%	173	96%
Grit System Max Hour Flow	MGD	180	129	72%	135	75%	173	96%
Primary Tank Max Day Flow	MGD	102	66.5	66%	71	70%	84	83%
Primary Tank Max Hour Flow	MGD	102	129	126%	135	132%	173	170%
Aeration Basins Average Loading	lb O2/d	107,744	99,924	93%	117,002	109%	164,295	152%
Aeration Basins Max Day Loading	lb O2/d	154,604	153,711	99%	180,306	117%	250,329	162%
Aeration Blower Average Day Loading	scfm	88,000	46,425	53%	54,356	62%	76,327	87%
Aeration Blower Max Day Loading	scfm	88,000	71,414	81%	83,765	95%	116,296	132%
Secondary Clarifier Solids Loading	lbs/d	5,537,560	3,544,193	64%	4,321,631	78%	8,307,678	150%
Secondary Clarifier Max Day Hydraulic Loading	MGD	190	82.5	43%	87.9	46%	104.6	55%
Gravity Thickener Max Day Solids Loading	lbs/d	118,800	162,966	137%	185,438	156%	289,334	244%
Gravity Thickener Max Day Hydraulic Loading	gpm	1,980	900	45%	1,000	51%	1,400	71%
Dissolved Air Flotation Thickener Max Day Solids Loading	lbs/d	83,160	107,716	130%	123,595	149%	179,539	216%
Dissolved Air Flotation Thickener Max Day Hydraulic Loading	gpm	3,089	1,700	55%	1,800	58%	2,000	65%
Anaerobic Digester Max Month Solids Loading	lbs/d	153,000	153,746	100%	175,806	115%	264,160	173%
Anaerobic Digester Max Month Hydraulic Loading	gpd	389,000	386,461	99%	442,324	114%	661,333	170%
Gravity Belt Thickener Max Week Solids Loading	lbs/h	6,600	4,072	62%	4,994	76%	9,420	143%
Gravity Belt Thickener Max Week Hydraulic Loading	gpm	500	303	61%	346	69%	519	104%
Metrogro Biosolids Storage Tank Loading	gallon/180 d	19,403,232	25,006,320	129%	29,704,140	153%	54,649,440	282%

Table 4-1
Internal Liquid Process Mass Balance

2020	Avg Day Conc mg/l	Avg Day MASS ppd	Max Month Conc mg/l	Max Month MASS ppd	Max Week Conc mg/l	Max Week MASS ppd	Max Day Conc mg/l	Max Day MASS ppd
Raw Wastewater								
Flow Rate, mgd	46.90		58.63		63.32		65.66	
BOD	267	104,556	257	125,467	257	135,923	325	177,745
TSS	259	101,459	15,996	121,751	18,578	142,043	30,155	233,356
VSS	220	86,240	13,347	103,488	15,527	120,736	25,372	198,352
FSS	39	15,219	37	18,263	40	21,306	64	35,003
TKN	42.6	16,650	40.9	19,980	39.4	20,813	42.6	23,310
NH3-N	26.4	10,323	25.3	12,388	24.4	12,904	26.4	14,452.2
Organic N	16	6,327	16	7,592	15	7,909	16	8,858
TP	6.4	2,500	5.6	2,750	5.7	3,000	5.9	3,250
Particulate P - Organic	2.6	1,000	2.2	1,100.0	2.3	1,200.0	2.4	1,300.0
Particulate P - Chemical	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Soluble P	3.8	1,500	3.4	1,650	3.4	1,800	3.6	1,950
Septage								
Flow Rate, mgd	0.051		0.051		0.051		0.051	
BOD	3,527	1,500	3,527	1,500	3,527	1,500	3,527	1,500
TSS	4,702	2,000	4,702	2,000	4,702	2,000	4,702	2,000
VSS	3,762	1,600	3,762	1,600	3,762	1,600	3,762	1,600
FSS	940	400	940	400	940	400	940	400
TKN	306	130	306	130	306	130	306	130
NH3-N	244.5	104.0	244.5	104.0	244.5	104.0	244.5	104.0
Organic N	61.1	26.0	61.1	26.0	61.1	26.0	61.1	26.0
TP	49	21	49	21	49	21	49	21
Particulate P - Organic	9.9	4.2	9.9	4.2	9.9	4.2	9.9	4.2
Particulate P - Chemical	0	0	0	0	0	0	0	0
Soluble P	39	17	39	17	39	17	39	17
Primary Clarifier Influent								
Flow Rate, mgd	47.96		59.65		64.31		66.47	
BOD	275	109,830	264	131,195	265	141,931	333	184,856
TSS	278	111,342	267	132,998	288	154,465	452	250,716
VSS	232	92,641	223	110,968	241	129,100	380	210,948
FSS	47	18,701	44	22,030	47	25,365	72	39,768
TKN	47	18,826	45	22,438	43	23,311	47	25,866
NH3-N	30	12,034	29	14,298	28	14,816	29	16,273
Organic N	17	6,792	16	8,141	16	8,494	17	9,593
TP	7.4	2,945	6.3	3,142	6.4	3,407	6.5	3,624
Particulate P - Organic	2.9	1,172	2.5	1,224	2.5	1,335	2.6	1,445
Particulate P - Chemical	0.6	229	0.5	228	0.4	234	0.4	197
Soluble P	3.9	1,544	3.4	1,690	3.4	1,839	3.6	1,983
Primary Clarifier Effluent								
Flow Rate, mgd	47.32		59.00		63.66		65.83	
BOD	167	65,898	160	78,717	160	85,159	202	110,914
TSS	99	38,970	95	46,549	102	54,063	160	87,751
VSS	82	32,424	79	38,839	85	45,185	134	73,832
FSS	17	6,545	16	7,711	17	8,878	25	13,919
TKN	43	16,897	41	20,166	39	20,953	42	23,213
NH3-N	30	11,871	29	14,142	28	14,667	29	16,114
Organic N	13	5,026	12	6,024	12	6,286	13	7,099
TP	5.2	2041	4.5	2208.9	4.5	2401	4.7	2571
Particulate P - Organic	1.1	434	0.9	453	0.9	494	1.0	534
Particulate P - Chemical	0.2	85	0.2	84	0.2	87	0.1	73
Soluble P	3.9	1523	3.4	1672	3.4	1820	3.6	1963

2030	Avg Day Conc mg/l	Avg Day MASS ppd	Max Month Conc mg/l	Max Month MASS ppd	Max Week Conc mg/l	Max Week MASS ppd	Max Day Conc mg/l	Max Day MASS ppd
Raw Wastewater								
Flow Rate, mgd	50.00		62.50		67.50		70.00	
BOD	288	120,264	277	144,317	278	156,343	350	204,449
TSS	276	115,118	18,216	138,142	21,150	161,165	34,313	264,771
VSS	235	97,850	15,212	117,420	17,693	136,990	28,899	225,056
FSS	41	17,268	40	20,721	43	24,175	68	39,716
TKN	47.0	19,597	45.1	23,516	43.5	24,496	47.0	27,436
NH3-N	29.1	12,150	28.0	14,580	27.0	15,188	29.1	17,010
Organic N	18	7,447	17	8,936	17	9,309	18	10,426
TP	6.8	2,824	6.0	3,106	6.0	3,389	6.3	3,671
Particulate P - Organic	2.7	1,130	2.4	1,243	2.4	1,356	2.5	1,468
Particulate P - Chemical	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Soluble P	4.1	1,694	3.6	1,864	3.6	2,033	3.8	2,203
Septage								
Flow Rate, mgd	0.054		0.054		0.054		0.054	
BOD	4,059	1,828	4,059	1,828	4,059	1,828	4,059	1,828
TSS	5,853	2,636	5,853	2,636	5,853	2,636	5,853	2,636
VSS	4,682	2,109	4,682	2,109	4,682	2,109	4,682	2,109
FSS	1,171	527	1,171	527	1,171	527	1,171	527
TKN	335	151	335	151	335	151	335	151
NH3-N	268.2	120.8	268.2	120.8	268.2	120.8	268.2	120.8
Organic N	67.1	30.2	67.1	30.2	67.1	30.2	67.1	30.2
TP	51	23	51	23	51	23	51	23
Particulate P - Organic	10.2	4.6	10.2	4.6	10.2	4.6	10.2	4.6
Particulate P - Chemical	0	0	0	0	0	0	0	0
Soluble P	41	18	41	18	41	18	41	18
Primary Clarifier Influent								
Flow Rate, mgd	51.04		63.49		68.45		70.73	
BOD	297	126,316	285	150,859	286	163,190	360	212,435
TSS	298	126,766	286	151,456	308	175,845	484	285,289
VSS	248	105,559	239	126,481	258	147,104	407	240,274
FSS	50	21,207	47	24,974	50	28,741	76	45,014
TKN	52	22,057	50	26,236	48	27,244	51	30,137
NH3-N	33	14,072	31	16,666	30	17,259	32	18,863
Organic N	19	7,984	18	9,570	17	9,985	19	11,275
TP	7.6	3,238	6.7	3,530	6.7	3,828	6.9	4,061
Particulate P - Organic	3.0	1,259	2.6	1,383	2.6	1,507	2.8	1,631
Particulate P - Chemical	0.6	240	0.5	243	0.4	248	0.3	196
Soluble P	4.1	1,739	3.6	1,904	3.6	2,072	3.8	2,235
Primary Clarifier Effluent								
Flow Rate, mgd	50.39		62.84		67.80		70.08	
BOD	180	75,790	173	90,515	173	97,914	218	127,461
TSS	106	44,368	101	53,009	109	61,546	171	99,851
VSS	88	36,946	84	44,268	91	51,486	144	84,096
FSS	18	7,422	17	8,741	18	10,059	27	15,755
TKN	47	19,802	45	23,577	43	24,485	46	27,033
NH3-N	33	13,894	31	16,496	30	17,096	32	18,690
Organic N	14	5,908	14	7,082	13	7,389	14	8,343
TP	5.4	2272	4.7	2486.6	4.8	2702	4.9	2890
Particulate P - Organic	1.1	466	1.0	512	1.0	558	1.0	604
Particulate P - Chemical	0.2	89	0.2	90	0.2	92	0.1	72
Soluble P	4.1	1717	3.6	1885	3.6	2053	3.8	2214

2060	Avg Day Conc mg/l	Avg Day MASS ppd	Max Month Conc mg/l	Max Month MASS ppd	Max Week Conc mg/l	Max Week MASS ppd	Max Day Conc mg/l	Max Day MASS ppd
Raw Wastewater								
Flow Rate, mgd	60.00		75.00		81.00		84.00	
BOD	346	173,000	332	207,600	333	224,900	420	294,100
TSS	358	179,000	28,205	214,800	32,782	250,600	53,537	411,700
VSS	304	152,150	23,709	182,580	27,598	213,010	45,387	349,945
FSS	54	26,850	52	32,220	56	37,590	88	61,755
TKN	56.0	28,000	53.7	33,600	51.8	35,000	56.0	39,200
NH3-N	34.7	17,360	33.3	20,832	32.1	21,700	34.7	24,304
Organic N	21	10,640	20	12,768	20	13,300	21	14,896
TP	6.4	3,200	5.6	3,520	5.7	3,840	5.9	4,160
Particulate P - Organic	2.6	1,280	2.3	1,408	2.3	1,536	2.4	1,664
Particulate P - Chemical	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Soluble P	3.8	1,920	3.4	2,112	3.4	2,304	3.6	2,496
Septage								
Flow Rate, mgd	0.064		0.064		0.064		0.064	
BOD	4,871	2,600	4,871	2,600	4,871	2,600	4,871	2,600
TSS	6,932	3,700	6,932	3,700	6,932	3,700	6,932	3,700
VSS	5,546	2,960	5,546	2,960	5,546	2,960	5,546	2,960
FSS	1,386	740	1,386	740	1,386	740	1,386	740
TKN	412	220	412	220	412	220	412	220
NH3-N	329.7	176.0	329.7	176.0	329.7	176.0	329.7	176.0
Organic N	82.4	44.0	82.4	44.0	82.4	44.0	82.4	44.0
TP	51	27	51	27	51	27	51	27
Particulate P - Organic	10.1	5.4	10.1	5.4	10.1	5.4	10.1	5.4
Particulate P - Chemical	0	0	0	0	0	0	0	0
Soluble P	40	22	40	22	40	22	40	22
Primary Clarifier Influent								
Flow Rate, mgd	60.86		75.77		81.68		84.11	
BOD	357	181,154	342	216,384	344	234,022	433	303,740
TSS	386	196,004	371	234,508	400	272,563	635	445,128
VSS	324	164,360	312	197,127	337	229,462	538	377,363
FSS	62	31,643	59	37,381	63	43,101	97	67,766
TKN	61	31,050	58	36,859	56	38,170	58	40,971
NH3-N	39	19,634	37	23,175	35	23,892	35	24,851
Organic N	22	11,416	22	13,684	21	14,279	23	16,120
TP	7.1	3,608	6.2	3,926	6.2	4,247	6.3	4,394
Particulate P - Organic	2.8	1,425	2.5	1,565	2.5	1,706	2.6	1,843
Particulate P - Chemical	0.4	223	0.3	214	0.3	205	0.0	34
Soluble P	3.9	1,959	3.4	2,147	3.4	2,336	3.6	2,517
Primary Clarifier Effluent								
Flow Rate, mgd	60.22		75.12		81.03		83.46	
BOD	216	108,692	207	129,830	208	140,413	262	182,244
TSS	137	68,601	131	82,078	141	95,397	224	155,795
VSS	115	57,526	110	68,994	119	80,312	190	132,077
FSS	22	11,075	21	13,083	22	15,085	34	23,718
TKN	56	27,873	53	33,103	51	34,268	53	36,588
NH3-N	39	19,425	37	22,977	35	23,702	35	24,660
Organic N	17	8,448	16	10,126	16	10,566	17	11,929
TP	5.1	2548	4.4	2786.6	4.5	3025	4.6	3192
Particulate P - Organic	1.1	527	0.9	579	0.9	631	1.0	682
Particulate P - Chemical	0.2	83	0.1	79	0.1	76	0.0	12
Soluble P	3.9	1939	3.4	2128	3.4	2318	3.6	2498

Table 4-1
Internal Liquid Process Mass Balance

	2020							
	Avg Day		Max Month		Max Week		Max Day	
	Conc mg/l	MASS ppd	Conc mg/l	MASS ppd	Conc mg/l	MASS ppd	Conc mg/l	MASS ppd
Aeration Tank Influent								
Flow Rate, mgd	47.32		59.00		63.66		65.83	
BOD	167	65,898	160	78,717	160	85,159	202	110,914
TSS	99	38,970	95	46,549	102	54,063	160	87,751
VSS	82	32,424	79	38,839	85	45,185	134	73,832
FSS	17	6,545	16	7,711	17	8,878	25	13,919
TKN	43	16,897	41	20,166	39	20,953	42	23,213
NH3-N	30	11,871	29	14,142	28	14,667	29	16,114
Organic N	13	5,026	12	6,024	12	6,286	12.9	7,099
TP	5.2	2,041	4.5	2,209	4.5	2,401	4.7	2,571
Particulate P - Organic	1.1	434	0.9	453	0.9	494	1.0	534
Particulate P - Chemical	0.2	85	0.2	84	0.2	87	0.1	73
Soluble P	3.9	1,523	3.4	1,672	3.4	1,820	3.6	1,963
Recycled Activated Sludge								
Flow Rate, mgd	12.00		14.96		16.14		16.69	
TSS	14,637	1,464,865	17,309	2,160,277	18,975	2,554,875	25,455	3,544,193
VSS	11,430	1,143,896	949,904	1,704,404	1,035,784	2,005,090	1,387,868	2,778,146
FSS	3,207	320,969	254,069	455,873	284,006	549,785	382,692	766,047
Plant Effluent								
Flow Rate, mgd	46.81		58.50		63.16		65.32	
BOD	5.0	1,952	5.0	2,440	5.0	2,634	5.0	2,724
TSS	5.0	1,952	5.0	2,440	5.0	2,634	5.0	2,724
VSS	3.9	1,524	3.9	1,925	3.9	2,067	3.9	2,135
FSS	1.1	428	1.1	515	1.1	567	1.1	589
TKN	0.9	366	0.9	459	0.9	492	0.9	499
NH3-N	0.5	195	0.5	244	0.5	263	0.5	272
Organic N	0.4	170	0.4	215	0.4	229	0.4	227
TP	0.4	138	0.3	166	0.3	179	0.3	170
Particulate P - Organic	0.1	58	0.1	66	0.1	71	0.1	59
Particulate P - Chemical	0.0	2.6	0.0	2.7	0.0	2.8	0.0	1.8
Soluble P	0.2	78	0.2	98	0.2	105	0.2	109

	2030							
	Avg Day		Max Month		Max Week		Max Day	
	Conc mg/l	MASS ppd	Conc mg/l	MASS ppd	Conc mg/l	MASS ppd	Conc mg/l	MASS ppd
Aeration Tank Influent								
Flow Rate, mgd	50.39		62.84		67.80		70.08	
BOD	180	75,790	173	90,515	173	97,914	218	127,461
TSS	106	44,368	101	53,009	109	61,546	171	99,851
VSS	88	36,946	84	44,268	91	51,486	144	84,096
FSS	18	7,422	17	8,741	18	10,059	27	15,755
TKN	47	19,802	45	23,577	43	24,485	46	27,033
NH3-N	33	13,894	31	16,496	30	17,096	32	18,690
Organic N	14	5,908	14	7,082	13	7,389	14.3	8,343
TP	5.4	2,272	4.7	2,487	4.8	2,702	4.9	2,890
Particulate P - Organic	1.1	466	1.0	512	1.0	558	1.0	604
Particulate P - Chemical	0.2	89	0.2	90	0.2	92	0.1	72
Soluble P	4.1	1,717	3.6	1,885	3.6	2,053	3.8	2,214
Recycled Activated Sludge								
Flow Rate, mgd	12.78		15.94		17.20		17.77	
TSS	16,766	1,787,070	19,846	2,638,017	21,748	3,118,838	29,156	4,321,631
VSS	914,165	1,400,910	1,092,019	2,086,904	1,190,444	2,454,409	1,594,009	3,396,862
FSS	251,989	386,160	288,382	551,113	322,263	664,429	433,956	924,769
Plant Effluent								
Flow Rate, mgd	49.89		62.34		67.30		69.57	
BOD	5.0	2,080	5.0	2,600	5.0	2,806	5.0	2,901
TSS	5.0	2,080	5.0	2,600	5.0	2,806	5.0	2,901
VSS	3.9	1,631	4.0	2,056	3.9	2,208	3.9	2,280
FSS	1.1	450	1.0	543	1.1	598	1.1	621
TKN	0.9	391	0.9	490	0.9	525	0.0	0
NH3-N	0.5	208	0.5	260	0.5	281	0.0	0
Organic N	0.4	183	0.4	230	0.4	245	0.0	0
TP	0.4	146	0.3	176	0.3	189	0.3	180
Particulate P - Organic	0.1	60	0.1	69	0.1	74	0.1	62
Particulate P - Chemical	0.0	2.5	0.0	2.7	0.0	2.7	0.0	1.7
Soluble P	0.2	83	0.2	104	0.2	112	0.2	116

	2060							
	Avg Day		Max Month		Max Week		Max Day	
	Conc mg/l	MASS ppd	Conc mg/l	MASS ppd	Conc mg/l	MASS ppd	Conc mg/l	MASS ppd
Aeration Tank Influent								
Flow Rate, mgd	60.22		75.12		81.03		83.46	
BOD	216	108,692	207	129,830	208	140,413	262	182,244
TSS	137	68,601	131	82,078	141	95,397	224	155,795
VSS	115	57,526	110	68,994	119	80,312	190	132,077
FSS	22	11,075	21	13,083	22	15,085	34	23,718
TKN	56	27,873	53	33,103	51	34,268	53	36,588
NH3-N	39	19,425	37	22,977	35	23,702	35	24,660
Organic N	17	8,448	16	10,126	16	10,566	17.1	11,929
TP	5.1	2,548	4.4	2,787	4.5	3,025	4.6	3,192
Particulate P - Organic	1.1	527	0.9	579	0.9	631	1.0	682
Particulate P - Chemical	0.2	83	0.1	79	0.1	76	0.0	12
Soluble P	3.9	1,939	3.4	2,128	3.4	2,318	3.6	2,498
Recycled Activated Sludge								
Flow Rate, mgd	15.27		19.05		20.55		21.17	
TSS	26,869	3,422,165	31,871	5,063,859	34,955	5,990,965	47,060	8,307,678
VSS	1,492,989	2,733,818	1,783,765	4,074,723	1,945,541	4,794,023	2,604,585	6,610,503
FSS	375,919	688,347	433,008	989,136	485,751	1,196,942	668,699	1,697,175
Plant Effluent								
Flow Rate, mgd	59.76		74.67		80.58		83.01	
BOD	5.0	2,492	5.0	3,114	5.0	3,360	5.0	3,461
TSS	5.0	2,492	5.0	3,114	5.0	3,360	5.0	3,461
VSS	4.0	1,991	4.0	2,505	4.0	2,689	4.0	2,754
FSS	1.0	501	1.0	608	1.0	671	1.0	707
TKN	0.9	470	0.9	589	0.9	631	0.9	635
NH3-N	0.5	249	0.5	311	0.5	336	0.5	346
Organic N	0.4	221	0.4	278	0.4	295	0.4	289
TP	0.3	158	0.3	191	0.3	206	0.3	196
Particulate P - Organic	0.1	56	0.1	65	0.1	70	0.1	58
Particulate P - Chemical	0.0	2.0	0.0	2.0	0.0	1.9	0.0	0.2
Soluble P	0.2	100	0.2	125	0.2	134	0.2	138

Table 4-2
Internal Solids Process Mass Balance

	2020							
	Avg Day		Max Month		Max Week		Max Day	
	Conc mg/l	MASS ppd	Conc mg/l	MASS ppd	Conc mg/l	MASS ppd	Conc mg/l	MASS ppd
Primary Sludge Production								
Primary Sludge Flow Rate, gpd	648,000		648,000		648,000		648,000	
Primary Sludge TSS	13,392	72,372	15,996	86,449	18,578	100,402	30,155	162,965
Primary Sludge VSS	11,142	60,217	13,347	72,129	15,527	83,915	25,372	137,116
Primary Sludge Soluble NH3-N	30	163	29	155	28	149	29	159
Primary Sludge TKN	357	1,928	420	2,272	436	2,358	491	2,653
Primary Sludge Soluble P	4	21	3	18	3	19	4	19
Primary Sludge Total P	167	904	173	933	186	1,007	195	1,054
Waste Activated Sludge (WAS) Production								
WAS Flow Rate, gpd	504,222		503,315		503,571		507,383	
WAS TSS	14,637	61,551	17,309	72,659	18,975	79,689	25,455	107,716
WAS VSS	11,430	48,065	13,657	57,326	14,891	62,541	19,953	84,434
WAS NH3-N	2.2	9	2.2	9	2.2	9	2.2	9
WAS TKN	1,281	5,385	1,529	6,420	1,649	6,926	2,123	8,984
WAS Soluble P	0.2	0.8	0.2	0.8	0.2	0.8	0.2	0.8
WAS Total P	453	1,903	487	2,043	529	2,222	567	2,401
Gravity Thickening of Primary Sludge								
Number of units in service	2		2		2		2	
Type	Circular		Circular		Circular		Circular	
Diameter, ft	55		55		55		55	
Total Surface Area, sf	4752		4752		4752		4752	
Solids Loading Rate, lb/d/sf	15.2		18.2		21.1		34.3	
Hydraulic Loading Rate, gpd/sf	136		136		136		136	
Gravity Thickened Primary Sludge Production								
TPSD Flow Rate, gpd	154,621		184,695		214,507		348,171	
TPSD TSS	5.5%	70,925	5.5%	84,720	5.5%	98,394	5.5%	159,706
TPSD VSS	45,762	59,012	45,890	70,686	45,968	82,237	46,276	134,374
TPSD Soluble NH3-N	30	39	29	44	28	49	29	85
TPSD TKN	1,372	1,769	1,375	2,118	1,237	2,214	871	2,530
TPSD Total P	675	870	585	902	545	975	353	1,024
TPSD Soluble P	3.9	5.0	3.4	5.2	3.4	6.1	3.6	10.4
Dissolved Air Flotation Thickening of WAS								
Number of units in service	2		2		2		2	
Type	Circular		Circular		Circular		Circular	
Diameter, ft	55		55		55		55	
Total Effective Surface Area, sf	4562		4562		4562		4562	
Recycle Rate Per DAF, gpd	900,000		900,000		900,000		900,000	
Solids Loading Rate, lb/d/sf	13.49		15.93		17.47		23.61	
Hydraulic Loading Rate, gpm/sf	0.35		0.35		0.35		0.35	
Thickened WAS Production								
TWAS Flow Rate, gpd	171,006		201,866		221,398		299,264	
TWAS TSS	41,000	58,474	41,000	69,026	41,000	75,705	41,000	102,330
TWAS VSS	32,016	45,662	32,348	54,460	32,177	59,414	32,138	80,213
TWAS Soluble NH3-N	2.2	3.1	2.2	3.7	2.2	4.1	2.2	5.4
TWAS TKN	3,583	5,110	3,619	6,094	3,561	6,575	3,418	8,531
TWAS Soluble P	4.0	5.7	4.0	6.7	4.0	7.4	4.0	10.0
TWAS TP	1,260	1,797	1,147	1,931	1,138	2,101	911	2,274
Anaerobic Digester Feed Sludge								
Flow Rate, gpd	325,627		386,561		435,905		647,435	
TSS	47,648	129,398	47,689	153,746	47,889	174,099	48,529	262,037
VSS	38,544	104,674	38,818	125,146	38,964	141,651	39,741	214,586
FSS	9,104	24,725	8,871	28,599	8,926	32,448	8,788	47,450
Org-N	2,518	6,838	2,532	8,164	2,403	8,735	2,032	10,970
NH3-N	15.4	41.9	14.9	48.0	14.7	53.5	16.8	90.7
TKN	2,533	6,880	2,547	8,212	2,417	8,789	2,048	11,061
Particulate P-Organic	897	2,437	807	2,602	781	2,838	572	3,088
Particulate P-Chemical	81	220	68	218	62	224	35	189
Soluble P	4	11	4	12	4	14	4	20
TP	982	2,667	879	2,832	846	3,076	611	3,298
Anaerobic Digester Loading and Performance								
Total Digester Volume, cu ft	845,053		845,053		845,053		845,053	
Total Digester Volume, gals	6,321,000		6,321,000		6,321,000		6,321,000	
Nominal Hydraulic Retention T	19		16		15		10	
Nominal VSS Loading Rate, pp	0.12		0.15		0.17		0.25	
VSS Reduction, %	62.0%		58.2%		53.9%		37.4%	

	2030							
	Avg Day		Max Month		Max Week		Max Day	
	Conc mg/l	MASS ppd	Conc mg/l	MASS ppd	Conc mg/l	MASS ppd	Conc mg/l	MASS ppd
Primary Sludge Production								
Primary Sludge Flow Rate, gpd	648,000		648,000		648,000		648,000	
Primary Sludge TSS	15,247	82,398	18,216	98,446	21,150	114,299	34,313	185,438
Primary Sludge VSS	12,696	68,613	15,212	82,213	17,693	95,618	28,899	156,178
Primary Sludge Soluble NH3-N	33	179	31	170	30	163	32	173
Primary Sludge TKN	417	2,255	492	2,658	511	2,760	574	3,104
Primary Sludge Soluble P	4	22	4	19	4	20	4	20
Primary Sludge Total P	179	966	193	1,044	208	1,125	217	1,171
Waste Activated Sludge (WAS) Production								
WAS Flow Rate, gpd	505,335		504,508		504,742		508,283	
WAS TSS	16,766	70,659	19,846	83,504	21,748	91,550	29,156	123,595
WAS VSS	13,143	55,391	15,700	66,059	17,115	72,046	22,917	97,147
WAS NH3-N	2.2	9	2.2	9	2.2	9	2.2	9
WAS TKN	1,475	6,217	1,761	7,410	1,899	7,993	2,444	10,358
WAS Soluble P	0.2	0.8	0.2	0.8	0.2	0.8	0.2	0.8
WAS Total P	504	2,126	549	2,311	597	2,513	639	2,710
Gravity Thickening of Primary Sludge								
Number of units in service	2		2		2		2	
Type	Circular		Circular		Circular		Circular	
Diameter, ft	55		55		55		55	
Total Surface Area, sf	4752		4752		4752		4752	
Solids Loading Rate, lb/d/sf	17.3		20.7		24.1		39.0	
Hydraulic Loading Rate, gpd/sf	136		136		136		136	
Gravity Thickened Primary Sludge Production								
TPSD Flow Rate, gpd	176,041		210,328		244,197		396,183	
TPSD TSS	5.5%	80,750	5.5%	96,477	5.5%	112,013	5.5%	181,729
TPSD VSS	45,799	67,241	45,931	80,569	46,011	93,705	46,322	153,055
TPSD Soluble NH3-N	33	49	31	55	30	62	32	106
TPSD TKN	1,419	2,083	1,422	2,494	1,279	2,606	901	2,978
TPSD Total P	634	931	576	1,010	536	1,091	345	1,140
TPSD Soluble P	4.1	6.0	3.6	6.3	3.6	7.4	3.8	12.5
Dissolved Air Flotation Thickening of WAS								
Number of units in service	2		2		2		2	
Type	Circular		Circular		Circular		Circular	
Diameter, ft	55		55		55		55	
Total Effective Surface Area, sf	4562		4562		4562		4562	
Recycle Rate Per DAF, gpd	900,000		900,000		900,000		900,000	
Solids Loading Rate, lb/d/sf	15.49		18.31		20.07		27.09	
Hydraulic Loading Rate, gpm/sf	0.35		0.35		0.35		0.35	
Thickened WAS Production								
TWAS Flow Rate, gpd	196,310		231,996		254,350		343,379	
TWAS TSS	41,000	67,126	41,000	79,329	41,000	86,972	41,000	117,415
TWAS VSS	32,140	52,621	32,435	62,756	32,265	68,444	32,227	92,290
TWAS Soluble NH3-N	2.2	3.6	2.2	4.2	2.2	4.7	2.2	6.2
TWAS TKN	3,604	5,901	3,636	7,035	3,578	7,589	3,435	9,838
TWAS Soluble P	4.0	6.5	4.0	7.7	4.0	8.5	4.0	11.5
TWAS TP	1,227	2,009	1,130	2,186	1,121	2,379	897	2,569
Anaerobic Digester Feed Sludge								
Flow Rate, gpd	372,351		442,324		498,547		739,562	
TSS	47,619	147,876	47,657	175,806	47,857	198,986	48,500	299,144
VSS	38,598	119,862	38,852	143,325	38,998	162,149	39,777	245,345
FSS	9,021	28,014	8,805	32,481	8,859	36,836	8,722	53,799
Org-N	2,554	7,931	2,567	9,469	2,436	10,129	2,060	12,704
NH3-N	16.8	52.1	16.1	59.5	15.9	66.2	18.1	111.9
TKN	2,571	7,984	2,583	9,529	2,452	10,195	2,078	12,816
Particulate P-Organic	869	2,698	799	2,949	774	3,217	567	3,498
Particulate P-Chemical	74	230	63	233	57	238	30	188
Soluble P	4	13	4	14	4	16	4	24
TP	947	2,940	866	3,196	835	3,470	601	3,710
Anaerobic Digester Loading and Performance								
Total Digester Volume, cu ft	845,053		845,053		845,053		845,053	
Total Digester Volume, gals	6,321,000		6,321,000		6,321,000		6,321,000	
Nominal Hydraulic Retention T	17		14		13		9	
Nominal VSS Loading Rate, pp	0.14		0.17		0.19		0.29	
VSS Reduction, %	59.4%		53.4%		48.8%		30.3%	

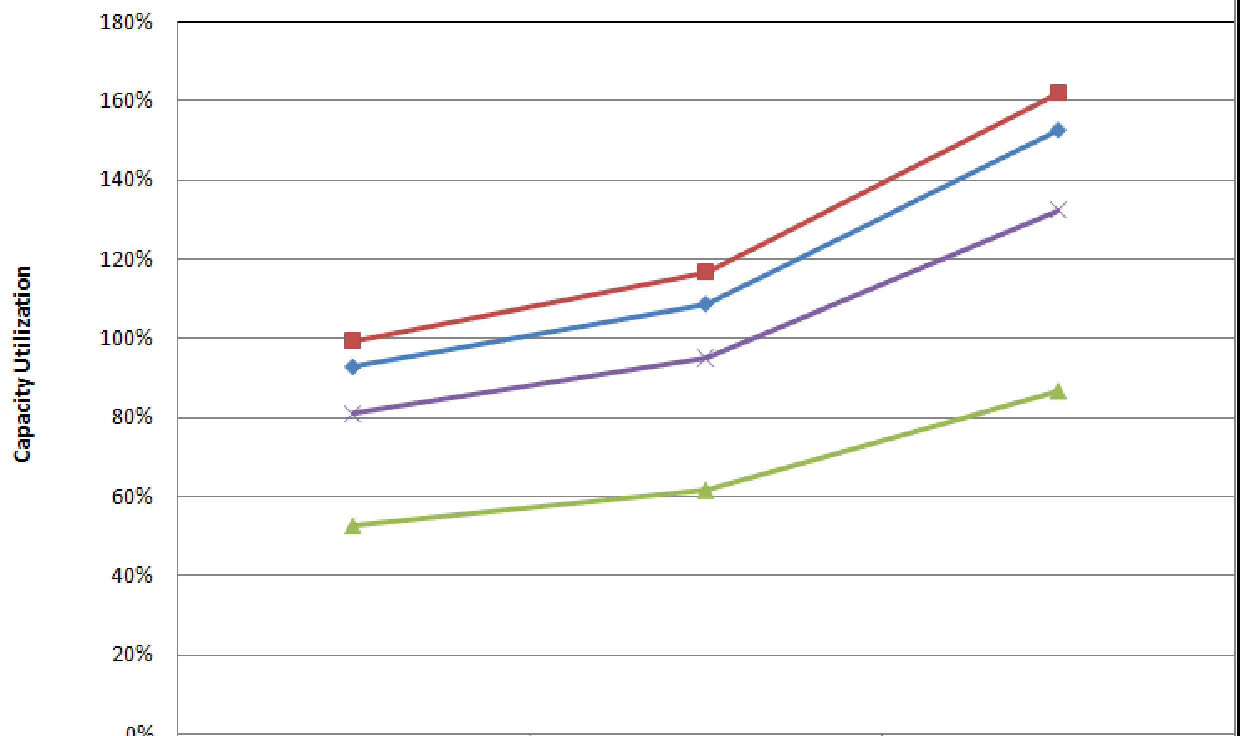
	2060							
	Avg Day		Max Month		Max Week		Max Day	
	Conc mg/l	MASS ppd	Conc mg/l	MASS ppd	Conc mg/l	MASS ppd	Conc mg/l	MASS ppd
Primary Sludge Production								
Primary Sludge Flow Rate, gpd	648,000		648,000		648,000		648,000	
Primary Sludge TSS	23,574	127,402	28,205	152,430	32,782	177,166	53,537	289,333
Primary Sludge VSS	19,768	106,834	23,709	128,133	27,598	149,150	45,387	245,286
Primary Sludge Soluble NH3-N	39	209	37	198	35	190	35	191
Primary Sludge TKN	588	3,177	695	3,756	722	3,902	811	4,383
Primary Sludge Soluble P	4	21	3	18	3	19	4	19
Primary Sludge Total P	196	1,059	211	1,139	226	1,222	222	1,202
Waste Activated Sludge (WAS) Production								
WAS Flow Rate, gpd	455,145		454,552		454,740		457,446	
WAS TSS	26,869	101,993	31,871	120,820	34,955	132,567	47,060	179,539
WAS VSS	21,465	81,478	25,645	97,220	27,971	106,081	37,446	142,861
WAS NH3-N	2.4	9	2.4	9	2.4	9	2.4	9
WAS TKN	2,385	9,052	2,847	10,794	3,069	11,639	3,932	15,003
WAS Soluble P	0.2	0.8	0.2	0.8	0.2	0.8	0.2	0.8
WAS Total P	630	2,390	685	2,595	743	2,819	785	2,996
Gravity Thickening of Primary Sludge								
Number of units in service	2		2		2		2	
Type	Circular		Circular		Circular		Circular	
Diameter, ft	55		55		55		55	
Total Surface Area, sf	4752		4752		4752		4752	
Solids Loading Rate, lb/d/sf	26.8		32.1		37.3		60.9	
Hydraulic Loading Rate, gpd/sf	136		136		136		136	
Gravity Thickened Primary Sludge Production								
TPSD Flow Rate, gpd	272,191		325,663		378,510		618,153	
TPSD TSS	5.5%	124,854	5.5%	149,382	5.5%	173,622	5.5%	283,547
TPSD VSS	46,121	104,697	46,233	125,570	46,303	146,167	46,627	240,380
TPSD Soluble NH3-N	39	88	37	100	35	111	35	183
TPSD TKN	1,320	2,997	1,320	3,586	1,188	3,749	832	4,290
TPSD Total P	452	1,027	408	1,108	377	1,190	228	1,177
TPSD Soluble P	3.9	8.8	3.4	9.2	3.4	10.8	3.6	18.5
Dissolved Air Flotation Thickening of WAS								
Number of units in service	2		2		2		2	
Type	Circular		Circular		Circular		Circular	
Diameter, ft	55		55		55		55	
Total Effective Surface Area, sf	4562		4562		4562		4562	
Recycle Rate Per DAF, gpd	900,000		900,000		900,000		900,000	
Solids Loading Rate, lb/d/sf	22.36		26.49		29.06		39.36	
Hydraulic Loading Rate, gpm/sf	0.34		0.34		0.34		0.34	
Thickened WAS Production								
TWAS Flow Rate, gpd	283,364		335,670		368,305		498,806	
TWAS TSS	41,000	96,894	41,000	114,779	41,000	125,938	41,000	170,562
TWAS VSS	32,753	77,404	32,991	92,359	32,809	100,777	32,624	135,718
TWAS Soluble NH3-N	2.4	5.8	2.4	6.8	2.4	7.5	2.4	10.1
TWAS TKN	3,637	8,596	3,662	10,253	3,599	11,056	3,426	14,254
TWAS Soluble P	4.0	9.5	4.0	11.2	4.0	12.3	4.0	16.6
TWAS TP	959	2,265	879	2,462	871	2,675	684	2,847
Anaerobic Digester Feed Sludge								
Flow Rate, gpd	555,556		661,333		746,814		1,116,959	
TSS	47,859	221,748	47,894	264,160	48,096	299,561	48,748	454,109
VSS	39,302	182,101	39,512	217,929	39,648	246,944	40,374	376,098
FSS	8,557	39,646	8,382	46,232	8,448	52,617	8,374	78,011
Org-N	2,482	11,499	2,490	13,732	2,358	14,687	1,970	18,351
NH3-N	20.2	93.6	19.3	106.4	19.0	118.2	20.7	192.7
TKN	2,502	11,593	2,509	13,839	2,377	14,805	1,991	18,544
Particulate P-Organic	660	3,059	606	3,343	585	3,646	425	3,957
Particulate P-Chemical	46	214	37	205	32	196	3	32
Soluble P	4	18	4	20	4	23	4	35
TP	710	3,292	647	3,569	621	3,865	432	4,024
Anaerobic Digester Loading and Performance								
Total Digester Volume, cu ft	845,053		845,053		845,053		845,053	
Total Digester Volume, gals	6,321,000		6,321,000		6,321,000		6,321,000	
Nominal Hydraulic Retention T	11		10		8		6	
Nominal VSS Loading Rate, pp	0.22		0.26		0.29		0.45	
VSS Reduction, %	44.4%		36.3%		29.7%		-10.0%	

Table 4-2
Internal Solids Process Mass Balance

	2020								2030								2060							
	Avg Day		Max Month		Max Week		Max Day		Avg Day		Max Month		Max Week		Max Day		Avg Day		Max Month		Max Week		Max Day	
	Conc	MASS	Conc	MASS	Conc	MASS	Conc	MASS	Conc	MASS	Conc	MASS	Conc	MASS	Conc	MASS	Conc	MASS	Conc	MASS	Conc	MASS	Conc	MASS
	mg/l	ppd	mg/l	ppd	mg/l	ppd	mg/l	ppd	mg/l	ppd	mg/l	ppd	mg/l	ppd	mg/l	ppd	mg/l	ppd	mg/l	ppd	mg/l	ppd	mg/l	ppd
Digested Sludge Production																								
Digested Sludge Flow Rate, gp	325,627		386,561		435,905		647,435		372,351		442,324		498,547		739,562		555,556		661,333		746,814		1,116,959	
Digested Sludge TSS w/ Chem	23,751	64,501	25,109	80,950	26,885	97,741	33,676	181,838	24,673	76,618	26,922	99,314	28,823	119,843	36,441	224,768	30,423	140,962	33,544	185,010	36,301	226,100	52,778	491,647
Digested Sludge VSS	14,647	39,776	16,238	52,351	17,960	65,292	24,888	134,387	15,652	48,605	18,117	66,832	19,964	83,007	27,719	170,969	21,867	101,315	25,161	138,778	27,853	173,484	44,403	413,636
Soluble NH3-N in Digested Slu	952	2,587	958	3,088	909	3,305	770	4,159	967	3,002	971	3,583	922	3,833	781	4,819	941	4,359	943	5,203	894	5,567	748	6,972
Digested Sludge TKN	2,533	6,880	2,547	8,212	2,417	8,789	2,048	11,061	2,571	7,984	2,583	9,529	2,452	10,195	2,078	12,816	2,502	11,593	2,509	13,839	2,377	14,805	1,991	18,544
Digested Sludge Soluble P	139	376	125	402	121	439	90	484	134	417	124	456	120	498	89	549	103	477	95	522	92	570	67	629
Digested Sludge TP	982	2,667	879	2,832	846	3,076	611	3,298	947	2,940	866	3,196	835	3,470	601	3,710	710	3,292	647	3,569	621	3,865	432	4,024
Digester Gas Production																								
Digester Gas Produced, scf/day	895,590		1,004,575		1,053,748		1,106,745		983,355		1,055,594		1,092,164		1,026,383		1,114,848		1,092,279		1,013,751		-518,031	
Digester Gas Energy Produced,	4.93E+08		5.53E+08		5.80E+08		6.09E+08		5.41E+08		5.81E+08		6.01E+08		5.65E+08		6.13E+08		6.01E+08		5.58E+08		-2.85E+08	
Gravity Belt Thickening of Digested Sludge																								
Number of Units	1		1		1		1		1		1		1		1		1		1		1		1	
Width, meters	2		2		2		2		2		2		2		2		2		2		2		2	
Digested Sludge Flowrate, gpd	325,627		386,561		435,905		647,435		372,351		442,324		498,547		739,562		555,556		661,333		746,814		1,116,959	
Hydraulic Loading Rate, gpm/r	113		134		151		225		129		154		173		257		193		230		259		388	
Solids Loading Rate, lb/hr/m	1,344		1,686		2,036		3,788		1,596		2,069		2,497		4,683		2,937		3,854		4,710		10,243	
Thickened Digested Sludge Production																								
Cake Flow Rate, gpd	138,924		174,354		210,517		391,648		165,023		213,905		258,123		484,113		303,608		398,480		486,982		1,058,925	
Cake TSS		62,566		78,522		94,808		176,382		74,320		96,334		116,248		218,025		136,733		179,460		219,317		476,898
Cake VSS	33,301	38,583	34,922	50,780	36,073	63,333	39,909	130,356	34,256	47,147	36,339	64,827	37,402	80,517	41,075	165,840	38,812	98,276	40,506	134,615	41,433	168,279	45,432	401,227
Cake Soluble NH3-N		1,104		1,393		1,596		2,516		1,330		1,733		1,985		3,154		2,382		3,135		3,630		6,610
Cake TKN	4,546	5,268	4,376	6,363	3,939	6,915	2,820	9,211	4,478	6,163	4,204	7,500	3,788	8,156	2,703	10,912	3,712	9,399	3,464	11,512	3,100	12,591	2,019	17,834
Cake Soluble P	139	160	125	181	121	212	90	293	134	185	124	221	120	258	89	359	103	261	95	314	92	372	67	596
Cake TP	2056	2,383	1746	2,539	1578	2,770	925	3,023	1913	2,632	1613	2,878	1459	3,141	848	3,425	1181	2,991	984	3,270	879	3,568	440	3,890
Metrogro Storage Tanks																								
Number of Tanks	3		3		3		3		3		3		3		3		3		3		3		3	
Tank Diameter	160		160		160		160		160		160		160		160		160		160		160		160	
Volume Each Tank	864,672		864,672		864,672		864,672		864,672		864,672		864,672		864,672		864,672		864,672		864,672		864,672	
Total Volume	2,594,015		2,594,015		2,594,015		2,594,015		2,594,015		2,594,015		2,594,015		2,594,015		2,594,015		2,594,015		2,594,015		2,594,015	
Detention Time	140		111		92		50		118		91		75		40		64		49		40		18	

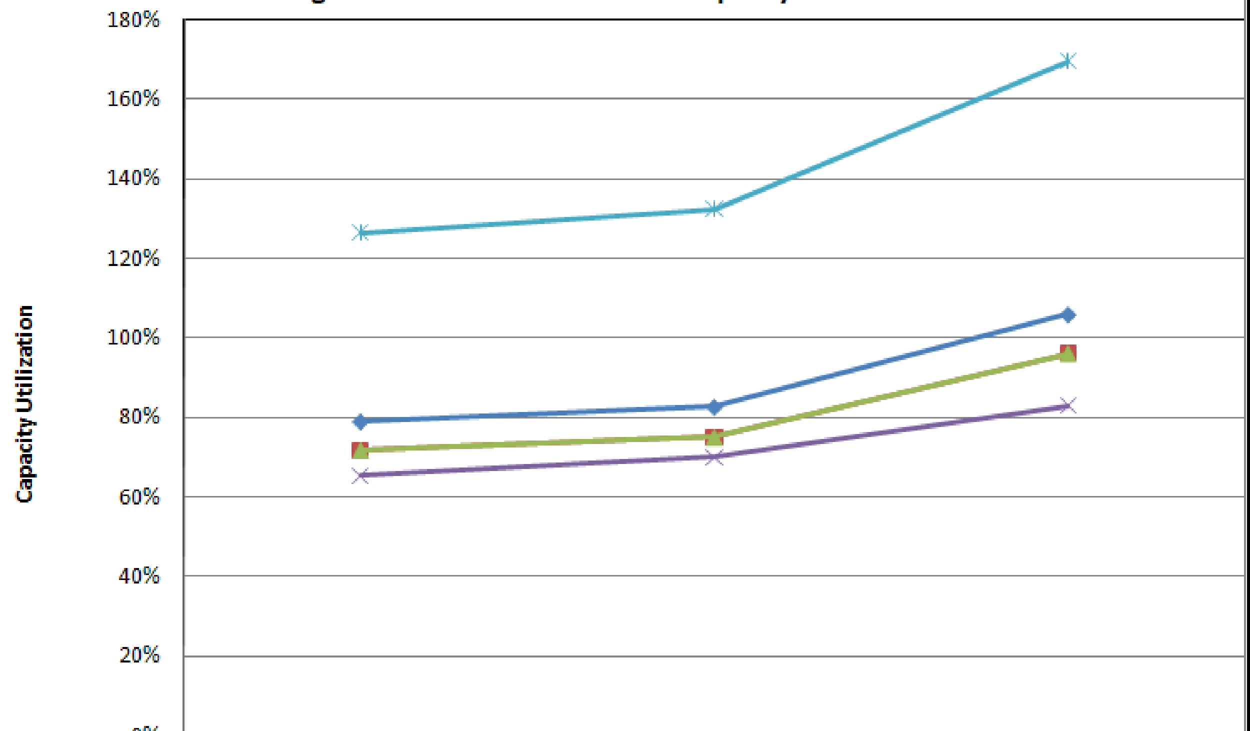
Notes:
1. Maximum month.

Figure 4-2: NSWTP Unit Process Capacity Utilization No.2



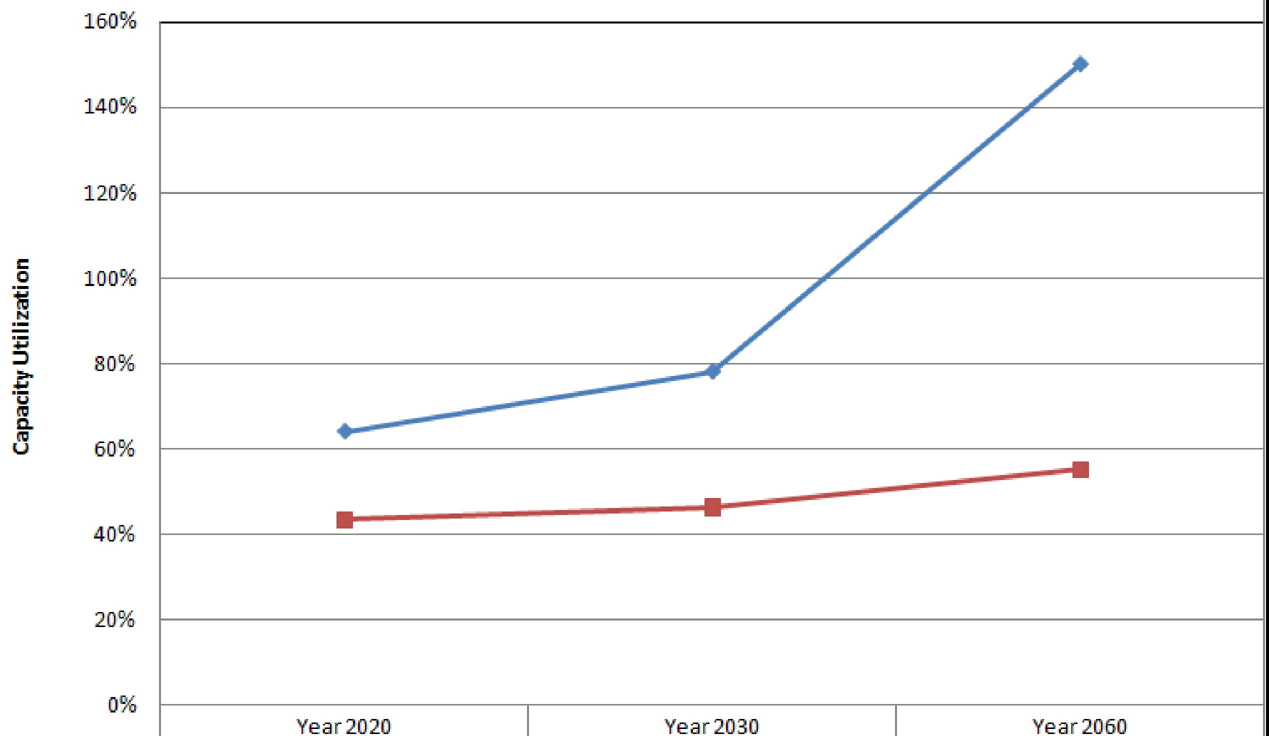
	Year 2020	Year 2030	Year 2060
Aeration Basins Average Loading	93%	109%	152%
Aeration Basins Max Day Loading	99%	117%	162%
Aeration Blower Average Day Loading	53%	62%	87%
Aeration Blower Max Day Loading	81%	95%	132%

Figure 4-1: NSWTP Unit Process Capacity Utilization No.1

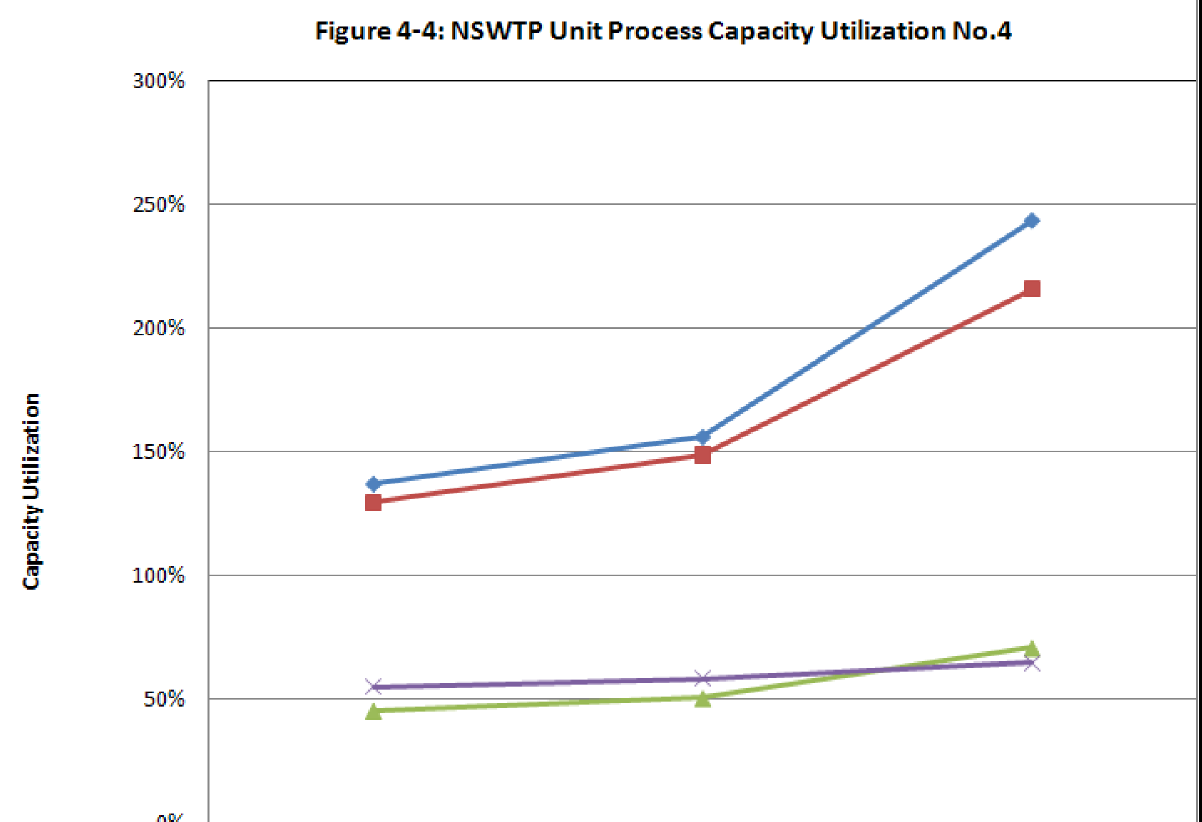


	Year 2020	Year 2030	Year 2060
Influent Flowmeter Max Hour Flow	79%	83%	106%
Fine Screening System Max Hour Flow	72%	75%	96%
Grit System Max Hour Flow	72%	75%	96%
Primary Tank Max Day Flow	66%	70%	83%
Primary Tank Max Hour Flow	126%	132%	170%

Figure 4-3: NSWTP Unit Process Capacity Utilization No.3

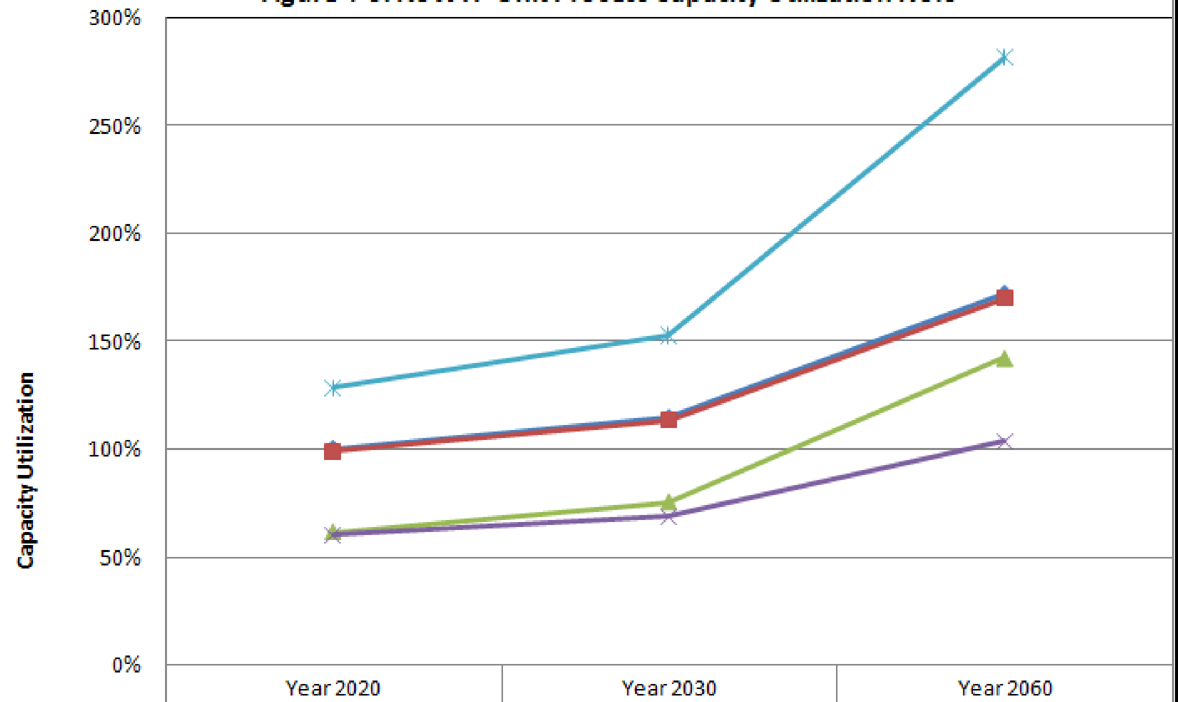


	Year 2020	Year 2030	Year 2060
Secondary Clarifier Solids Loading	64%	78%	150%
Secondary Clarifier Max Day Hydraulic Loading	43%	46%	55%



	Year 2020	Year 2030	Year 2060
Gravity Thickener Max Day Solids Loading	137%	156%	244%
Dissolved Air Flotation Thickener Max Day Solids Loading	130%	149%	216%
Gravity Thickener Max Day Hydraulic Loading	45%	51%	71%
Dissolved Air Flotation Thickener Max Day Hydraulic Loading	55%	58%	65%

Figure 4-5: NSWTP Unit Process Capacity Utilization No.5



	Year 2020	Year 2030	Year 2060
Anaerobic Digester Max Month Solids Loading	100%	115%	173%
Anaerobic Digester Max Month Hydraulic Loading	99%	114%	170%
Gravity Belt Thickener Max Week Solids Loading	62%	76%	143%
Gravity Belt Thickener Max Week Hydraulic Loading	61%	69%	104%
Metrogro Biosolids Storage Tank Loading	129%	153%	282%

Technical Memo for

Madison Metropolitan Sewerage District, Wisconsin

Technical Memo 3

Conveyance Facilities Analysis (CFA)
August 2009

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SECTION 1
INTRODUCTION

1.01 TECHNICAL MEMO OVERVIEW

The Collection Facilities Analysis (CFA) Technical Memo reviews the existing Madison Metropolitan Sewerage District (MMSD) conveyance infrastructure with regard to condition of the infrastructure asset, age of the infrastructure asset, and the ability of the infrastructure assets to meet projected capacity requirements for the years 2020, 2030, and 2060 assuming all wastewater will continue to be treated at the Nine Springs Wastewater Treatment Plant (NSWTP). The analyses presented in this technical memo will provide a baseline for comparison of potential alternatives to treating wastewater at the NSWTP.

This technical memo will provide information that will be used in the Master Plan Financial Model Technical Memo. Specifically it will designate the anticipated resources required and the approximate timing for major infrastructure replacement that will be needed regardless of whether all wastewater will be treated at NSWTP or at other facilities in combination with the NSWTP.

This technical memo will provide information that will serve as the basis for alternative analyses to be presented in the Master Plan Alternative Analysis Technical Memo, including estimated costs for construction of pumping facilities and the ongoing operational costs for pumping stations.

The Development of Flows Technical Memo serves as the basis for all flows presented in this technical memo. Flows presented in this technical memo were based on the Capital Area Regional Planning Commission (CARPC) analysis of population and flows prepared for the MMSD 2008 Collection System Facilities Plans. Any assumptions independent of the flow development in that technical memo will be presented and discussed in this technical memo.

The Impact of Current and Future Regulations Technical Memo will develop the background on the potential regulatory issues associated with implementation of alternative flow routing presented in this technical memo.

This memo includes an evaluation of the following elements and characteristics of the existing conveyance system:

- Overview of the existing conveyance system operation (2010).
- Summary of each pumping station including related interceptors and force mains.
- Analyses of the existing conveyance system with and without satellite wastewater treatment plants.
- Development of projected costs and timing for conveyance system improvements independent of satellite treatment plants.

The technical memo is organized in a linear fashion with each pumping station described in its numeric sequence. There are many complex pumping station interactions in the MMSD system. As an aid to the reader, the most efficient way to read this technical memo would be to review Section 2.01 that provides an overview of the conveyance system and Section 2.02 that describes both routine and alternate operations of the conveyance system. These sections provide an overall look at the conveyance system. Sections 2.03 to 2.19 describe in detail the 17 wastewater pumping stations and the interceptor and force main components associated with each of these stations. Section 2.20

describes in detail the effluent pumping station and the force main components associated with the Badfish Creek effluent discharge and the Badger Mill Creek effluent discharge.

As an aid to understanding the overall operations, we would recommend that the sections be reviewed in the following fashion. An overall system schematic is presented in Figure 1.01-1.

- Section 2.09 PS 7-6300 Metropolitan Lane

  PS 7 is the District's largest station and in many ways its most critical pumping station. This station discharges directly to the NSWTP.

- Section 2.08 PS 6-402 Walter Street (Olbrich Park)

  This is one of three stations that pumps to PS 7. One pumping station, PS 1 (Section 2.03), is piped to discharge to PS 6. Under 2010 operations, a portion of PS 1 flows are discharged to PS 6 while the majority of flows are discharged to PS 2 (Section 2.04).

- Section 2.12 PS 10-110 Regas Road (Main Post Office)

  This is one of three stations that pumps to PS 7. PS 14 and PS 13 also contribute flow to PS 10.

- Section 2.11 PS 9-4612 Larson Beach Road, Mc Farland

  This is one of the three stations that discharges to PS 7. There are no contributory MMSD pumping stations upstream of PS 9.

- Section 2.15 PS 13-3634 Amelia Earhart (Truax Field)

  This station discharges to PS 10 and receives flow from PS 14.

- Section 2.16 PS 14-5000 School Road

  This station discharges to PS 13. There are no contributory MMSD pumping stations upstream of PS 14.

- Section 2.13 PS 11-4760 East Clayton Road

  This station discharges directly to the NSWTP and is the fourth largest pumping station contributing directly to the NSWTP. It received flows from PS 12.

- Section 2.14 PS 12-2739 Fitchrona Road

  This station receives pumped flow from PS 16 and PS 17, and as an alternate operating mode, from PS 15 through PS 16. Flows from this station are pumped to PS 11.

- Section 2.18 PS 16-1301 Gammon Road

  This station ordinarily only receives flows from its gravity drainage service area. Alternatively flows from PS 15 may be pumped to PS 16.

- Section 2.19 PS 17- 407 Bruce Street, Verona

  This station only receives flows from its gravity drainage service area and discharges to PS 12.

- Section 2.10 PS 8- 967 Plaenert Street

  This station discharges directly to the NSWTP. This station receives flow from PS 5 and PS 15 in its normal operating mode.

- Section 2.07 PS 5- 5221 Lake Mendota Drive (Spring Harbor)

  This station receives flow on a routine basis from its gravity drainage service area but may receive flow from either the PS 15 service area or PS 16 service area in alternate operating modes.

- Section 2.17 PS 15- 2115 Allen Boulevard (Marshall Park)

  This station receives flow only from its gravity drainage service area.

- Section 2.04 PS 2- 833 West Washington Ave (Brittingham Park)

  This station receives flow from its gravity drainage service area plus most of the flow from PS 1.

- Section 2.03 PS 1- 104 North First Street

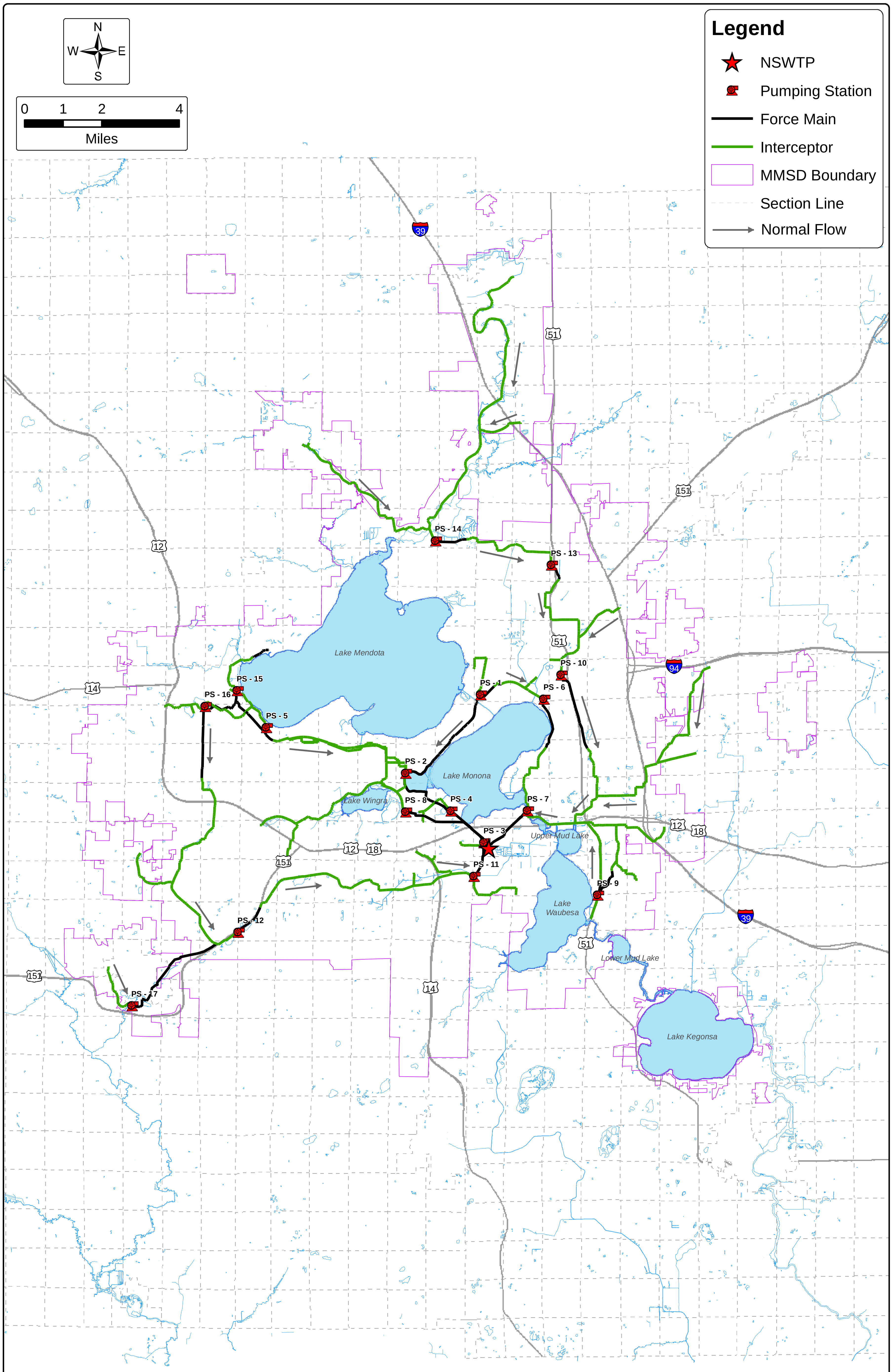
  This station receives flow only from its gravity drainage service area. Wastewater is pumped to PS 2 via the Cross Town Force Main (CTFM) or to PS 6.

- Section 2.06 PS 4- 522 John Nolen Drive

  This station receives flow only from its gravity drainage service area. The station discharges to a force main shared by PS 2, PS 3, and PS 4.

- Section 2.05 PS 3-Nine Springs

  This station receives flow only from its gravity drainage service area. The station discharges to a force main shared by PS 2, PS 3, and PS 4.



EXISTING MMSD COLLECTION SYSTEM
NORMAL AND ALTERNATIVE OPERATING MODES

INTERCEPTOR AND FORCE MAIN ANALYSIS
CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT

STRAND ASSOCIATES, INC.® ENGINEERS

FIGURE 1.01-1
1547.001

S:\MAD\1500--1599\1547\001\Data\GIS\Figures\Figure 1.01-1 MMSD Collection System 11x17.mxd

1.02 ABBREVIATIONS AND DEFINITIONS

A. Abbreviations

The following abbreviations were used in the preparation of this technical memo.

ACP	asbestos cement pipe
ADF	average daily flow
CIP	cast iron pipe
DIPLP	ductile iron polyethylene lined pipe
CTFM	Crosstown Force Main
DHFS	Wisconsin Department of Health and Family Services
DIP	ductile iron pipe
EI	East Interceptor
FEI	Far East Interceptor
FRP	fiberglass reinforced pipe
LBMCI	Lower Badger Mill Creek Interceptor
mgd	million gallons per day
MMSD	Madison Metropolitan Sewerage District
NEI	Northeast Interceptor
NSVI	Nine Springs Valley Interceptor
NSWTP	Nine Springs Wastewater Treatment Plant
OWI	Old West Interceptor
PHF	peak hourly flow
PVCL	PVC lined pipe
PVCP	PVC pipe
RCP	reinforced concrete pipe
RCCP	reinforced concrete cylinder pipe
RCPTP	reinforced concrete pipe with t-lok lining
PS #	Pumping Station #
SEI	Southeast Interceptor
SI	South Interceptor
SWI	Southwest Interceptor
VCPL	vitrified clay pipe with liner
VCP	vitrified clay pipe
WDNR	Wisconsin Department of Natural Resources
WI	West Interceptor
WWTP	wastewater treatment plant

B. Definitions

Madison Design Curve (MDC)–Peak Hourly Flow (PHF) factor developed in the 1961 Greeley and Hansen Report on Sewerage and Sewage Treatment. The formula is applied to average daily flows (ADFs) in the range of 1 mgd to 20 mgd as follows: PHF = $(ADF)^{0.842}$ x 4. All PHFs presented in this technical memo were prepared in this fashion unless specifically noted otherwise. For ADFs less than 1 mgd, the peaking factor is 4. For ADFs greater than 20 mgd, the peaking factor is 2.5.

SECTION 2
EXISTING COLLECTION SYSTEM FACILITY EVALUATION

2.01 OVERVIEW OF MMSD CONVEYANCE SYSTEM

This section provides a general overview of the MMSD conveyance system. The following sections highlight the pumping stations as well as the gravity sewers that feed the pumping stations and the force mains to which the pumping stations discharge.

The MMSD currently operates 18 pumping stations. Seventeen of the pumping stations convey wastewater to the NSWTP while the other pumps treated effluent either to Badfish Creek or Badger Mill Creek. All wastewater treated at the NSWTP is pumped to the treatment plant.

The overall view of the MMSD service area is shown in Figure 2.01-1. Each pumping station location is shown in this figure. The major surface water watersheds located in MMSD (Sugar River, Yahara Lakes, Badfish Creek, and the Koshkonong Creek) are also noted on this figure.

Table 2.01-1 summarizes the current hydraulic capacities of pumping stations. Table 2.01-2 presents the 2008 Pumping Station Condition Assessment and Priority Ranking prepared by MMSD staff. Table 2.01-3 presents a summary of the electrical system characteristics of the pumping stations.

Table 2.01-4 summarizes MMSD's existing force mains. The comparison of the capacities of the force mains to the projected peak flows is based on the flows contained in the *2008 MMSD Collection System Evaluation* prepared by the Capital Area Regional Planning Commission (CARPC). MMSD owns and operates 28.2 miles of force mains for wastewater conveyance. Average daily flows for each of the pumping stations from 1996 through 2006 are summarized in Table 2.01-5.

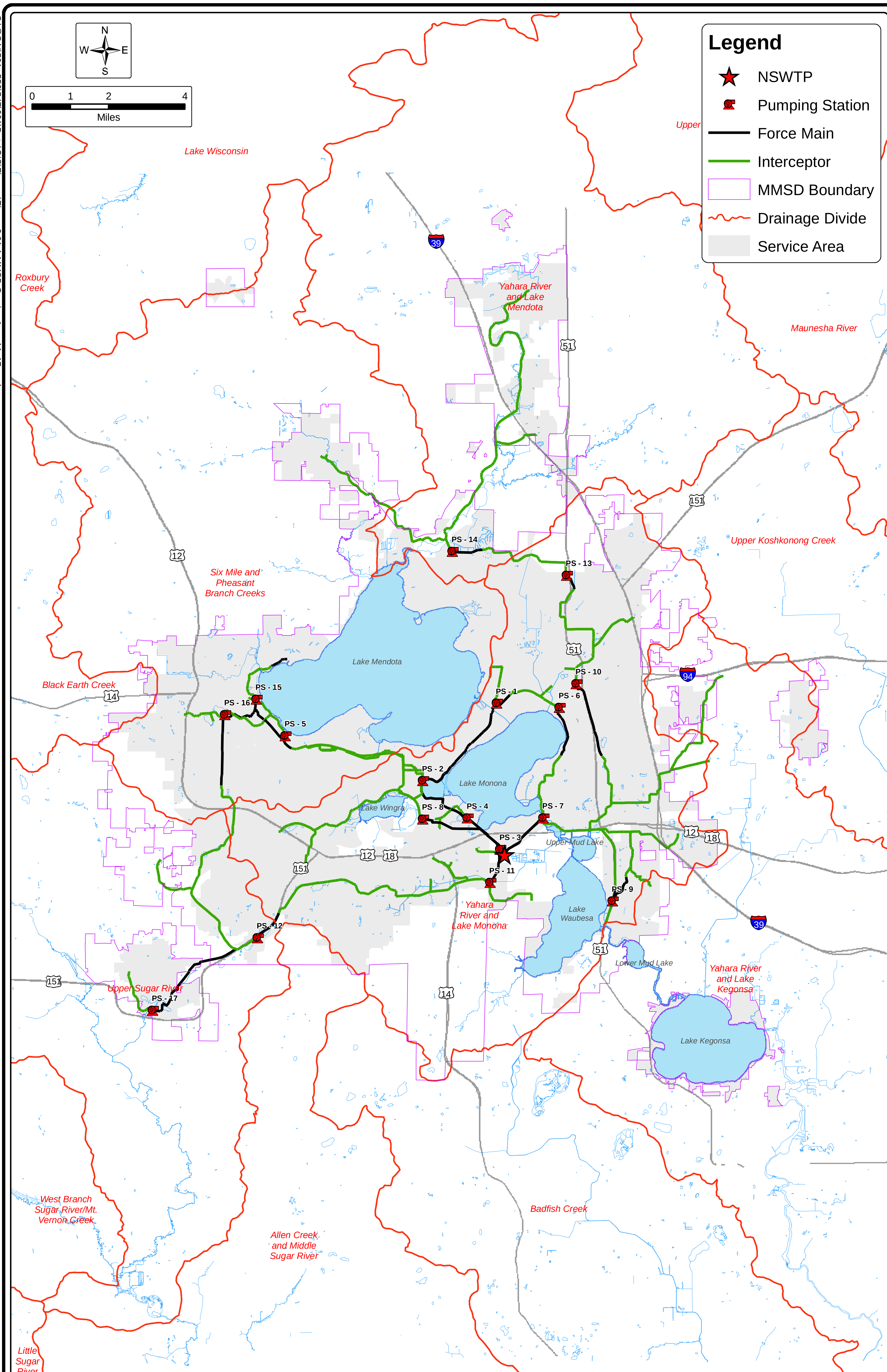
Wastewater is conveyed to the pumping stations through interceptor sewers. MMSD (2008) owns 94 miles of interceptors. Table 2.01-6 provides an overview of the adequacy of the capacity of interceptor sewers. A more detailed review of these is included in Section 3 of this report.

MMSD's 2002 Collection System Facilities Plan and annual reports provide a summary of televising done to review the condition of its existing system. Table 2.01-7 summarizes the past history of MMSD in reviewing system condition.

The general condition (2008) of MMSD's interceptors is very good based on the relatively small list of interceptor segments identified as needing repair. MMSD has already repaired all the items on this list indicated as either needing repair or replacement. MMSD is in the process of modifying its program to target higher priority interceptors for more frequent review.

Contributory customers also have pumping stations within their own collection systems as summarized in Table 2.01-8. Collection systems owned by the contributory customers convey their wastewater to these pumping stations some of which, MMSD maintains on a contract basis with its customers as also noted in the table. The total length of collection sewers connected to MMSD's system was approximately 1,332 miles in 2008. Average daily flows from each of these contributory customers for the years 2000 through 2007 are summarized in Table 2.01-9.

In 2002, MMSD began a recurring and ongoing Facilities Planning effort for the conveyance system. MMSD is in the process of updating the 2002 Collection System Facilities Plan in conjunction with the development of the 50-Year Master Plan. This technical memo uses and summarizes some of the information used to create each of these plans. These plans are done to determine and plan for MMSD's needs in the conveyance system. They also provide the justification required for State of Wisconsin Clean Water Fund loan eligibility. Finally they identify the costs and timing of projects to allow MMSD to identify revenues required in the future so that the impacts of rate increases on the system users can be minimized.



MMSD SERVICE AREA

CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT
CITY OF MADISON, WISCONSIN

STRAND ASSOCIATES, INC.® ENGINEERS

FIGURE 2.01-1
1-547.001

S:\@SAI\501--550\547\001\Data\GIS\Figures\Figure 2.01-1 MMSD Service Area 11x17.mxd

MADISON METROPOLITAN SEWERAGE DISTRICT
50-YEAR MASTER PLAN (2010-2060)

TECHNICAL MEMO 3
CONVEYANCE FACILITIES ANALYSIS

TABLE 2.01-1
EXISTING PUMPING STATION SUMMARY (2010)

Pumping Station	Location	Original Construction	Capacity (mgd) Maximum	Firm	Completed or Proposed Upgrade	Potential Diverted Flow	Existing or Potential Internal Flow Diversion	Gravity Service Area (acres) 2000	2030	2060
1	104 North First Street	1950			2004		PS 13 or PS 14 (Future)	2,824	2,824	2,824
	To Pumping Station 6		18.0	15.0			PS 6			
	To Pumping Station 2		20.3	20.3			PS 2			
2	833 W. Washington Ave (Brittingham Park)	1964	41.0	41.0	2004		PS8, PS 1	923	1,179	1,179
3	Nine Spings	1959	1.5	1.5				514	514	514
4	522 John Nolen Drive	1967	4.2	4.2				1,331	1,331	1,331
5	5221 Lake Mendota Drive (Spring Harbor)	1996	3.6	3.6			PS2	1,101	1,016	1,016
6	402 Walter Street	1950	24.2	24.2	2010		PS 10	2,604	2,784	2,784
7	6300 Metropolitan Lane	1950	45.0	39.0	1992	Stoughton Plant, Sun Prairie Plant, Mendota	PS 18 (Future)	9,265	19,221	26,032
8	967 Plaenert Drive	1964	34.1	34.0	2010		PS 2	8,160	7,904	7,904
9	4612 Larson Beach Road, McFarland	1962	4.5	4.5		Stoughton Plant		2,615	4,955	6,495
10	110 Regas Road	1965	42.2	42.2	2004	Mendota Plant	PS 6	5,374	7,404	7,404
11	4760 East Clayton Road	1966	31.2	25.5	Rehabilitation Scheduled for 2013-2015	Sugar River		7,345	10,014	12,964
12	2739 Fitchrona Road	1969	23.5	16.6	Rehabilitation Scheduled for 2013-2015	Sugar River		4,548	8,253	8,482
13	3634 Amelia Earhart Dri ve	1970	20.2	20.0	Firm Capacity 2008	Mendota Plant		5,041	9,349	9,349
14	5000 School Road	1971	15.6	15.0	Firm Capacity 2008	Mendota Plant		8,202	16,710	21,735
15	2115 Allen Blvd	1975	8.8	5.8			PS 16, PS 5	3,463	6,275	7,194
16	1301 Gammon Road	1982	18.7	18.7				3,647	4,994	5,221
17	407 Bruce Street, Verona	1996	4.6	4.6		Sugar River		1,902	9,166	10,027
								68,859	113,893	132,455

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S:\MAD\1500--1599\1547\001\Spr\Technical Memos\CFA Spreadsheets\CFA Tables Sections 1, 2 and 3 With District Comments (042009).xlsx\Table 2.01-1-PS Capacity\9/30/2009

MADISON METROPOLITAN SEWERAGE DISTRICT
50-YEAR MASTER PLAN (2010-2060)

TECHNICAL MEMO 3
CONVEYANCE FACILITIES ANALYSIS

TABLE 2.01-2
PUMPING STATION CONDITION ASSESSMENT AND PRIOIRTY RANKING (2008)

PS	Adequacy, Condition of Mission Critical Category Maximum Flow Capacity (5 Points)	Firm Flow Capacity (5 Points)	Power System Redundancy (5 Points)	Mechanical Condition (5 Points)	Building and Structural Condition (5 Points)	Electrical Condition (5 Points)	Total	Station Weighting Factor	Overall Rating	Ordinal Ranking
1	1	1	1	1.5	1	1	6.5	1.75	11.38	12
2	1	1	1	1.5	1	1	6.5	2.00	13.00	10
3	2	2	3	2	4	1	14	1.00	14.00	9
4	3	3	3	2	2	3	16	1.10	17.60	6
5	1	1	1	1	1	1	6	1.20	7.20	17
6	1	1	1	1	1	1	6	1.30	7.80	15
7	4	4	2.5	2	1	1.5	15	2.00	30.00	1
8	1	1	1	1	1	1	6	1.90	11.40	11
9	1	1	1	1	2	1	7	1.05	7.35	16
10	1	1	1	1.5	1	1	6.5	1.60	10.40	13
11	3	4	3	2	2	4	18	1.50	27.00	3
12	3	5	4	2	2	3.5	19.5	1.50	29.25	2
13	3	3	4	1	3	3.5	17.5	1.10	19.25	4
14	2	2	4	1	3	3.5	15.5	1.10	17.05	7
15	1	1	4	2	4	3	15	1.20	18.00	5
16	1	1	2	2	1	2	9	1.05	9.45	14
17	5	5	1	3	1	1	16	1.05	16.80	8
Notes	Adequacy and Condition of Mission Critical Category.									
	1-Excellent									
	2-Good									
	3-Adequate									
	4-Poor									
	5-Very Poor									
	All ratings based on MMSD staff assessments									
	Station weighting factors are based on MMSD staff assessments and range from 1 to 2.									

TABLE 2.01-3
ELECTRICAL SYSTEM CHARACTERISTICS MMSD PUMPING STATIONS (2010)

Pumping Station	Location	Firm Capacity (mgd)	Connected HP Total	Connected HP Firm	Standby Power
1	104 North First Street				c,d
	To Pumping Station 6	15.0	300	150	
	To Pumping Station 2	20.3	1,200	600	
2	833 W. Washington Ave (Brittingham Park)	41.0	2,400	1,800	c,d
3	Nine Spings	1.5	60	30	b
4	522 John Nolen Drive	4.2	240	100	b
5	5221 Lake Mendota Drive (Spring Harbor)	3.6	150	100	b,c
6	402 Walter Street	24.2	600	375	c, future d
7	6300 Metropolitan Lane	39.0	1,200	600	b,c
8	967 Plaenert Drive	34.0	1,100	800	c,future d
9	4612 Larson Beach Road, McFarland	4.5	120	80	b,c
10	110 Regas Road	42.2	1,800	1,200	c,d
11	4760 East Clayton Road	25.5	775	400	c
12	2739 Fitchrona Road	16.6	450	250	c
13	3634 Amelia Earhart Dri ve	20.0	200	100	c
14	5000 School Road	15.0	220	120	c
15	2115 Allen Blvd	5.8	400	100	c
16	1301 Gammon Road	18.7	1,500	1,000	c
17	407 Bruce Street, Verona	4.6	300	200	a
Notes	Standby Power Capabilities are as follows: a-standby generator, b-portable generator connection, c-redundant power supplies, d-with future generator connection capability.				

MADISON METROPOLITAN SEWERAGE DISTRICT
50-YEAR MASTER PLAN (2010-2060)

TECHNICAL MEMO 3
CONVEYANCE FACILITIES ANALYSIS

TABLE 2.01-4
EXISTING MMSD FORCE MAINS

Pumping Station Force Main	Length (feet)	Diameter	Material	Year Installed	Nominal Force Main Capacity Velocity Limiting @ 8 fps (mgd)	Nominal Force Main Capacity Pressure Limiting (mgd)	Required Capacity 2030 Low (mgd)	Required Capacity 2030 High (mgd)	Required Capacity 2060 Low (mgd)	Required Capacity 2060 High (mgd)	Excess Available Capacity 2060 (mgd)	Additional Required Capacity 2060 Based on 8 fps	Year Addition Required High	Year Addition Required Low
1	2,638	30	RCCP	1948	25.4		16.08	16.90	16.90	18.45	6.92			
CTFM	14,213	30	DIP	2002	25.4		16.08	16.90	16.90	18.45	6.92			
	998	20	PVC	1995	11.3		16.08	16.90	16.90	18.45		7.17		
	1,346	24	DIP	2000	16.2		16.08	16.90	16.90	18.45		2.21		
2	9,890	36	DIP	2001	36.5		27.06	29.53	29.53	33.69	2.85			
2 and 4	6,395	36	DIP	2001	36.5		29.36	31.88	31.88	36.12	0.42			
	364	36	DIP	2005	36.5		29.36	31.88	31.88	36.12	0.42			
2, 3, and 4	1,123	36	DIP	2001	36.5		30.10	32.68	32.68	36.90		0.36	2057	
3	5	8	CIP	1959	1.8		1.29	1.40	1.40	1.40	0.40			
	21	8	DIP	2000	1.8		1.29	1.40	1.40	1.40	0.40			
4	100	16	CIP	1959	7.2		3.93	4.09	4.09	4.29	2.93			
	53	16	DIP	2000	7.2		3.93	4.09	4.09	4.29	2.93			
5	28	16	DIP	1996	7.2		2.40	2.52	2.52	2.68	4.54			
	457	16	RCCP	1959	7.2		2.40	2.52	2.52	2.68	4.54			
5 and 15	1,742	24	RCCP	1959	16.2		7.47	8.54	8.54	9.52	6.72			
6	7,214	36	RCCP	1948	36.5		6.36	6.37	6.37	7.14	29.40			
7	6,996	36	RCCP	1948	36.5	27.5	22.95	29.93	29.93	36.15		8.65	2025	2042
	6,996	36	RCCP	1963	36.5	27.5	22.95	29.93	29.93	36.15		8.65	2025	2042
	1,332	48	RCCP	1963	65.0		45.90	59.86	59.86	72.30		7.35	2042	
	323	48	DIP	2005	65.0		45.90	59.86	59.86	72.30		7.35	2042	
8	13,174	42	RCCP	1964	49.7		24.27	26.17	26.17	28.02	21.71			
	194	36	RCCP	1964	36.5		24.27	26.17	26.17	28.02	8.52			
	334	42	DIP	2005	49.7		24.27	26.17	26.17	28.02	21.71			
9	4,329	20	DIP	1987	11.3		4.24	4.93	4.93	6.39	4.89			
	40	14	DIP	1987	5.5		4.24	4.93	4.93	6.39		0.86		
	2,197	10			2.8		4.24	4.93	4.93	6.39		3.57		
10	11,109	36	RCCP	1964	36.5		29.25	35.26	35.26	38.74		2.20	2040	
11	4,173	36	RCCP	1965	36.5		32.51	39.17	39.17	44.82		8.28	2025	2050
12	4,786	36	RCCP	1968	36.5		23.24	28.93	28.93	32.3	4.24			
13	1,927	36	RCCP	1969	36.5		21.56	25.77	25.77	29.44	7.10			
14	3,108	30	RCCP	1971	25.4		14.58	16.18	16.18	20.16	5.21			
	1,358	30	RCCP	1971	25.4		15.30	16.90	16.90	20.84	4.53			
15-8	1,360	24	DIP	1974	16.2		5.63	6.65	6.65	7.57	8.67			
	1,071	24	DIP	1974	16.2		5.63	6.65	6.65	7.57	8.67			
	4,837	20	RCCP	1959	11.3		5.63	6.65	6.65	7.57	3.71			
	18	24	RCCP	1959	16.2		5.63	6.65	6.65	7.57				
16	7,214	36	DIP	1979	36.5		8.53	10.24	10.24	10.55	25.99			
	2,965	30	DIP	1980	25.4		8.53	10.24	10.24	10.55	14.82			
17	13,357	16	DIP	1995	7.2		7.82	11.25	11.25	13.57		6.35	2015	2026
	3,071	20	DIP	1995	11.3		7.82	11.25	11.25	13.57		2.29	2025	2052
Total (feet)	142,947													
Total (Miles)	27.1													

Notes: MMSD has 50-million gallons of treated storage at the NSWTP that is used if the effluent flows exceed 78.6 mgd. If flows extend for a period of time, discharge to Nine Springs Creek will occur via an overflow structure.
Estimates for velocities in Force Mains 2, 3, and 4 are based on all pumping stations pumping at firm capacity at the same time.
Material abbreviations are based on the Collection System Facilities Plan Update-2008.
The old PS 7 FM (constructed in 1948) is limited to 100 feet of head (approximately 43 psi) which limits the flow from PS 7 to the NSWTP to 55 mgd (27.5 mgd in each force main.

MADISON METROPOLITAN SEWERAGE DISTRICT
50-YEAR MASTER PLAN (2010-2060)

TECHNICAL MEMO 3
CONVEYANCE FACILITIES ANALYSIS

TABLE 2.01-5
EXISTING PUMPING STATION FLOWS (1996-2006)

PS	Average	1996	1997	1998	1999	2000	2001	2002	2003	2004	2005	2006
1	5.91	7.01	6.62	6.60	6.84	6.39	6.49	5.82	5.09	3.99	5.01	5.21
2	5.29	3.97	3.51	4.27	4.15	4.45	4.20	5.07	5.00	5.83	9.05	8.68
3	0.36	0.34	0.34	0.34	0.35	0.31	0.32	0.48	0.43	0.44	0.32	0.32
4	1.08	1.07	1.03	1.26	1.22	0.92	1.10	1.07	1.05	1.14	0.97	1.02
5	0.64	0.90	0.60	0.51	0.56	0.70	0.66	0.58	0.60	0.71	0.65	0.56
6	6.45	8.25	8.19	8.23	8.32	7.73	7.54	7.12	6.38	5.07	2.51	1.63
7	18.46	19.50	18.74	19.36	20.05	20.15	20.15	19.15	18.01	18.58	14.52	14.80
8	8.05	8.10	8.02	8.21	8.33	8.77	8.50	8.05	7.65	7.86	7.52	7.52
9	0.81	0.82	0.79	0.80	0.81	0.81	0.82	0.79	0.77	0.83	0.80	0.85
10	9.44	9.59	9.79	9.95	10.53	10.76	9.84	9.50	8.26	9.37	8.21	8.00
11	7.70	7.12	7.32	6.67	7.44	7.49	7.87	7.88	7.89	8.49	8.24	8.26
12	4.67	5.03	4.71	4.16	4.70	4.31	4.48	4.51	4.56	4.86	4.98	5.05
13	5.13	4.94	5.18	4.98	5.29	5.06	4.84	4.84	4.82	5.30	5.19	5.94
14	3.33	3.05	2.84	2.90	3.23	3.33	3.49	3.44	3.66	3.66	3.40	3.62
15	1.23	0.99	1.12	1.23	1.29	1.30	1.33	1.26	1.26	1.29	1.19	1.24
16	1.67	2.25	2.05	1.29	1.45	1.51	1.50	1.55	1.59	1.61	1.73	1.81
17	0.69	0.63	0.61	0.66	0.66	0.68	0.71	0.72	0.72	0.77	0.73	0.73
Collection System												
Total Pumped Flow	78.79	75.46	81.43	81.42	85.20	84.66	83.84	81.84	77.74	79.81	67.50	67.73
Effluent Pumping												
Badfish Creek		41.70	40.81	42.06	41.23	41.50	40.40	38.38	36.85	40.29	37.48	38.63
Badger Mill Pumping		0.00	0.00	0.81	2.20	2.38	3.00	2.99	2.99	2.78	3.11	3.09
Total Effluent Pumping		41.70	40.81	42.87	43.43	43.88	43.40	41.37	39.84	43.07	40.59	41.72

TECHNICAL MEMO 3
CONVEYANCE FACILITIES ANALYSIS

TABLE 2.01-6
GRAVITY CONVEYANCE FACILITIES

Pumping Station Service Area	Total Gravity Sewer (miles)	Interceptors Reaching Capacity (miles)			
		2010	2010-2020	2020-2030	2030-2060
1	2.64	0.00	0.00	0.00	0.00
2	2.46	0.41	0.00	0.00	0.00
3	1.02	0.72	0.00	0.00	0.00
4	1.58	0.00	0.00	0.00	0.00
5	3.00	0.00	0.00	0.00	0.00
6	1.91	0.00	0.00	0.00	0.00
7	19.41	2.07	3.32	3.95	2.62
8	13.91	1.68	0.71	0.83	0.75
9	0.64	0.00	0.00	0.00	0.63
10	5.71	1.74	2.08	0.00	0.00
11	10.16	0.00	1.48	3.81	0.75
12	7.87	0.00	0.67	0.00	0.00
13	3.05	0.00	0.03	0.33	0.86
14	15.85	0.00	0.88	2.61	7.96
15	1.71	0.00	0.00	0.04	0.44
16	1.61	0.00	0.00	0.53	0.19
17	1.48	0.00	0.00	0.00	0.00
Total	94.02	6.62	9.17	12.09	14.20
		7.0%	9.8%	12.9%	15.1%
Note: See Section 3.01 C for detailed analysis.					

MADISON METROPOLITAN SEWERAGE DISTRICT
50-YEAR MASTER PLAN (2010-2060)

TECHNICAL MEMO 3
CONVEYANCE FACILITIES ANALYSIS

TABLE 2.01-7
TELEVISING HISTORY (2000-2008)

	Miles Televised								
	Year								
Interceptor	**2000**	**2001**	**2002**	**2003**	**2004**	**2005**	**2006**	**2007**	**2008**
SWI-Northerly Leg	1.07								
SWI-Southerly Leg	1.01								
SWI-Main Leg	3.82								
NEI-PS 14 to Airport	1.72								
NEI-PS 10 to SEI	2.46								
FEI and Cottage Grove Extension		3.40							
SEI		1.78							
EI-PS 1 to PS 6		1.48							
EI-PS 6 to PS 7		2.16							
Rimrock Interceptor			0.72						
EI/East Monona Extension			0.41						
SI and Baird Street Extension			1.40						
WI Spring Street Extension			0.78						
NEI-Waunakee Extension			4.20						
NSVI				6.50					
NSVI-Waubesa Extension				1.80					
NSVI-Hwy 14 Extension				1.77					
SEI				2.24					
West Interceptor-Gammon Extension				2.84					
NSVI-Mineral Point Extension					6.23				
NSVI-Midtown Extension					1.57				
NEI-Deforest Extension					9.16				
NEI-Highway 19 Extension					1.19				
SEI-Blooming Grove Extension					2.73				
SEI-Sigglekow Extension					1.01				
SEI-McFarland Relief					1.08				
SI/Baird Street Extension						0.30			
SI/Lakeside Extension						1.10			
FEI/Door Creek Extension						3.37			
NEI/ P.S. 13 to P.S. 10						4.14			
WI/PS 15 to PS 5						0.64			
WI/Randall Relief						11.45			
NEI PS 14 to PS 13							2.98		
EI							4.62		
FEI/Cottage Grove Extension							3.40		
WI/Spring Street Relief							0.87		
WI/Midvale Relief							0.50		
NEI PS 14 to PS 13								2.98	
EI								4.62	
FEI/Cottage Grove Extension								3.40	
WI/Spring Street Relief								0.87	
WI/Midvale Relief								0.50	
Northeast Interceptor/Waunakee Extension									4.93
Southwest Interceptor									3.39
West Interceptor									4.68

MADISON METROPOLITAN SEWERAGE DISTRICT
50-YEAR MASTER PLAN (2010-2060)

TECHNICAL MEMO 3
CONVEYANCE FACILITIES ANALYSIS

TABLE 2.01-8
PUMPING STATIONS TRIBUTARY TO MMSD (2006)

Owner	Maintained by Owner	Maintained by MMSD	Tributary Sewer Length
Cities			
Fitchburg			53
Madison		29	752
Middleton	8		77
Monona	7		38
Verona		1	50
Villages			
Cottage Grove	4		32
Dane	1		11
DeForest	1		39
Maple Bluff		3	7
McFarland	4		30
Shorewood Hills	1		13
Waunakee	2		60
Townships			
Blooming Grove			NDA
Burke			NDA
Madison		3	NDA
Verona			NDA
Others			
UW Campus			
Pumping Stations	6		
Grinder Pumps	4		
UW Arboretum	1		
Dane County Landfill	1		
Dane County Vilas Zoo	1		
Dane County Lake Farm Park		1	

TABLE 2.01-8
PUMPING STATIONS TRIBUTARY TO MMSD (2006)

Owner	Maintained by Owner	Maintained by MMSD	Tributary Sewer
Districts			
Blooming Grove SD 2	1		3
Blooming Grove SD 10			0
Burke Utility District 1			1
Burke Utility District 2			NDA
Burke Utility District 6			NDA
Token Creek SD	1		4
Town of Dunn SD 1		4	5
Town of Dunn SD 3		3	7
Town of Dunn SD 4			6
Kegonsa SD			20
Pumping Stations	5		
Grinder Pumps	354		
Middleton SD 5			1
Pleasant Springs SD 1			33
Pumping Stations	9		
Grinder Pumps	55		
Verona Utility District 1			3
Vienna Utility District 1	1		3
Vienna Utility District 2	1		3
Westport Utility District 1			60
Pumping Stations	10		
Grinder Pumps	1		
Westport Utility District 2			NDA
Westport Utility District 3			NDA
Westport Utility District 4			NDA
Cherokee Golf and Tennis			NDA
Windsor SD 1	3		17
Windsor SD 3			NDA
Illinois Seed Foundation			NDA
Hidden Springs SD			NDA
Lake Windsor SD			2
Morrisonville SD	1		2
Oak Springs SD			2
Totals	**483**	**44**	**1,332**
Note: NDA means no data available			

MADISON METROPOLITAN SEWERAGE DISTRICT
50-YEAR MASTER PLAN (2010-2060)

TECHNICAL MEMO 3
CONVEYANCE FACILITIES ANALYSIS

TABLE 2.01-9
CONTRIBUTING EXISTING FLOWS (2000-2008)

	Average Daily Flows (mgd)								
	Year								
	2000	**2001**	**2002**	**2003**	**2004**	**2005**	**2006**	**2007**	**2008**
Cities									
Fitchburg	1.44	1.48	1.56	1.60	1.90	1.68	1.95	1.88	1.96
Madison	29.65	28.82	27.02	25.77	28.40	26.45	26.90	28.81	31.65
Middleton	1.91	1.99	1.97	1.93	1.82	1.70	1.67	1.76	1.94
Monona	0.91	0.93	1.00	0.88	0.88	0.90	0.85	0.95	1.04
Verona	0.67	0.71	0.72	0.73	0.76	0.73	0.74	0.82	0.91
Villages									
Cottage Grove	0.34	0.34	0.34	0.34	0.56	0.58	0.63	0.68	0.76
Dane	0.05	0.05	0.05	0.05	0.05	0.06	0.06	0.06	0.06
DeForest	0.62	0.69	0.72	0.74	0.70	0.62	0.64	0.77	1.05
Maple Bluff	0.17	0.15	0.14	0.14	0.17	0.16	0.17	0.19	0.26
McFarland	0.56	0.55	0.55	0.50	0.56	0.55	0.62	0.64	0.70
Shorewood Hills	0.17	0.18	0.18	0.19	0.19	0.18	0.20	0.19	0.19
Waunakee	1.16	1.34	1.34	1.23	1.26	1.24	1.37	1.53	1.72
Townships and Districts									
Town of Blooming Grove	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01
Town of Burke	0.01	0.01	0.01	0.01	0.01	0.01	0.01		
Town of Madison	0.90	0.90	0.94	0.95	0.94	0.88	0.87	0.86	0.93
Town of Verona	0.01	0.01	0.01	0.01	0.01	0.01		0.01	
Blooming Grove SD 2	0.21	0.15	0.13	0.09	0.16	0.17	0.11	0.14	0.23
Blooming Grove SD 10	0.02	0.02	0.02	0.02	0.02	0.02	0.02	0.02	
Burke Utility District 1	0.01	0.01	0.15	0.01	0.02	0.02	0.01	0.02	0.00
Burke Utility District 2	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01
Burke Utility District 6	0.01	0.01	0.01	0.01	0.01	0.01	0.00	0.01	
Token Creek SD	0.04	0.05	0.05	0.05	0.06	0.05	0.05	0.07	0.12
Town of Dunn SD 1	0.11	0.11	0.11	0.11	0.17	0.15	0.14	0.17	0.25
Town of Dunn SD 3	0.08	0.08	0.07	0.08	0.08	0.07	0.07	0.07	0.07
Town of Dunn SD 4	0.03	0.03	0.03	0.02	0.02	0.03	0.02	0.07	0.04
Kegonsa SD	0.03	0.13	0.13	0.13	0.15	0.18	0.15	0.03	0.17
Middleton SD 5	0.02	0.02	0.02	0.02	0.02	0.02	0.02	0.02	0.02
Pleasant Springs SD 1	0.09	0.10	0.10	0.10	0.10	0.10	0.09	0.08	0.06
Verona Utility District 1	0.02	0.02	0.02	0.02	0.02	0.02	0.02	0.05	0.02
Vienna Utility District 1	0.05	0.04	0.04	0.05	0.05	0.02	0.04	0.06	0.05
Vienna Utility District 2	0.01	0.01	0.01	0.01	0.01	0.02	0.03	0.03	0.04
Westport Utility District 1	0.09	0.01	0.17	0.18	0.18	0.17	0.19	0.2	0.16
Westport Utility District 2	0.26	0.11	0.30	0.34	0.40	0.32	0.37	0.38	0.43
Westport Utility District 3	0.01	0.27	0.02	0.02	0.01	0.01	0.01	0.02	0.02
Westport Utility District 4	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01
Cherokee Golf and Tennis	0.01	0.01	0.01	0.01	0.01	0.01	0.00	0.01	0.01
Windsor SD 1	0.19	0.20	0.21	0.21	0.19	0.20	0.19	0.225	0.28
Windsor SD 3	0.01	0.01	0.01	0.01	0.01	0.01	0.00	0.01	0.01
Illinois Seed Foundation	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01
Hidden Springs SD	0.01	0.01	0.01	0.01	0.01	0.01	0.00	0.01	0.04
Lake Windsor SD	0.03	0.04	0.04	0.03	0.04	0.03	0.03	0.04	0.06
Morrisonville SD	0.05	0.08	0.05	0.04	0.06	0.05	0.05	0.07	0.08
Oak Springs SD	0.04	0.04	0.04	0.03	0.03	0.03	0.03	0.04	0.04
Infiltration into District Interceptors	1.99	1.99	1.99	1.94	1.94	1.94	1.87	1.87	1.84
Total Flow at Nine Springs	**42.01**	**41.73**	**40.32**	**38.64**	**42.01**	**39.44**	**40.21**	**42.89**	**47.25**
Notes	All flows are based on values provided in MMSD annual reports.								
	Town of Burke and Town of Verona annexed to City of Madison in 2005								

2.02 MMSD CONVEYANCE SYSTEM OPERATION

A. Routine Operations

Figure 2.02-1 is a schematic layout of the MMSD conveyance system that shows the average daily flow routings for the years 2000, 2030 Low, 2030 High, and 2060. The average daily flows presented in this figure are based on the CARPC's Collection System Evaluation (2008).

See the Development of Flows Technical Memo (TM-2) for the background information regarding the development of these average daily flows. The flows from PS 2 were developed using an allocation of 100 percent of the flow being pumped from PS 1. The flows for PS 6 were developed using an allocation of 0 percent of the flow being pumped from PS 1 to PS 6.

Figure 2.02-2 presents peak flows for the normal operating mode for the years 2000, 2020, 2030 Low, 2030 High, and 2060. The peak flows presented in this table were developed using the average daily flows presented in Figure 2.02-1 multiplied by the Madison Design Curve presented in the 1961 Greeley and Hansen Report on Sewerage and Sewage Treatment. Note that for peak flow estimates, the peak flow for a pumping station will likely be different from the sum of the upstream pumping station peak flows plus the pumping station service area's peak flow. This is due to higher flow pumping stations having proportionately lower peaking factors.

B. Alternate Operations

MMSD has developed alternate operating modes for the conveyance system to provide for redundancy and flexibility in system operations as well as minimizing potential downstream impacts on pumping stations, force mains, or interceptors that under certain peak flow conditions may not have adequate capacity. The following alternate operating modes were available to MMSD staff in 2008. Each will be described separately.

1. PS 15 to PS 16

When PS 16 was first constructed, the intent was for PS 15 to pump wastewater to PS 16 as the primary mode of operation. Because of the relatively high cost of pumping wastewater from PS 15 to PS 16 as well as the odors caused at PS 16 (next to a Middleton Grade School), the MMSD staff decided to modify interceptors downstream of the original PS 15 routing and minimize the use of the option of pumping from PS 16 to PS 15. This has also eliminated many downstream concerns for other pumping stations (particularly PS 12 and PS 11) and their related interceptors and force mains. This connection remains to provide operational redundancy for PS 8.

2. Cross-Town Force Main PS 2

The normal operating mode of the CTFM is for PS 1 to pump to PS 2. PS 2, however, does have the piping flexibility to pump to PS 1 as an alternate operating mode should any of the conveyance system downstream of PS 2 normal routing have operating issues that would limit its capacity. The CTFM was upgraded in 2004 and serves as a key system component in allowing most of this wastewater that flows to PS 1 to be routed away from PS 7. This provides key system flexibility as well as improved redundancy at PS 7.

Under normal conditions in 2010, all flows during the day are routed from PS 1 through the CTFM to PS 2. To exercise the two pumps that pump to PS 6 and to keep the sewage fresh in the force main to PS 6, one of the pumps operates for one wet well cycle each night pumping to PS 6. The total volume pumped in such a cycle varies from 150,000 to 200,000 gallons.

3. Gravity Diversion of PS 2 to PS 8 via Southwest Interceptor

A gravity diversion link exists between PS 2 and PS 8. This link, consisting of a portion of the Southwest Interceptor (SWI) along Haywood Street that normally conveys wastewater to PS 2, can deliver wastewater from PS 2 to PS 8 should enough differential head develop to drive the flow backwards through the interceptor. At present, the diversion capacity of this sewer under high flows is approximately 6 mgd. Such a diversion allows flow to be diverted from PS 2 to the West Interceptor–Randall Relief upstream of PS 8 during high flow events, which consequently provides capacity relief to PS 2.

Flow from PS 8 can also be diverted to PS 2 through this line when the depth of flow in the West Interceptor–Randall Relief reaches the invert elevation of the SWI at the intersection of Wingra Drive and Haywood Street.

4. Gravity Diversion of PS 15 to PS 5 via Original West Interceptor

There is a limited amount of capacity available at PS 5 to provide relief to PS 15 if maintenance is required at PS 15. This diversion is scheduled for rehabilitation because of the poor condition of the pipe.

5. Gravity Diversion of PS 16 to PS 5 via West Interceptor Gammon Extension

There is a limited diversion capacity available from PS 16 directly to PS 5 through the WI-Gammon Extension, but since the interceptor is only 10 inches to 14 inches, the capacity available would be of use only under low flow conditions to relieve flow at PS 16.

6. Potential Link between PS 6 and PS 10

As an improvement to the overall system flexibility and redundancy, MMSD in its 2002 Collection System Facilities Plan identified a potential force main or gravity sewer project that would allow wastewater to be transferred between PS 6 and PS 10. The project was identified as a potential project for the years beyond 2020 in the 2002 Collection System Facilities Plan.

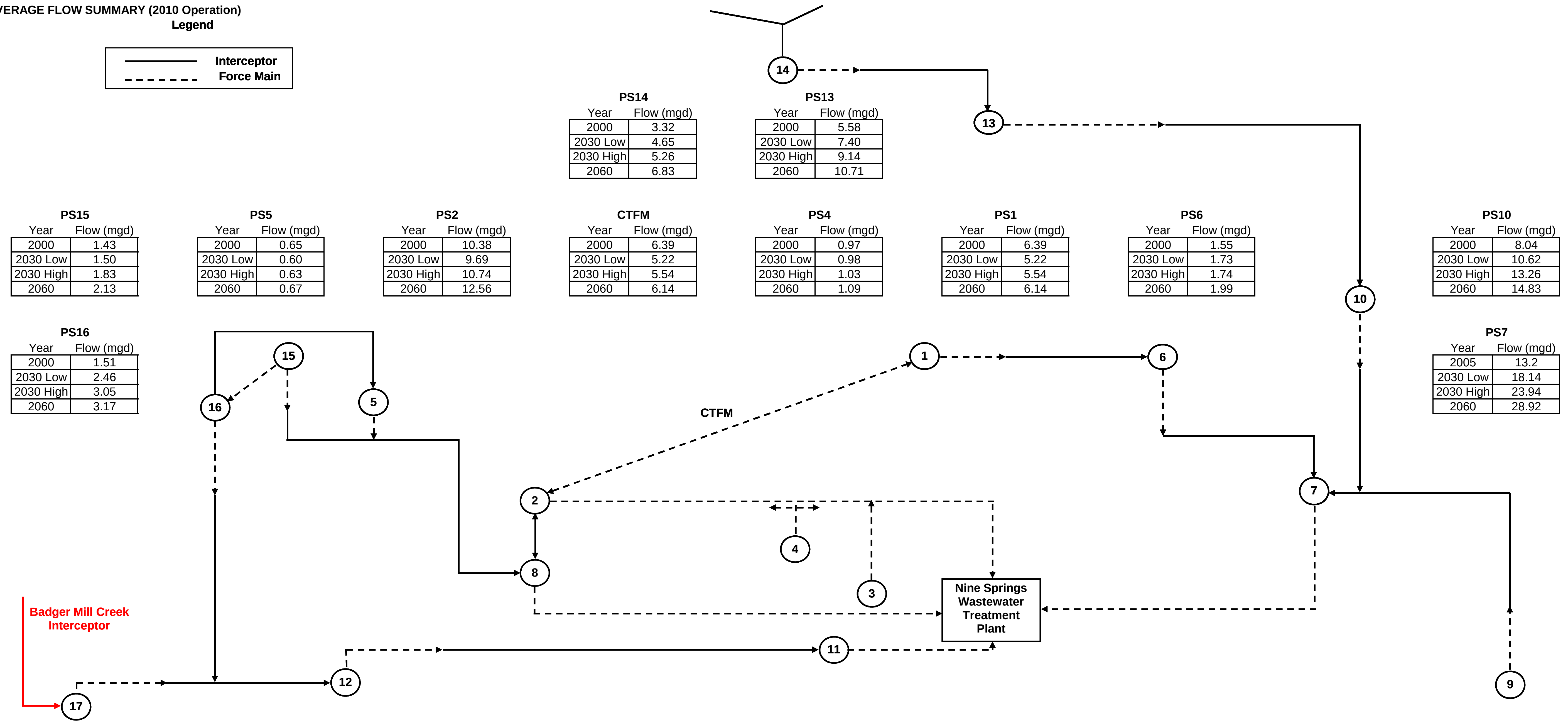
C. System Operation-Nine Springs WTP

All flows entering and leaving the NSWTP, with the exception of an overflow of the 50-million-gallon effluent storage lagoons, are pumped. Pumping Stations 2, 3, 4, 7, 8 and 11 pump directly to the Nine Springs Wastewater Treatment Plant. Electrical costs (2006) associated with pumping all wastewater from each individual pumping station are presented in Figure 2.02-3. The costs for pumping represent a significant portion of the MMSD overall electricity requirements. Effluent pumping is located at the NSWTP site with a capacity limit of 75 mgd to Badfish Creek and 4.3 mgd to Badger Mill Creek.

MADISON METROPOLITAN SEWERAGE DISTRICT
50-YEAR MASTER PLAN (2010-2060)

TECHNICAL MEMO 3
CONVEYANCE FACILITIES ANALYSIS

FIGURE 2.02-1
COLLECTION SYSTEM SCHEMATIC
AVERAGE FLOW SUMMARY (2010 Operation)



PS14

Year	Flow (mgd)
2000	3.32
2030 Low	4.65
2030 High	5.26
2060	6.83

PS13

Year	Flow (mgd)
2000	5.58
2030 Low	7.40
2030 High	9.14
2060	10.71

PS15

Year	Flow (mgd)
2000	1.43
2030 Low	1.50
2030 High	1.83
2060	2.13

PS5

Year	Flow (mgd)
2000	0.65
2030 Low	0.60
2030 High	0.63
2060	0.67

PS2

Year	Flow (mgd)
2000	10.38
2030 Low	9.69
2030 High	10.74
2060	12.56

CTFM

Year	Flow (mgd)
2000	6.39
2030 Low	5.22
2030 High	5.54
2060	6.14

PS4

Year	Flow (mgd)
2000	0.97
2030 Low	0.98
2030 High	1.03
2060	1.09

PS1

Year	Flow (mgd)
2000	6.39
2030 Low	5.22
2030 High	5.54
2060	6.14

PS6

Year	Flow (mgd)
2000	1.55
2030 Low	1.73
2030 High	1.74
2060	1.99

PS10

Year	Flow (mgd)
2000	8.04
2030 Low	10.62
2030 High	13.26
2060	14.83

PS16

Year	Flow (mgd)
2000	1.51
2030 Low	2.46
2030 High	3.05
2060	3.17

PS7

Year	Flow (mgd)
2005	13.2
2030 Low	18.14
2030 High	23.94
2060	28.92

PS17

Year	Flow (mgd)
2000	0.67
2030 Low	2.22
2030 High	3.41
2060	4.27

PS12

Year	Flow (mgd)
2000	4.47
2030 Low	8.08
2030 High	10.48
2060	11.95

PS8

Year	Flow (mgd)
2000	8.87
2030 Low	8.51
2030 High	9.31
2060	10.09

PS11

Year	Flow (mgd)
2000	7.56
2030 Low	12.04
2030 High	15.03
2060	17.63

PS3

Year	Flow (mgd)
2000	0.31
2030 Low	0.32
2030 High	0.35
2060	0.35

NSWTP

Year	Flow (mgd)
2000	41.29
2030 Low	49.68
2030 High	60.40
2060	60.40

PS9

Year	Flow (mgd)
2000	0.81
2030 Low	1.07
2030 High	1.28
2060	1.75

Prepared by Strand Associates, Inc.
\S:\MAD\1500--1599\1547\001\Spr\Technical Memos\CFA Spreadsheets\\CFA Tables Sections 1, 2 and 3 With District Comments (042009).xlsx\Figure 2.02-1 Average Flows\8/25/2009

MADISON METROPOLITAN SEWERAGE DISTRICT
50-YEAR MASTER PLAN (2010-2060)

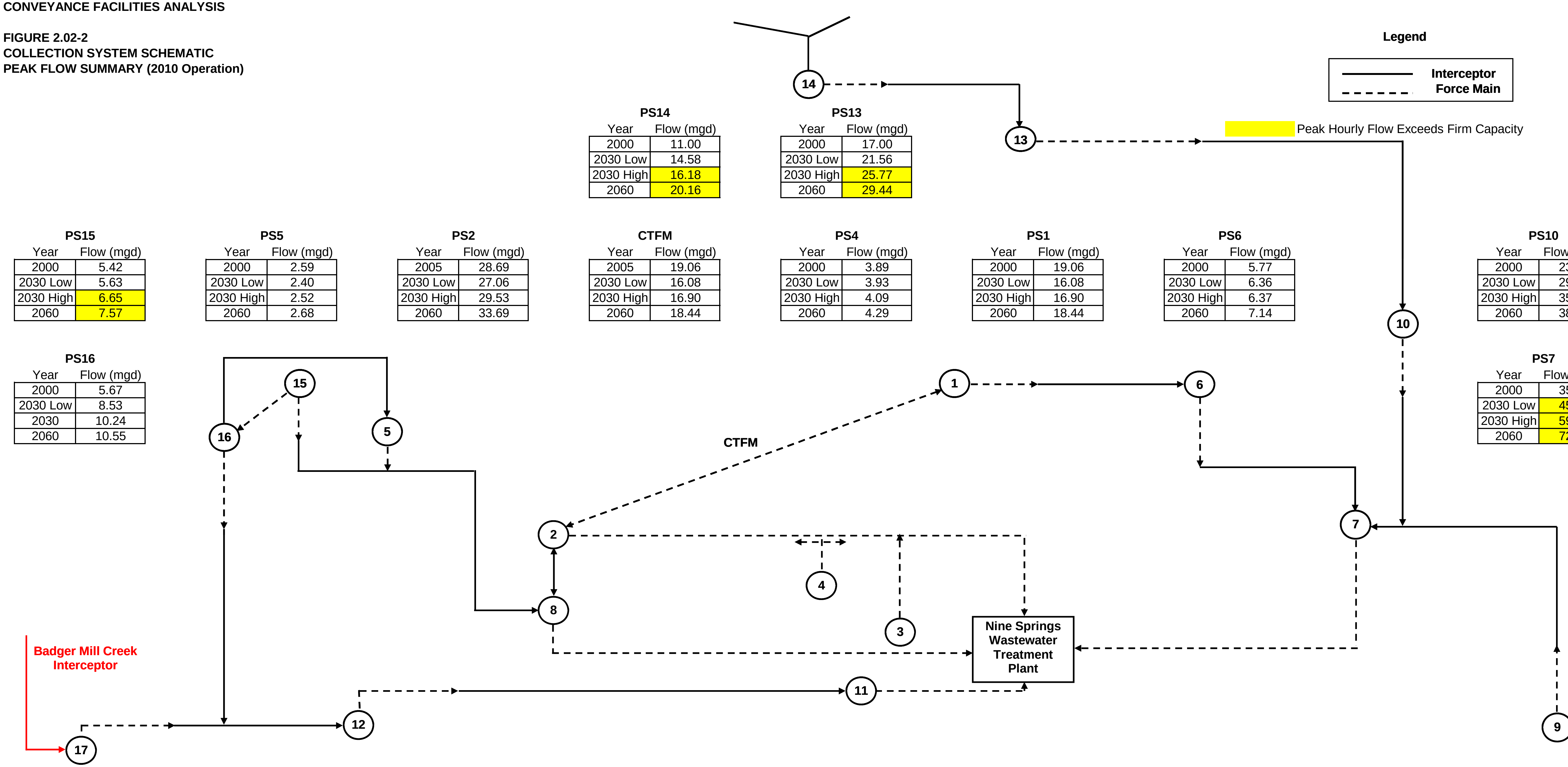
TECHNICAL MEMO 3
CONVEYANCE FACILITIES ANALYSIS

FIGURE 2.02-2
COLLECTION SYSTEM SCHEMATIC
PEAK FLOW SUMMARY (2010 Operation)

Legend

Interceptor
Force Main

Peak Hourly Flow Exceeds Firm Capacity



PS14

Year	Flow (mgd)
2000	11.00
2030 Low	14.58
2030 High	16.18
2060	20.16

PS13

Year	Flow (mgd)
2000	17.00
2030 Low	21.56
2030 High	25.77
2060	29.44

PS15

Year	Flow (mgd)
2000	5.42
2030 Low	5.63
2030 High	6.65
2060	7.57

PS5

Year	Flow (mgd)
2000	2.59
2030 Low	2.40
2030 High	2.52
2060	2.68

PS2

Year	Flow (mgd)
2005	28.69
2030 Low	27.06
2030 High	29.53
2060	33.69

CTFM

Year	Flow (mgd)
2005	19.06
2030 Low	16.08
2030 High	16.90
2060	18.44

PS4

Year	Flow (mgd)
2000	3.89
2030 Low	3.93
2030 High	4.09
2060	4.29

PS1

Year	Flow (mgd)
2000	19.06
2030 Low	16.08
2030 High	16.90
2060	18.44

PS6

Year	Flow (mgd)
2000	5.77
2030 Low	6.36
2030 High	6.37
2060	7.14

PS10

Year	Flow (mgd)
2000	23.13
2030 Low	29.25
2030 High	35.26
2060	38.74

PS16

Year	Flow (mgd)
2000	5.67
2030 Low	8.53
2030	10.24
2060	10.55

PS7

Year	Flow (mgd)
2000	35.13
2030 Low	45.90
2030 High	59.85
2060	72.30

PS17

Year	Flow (mgd)
2000	2.69
2030 Low	7.82
2030 High	11.25
2060	13.57

PS12

Year	Flow (mgd)
2000	14.12
2030 Low	23.24
2030 High	28.93
2060	32.30

PS8

Year	Flow (mgd)
2000	25.13
2030 Low	24.27
2030 High	26.17
2060	28.02

PS11

Year	Flow (mgd)
2000	21.98
2030 Low	32.51
2030 High	39.17
2060	44.82

PS3

Year	Flow (mgd)
2000	1.24
2030 Low	1.29
2030 High	1.40
2060	1.40

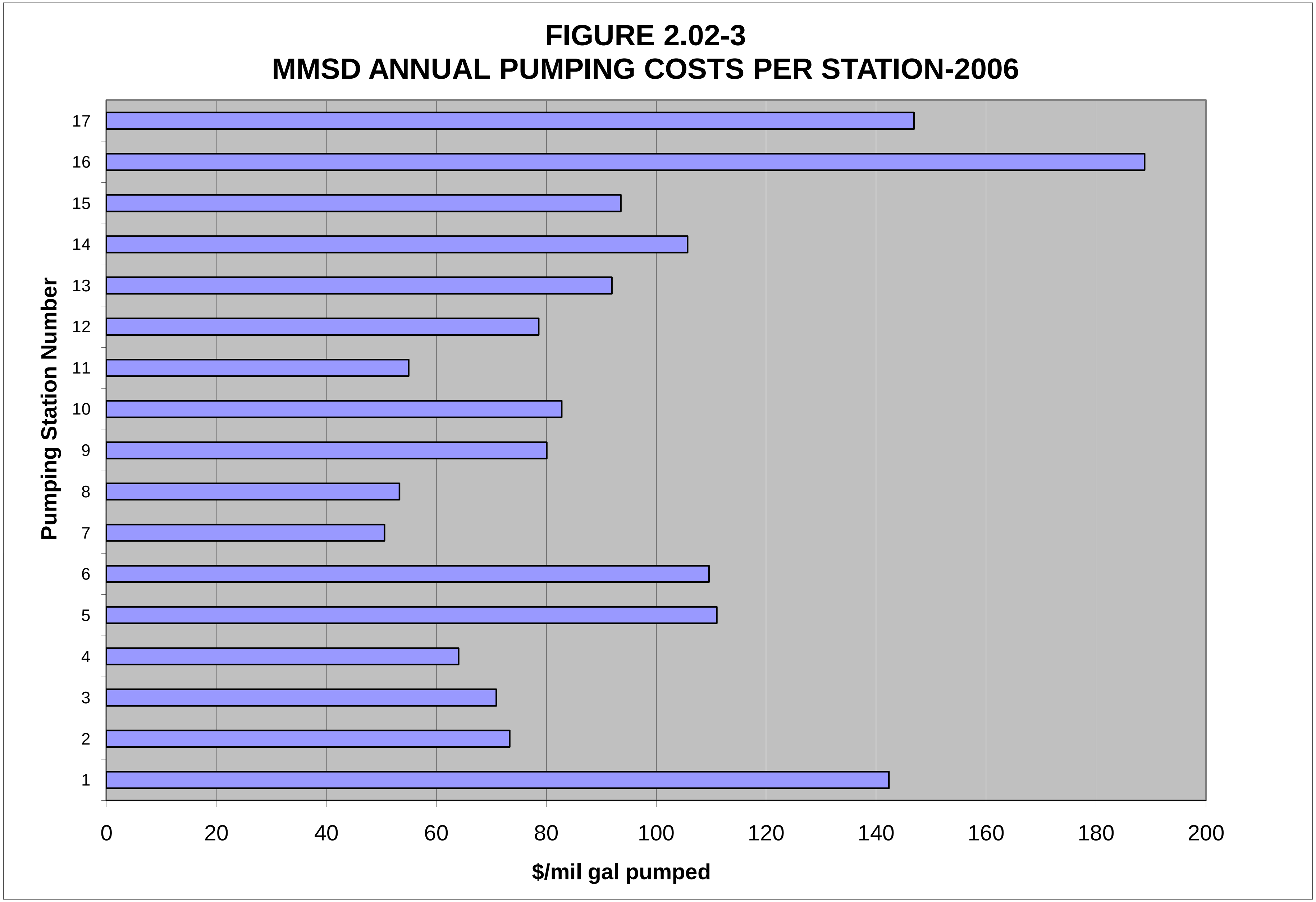
NSWTP

Year	Flow (mgd)
2000	103.23
2030 Low	124.20
2030 High	151.00
2060	151.00

PS9

Year	Flow (mgd)
2000	3.24
2030 Low	4.24
2030 High	4.93
2060	6.39

Prepared by Strand Associates, Inc.
S:\MAD\1500--1599\1547\001\Spr\Technical Memos\CFA Spreadsheets\\CFA Tables Sections 1, 2 and 3 With District Comments (042009).xlsx\Figure 2.02-2 Peak Flows\8/25/2009



S:\MAD\1500--1599\1547\001\Spr\Technical Memos\CFA Spreadsheets\\CFA Tables Sections 1, 2 and 3 With District Comments (042009).xlsx\Figure 2.02-3 MMSD Annual Pump\8/25/2009

2.03 PS 1-104 NORTH FIRST STREET, MADISON, WISCONSIN

A. Areas Served by Pumping Station

PS 1 conveys gravity drainage from the contributory sewers within its service area. Areas of the District that comprise the PS 1 Service Area include portions of the City of Madison, all of the Village of Maple Bluff, and portions of the Town of Burke and the Town of Madison. Figure 2.03-1 shows the location of PS 1 as well as highlighting features of this pumping station.

PS 1 collects wastewater from the North Basin Interceptor and City of Madison sewers and pumps to either the East Interceptor (EI) or through the CTFM to PS 2. Pumping from PS 1 to the EI in 2008 was typically at night to provide flow in the force main to the EI to minimize the long-term deposition of solids that could create maintenance or downstream loadings issues when the force main from PS 1 to the EI would be needed in peak flow events. In addition, since the pumps that pump to the EI are smaller, their operation is more efficient during low flows. Pumping from PS 1 through the CTFM was the normal mode of operation during the day in 2008. Average daily flows as well as peak hourly flows presented in the collection system analyses in this technical memo are based on the assumption that 0 percent of the flow reaching PS 1 will be pumped to PS 6 and 100 percent of the flow reaching PS 1 will be pumped to PS 2 via the CTFM. Figure 2.03-2 shows the gravity drainage area of the District served by PS 1 and the interceptors and force mains serving PS 1 as well as the capacity needs for any infrastructure associated with PS 1.

B. Description of Pumping Station

1. History of Station

PS 1, located at 104 North First Street in Madison, was constructed and placed into service in 1950. PS 1 is located at the site of the first two treatment plants owned by the City of Madison as well as the offices of the District from 1950 through 1982. This pumping station included two pumps from the old Booster Station No. 1 that were used to pump through the CTFM. Modifications to this station include:

a. The rebuilding of the cross-town pump motors in 1982.
b. Replacement of the automatic power transfer switch in 1983.
c. Installation of a new telemetry system in 1984.
d. Changes to the electrical and control systems in 1988.
e. Radios for telemetry in 1990 (replaced in 2000).
f. Repairs to the concrete canopy in 1994.
g. Replacement of the roof in 1999.
h. A major upgrade in 2004 that included removal of bar screens, new electrical systems, HVAC, and new pumps, motors and variable frequency drives to allow for additional pumping from PS 1 to PS 2 via the CTFM.

2. Current Design Capacities and Limitations

The maximum pumping capacity of PS 1 is 38.3 mgd with PS 1 pumping 18 mgd to PS 6 via the PS1 FM and the EI, and 20.3 mgd through the CTFM. Firm pumping capacity for PS 1 is 35.3 mgd with 15 mgd capacity to PS 6 via the PS1 FM and EI, and 20.3 mgd capacity to PS 2 via the CTFM. Average daily flows from 1996 through 2007 have varied from 4 mgd to 7 mgd. Higher flows occur during wet years while the lower flows occur during dry years.

Under normal conditions in 2010, all flows during the day are routed from PS 1 through the CTFM to PS 2. To exercise the two pumps that pump to PS 6 and to keep the sewage fresh in the force main to PS 6, one of the pumps operates for one wet well cycle each night pumping to PS 6. The total volume pumped in such a cycle varies from 150,000 to 200,000 gallons.

The 2008 Condition Assessment rated this pumping station with an overall rating of 11.38, making its ordinal ranking 12 out of 17. All elements of the station were rated as excellent with the exception of mechanical condition, which was rated good to excellent. PS 1 has a criticality factor of 1.75.

The actual ranking for each element was as follows:

Adequacy	Rating
Maximum Flow Capacity	Excellent
Firm Flow Capacity	Excellent
Power System Redundancy	Excellent
Mechanical Condition	Good to Excellent
Building and Structural Condition	Excellent
Electrical Condition	Excellent
Criticality Factor	1.75
Overall Score	11.38

The drainage area for PS 1 is entirely developed, and no District pumping stations contribute flow to this pumping station. Without a change in the land use in the area tributary to PS 1 or a reduction of infiltration/inflow, the flows to PS 1 would not be expected to change appreciably in the future. However, this area does include areas of Madison where higher density residential development is currently occurring and is expected to continue to occur. In addition, this area has also been an area where significant rehabilitation of contributory customer's sewers has occurred with some reduction in infiltration/inflow having been documented. There is also a provision for pumping from PS 2 to PS 1 as an alternate operating mode should maintenance be required for the PS 2 force main.

3. Additional Near-Term Planned Improvements (2010-2020)

With the 2004 renovation, this pumping station is in excellent structural condition and has new electrical controls, new pumps, new ventilation, and better electrical redundancy. No major projects are anticipated in the near term (2010-2020). In the 2008 Condition Assessment, this station received a priority ranking of 12 (of 17) indicating it is one of the stations least likely to need near-term improvements.

4. Long-Term Considerations (2020-2060)

a. Pumping Station: No additional capacity improvements are required for this pumping station prior to the year 2060. The pumping station's last major rehabilitation occurred in 2004. Based on typical lives of equipment and facilities, it is anticipated the following rehabilitation projects would occur prior to 2060:

(1)	Roofing (25-Year Life)	2029 and 2054
(2)	Electrical (25-Year Life)	2029 and 2054
(3)	HVAC Equipment (20-Year Life)	2024 and 2044

b. Force Mains: The basis for determining force main capacity needs is a nominal 8 fps velocity in the force main. Velocities in two segments of the CTFM, (CTFMi and CTFMii) from PS 1 to PBXT-01337 (1,346 feet of 24-inch pipe) and from RDXT-09244 to PBXT-10254, (998 feet of 20-inch pipe), will exceed the nominal 8 fps by2060. The projected 2060 deficit for CTFMi is 2.21 mgd, resulting in a peak velocity of 10.0 fps, and the projected 2060 deficit for CTFMii is 7.17 mgd, resulting in a peak velocity of 14.4 fps. Segment CTFMii was at capacity in the year 2000. The pumps at PS 1 pumping into the CTFM have a firm capacity of 20.3 mgd. Although the velocity within these short sections of the force main can exceed 8 fps during high flow events, higher flow velocities in these short segments for the limited duration and frequency of these events is probably acceptable.

c. Interceptors: Only one short (3-foot) interceptor segment (2Xiii) that drains by gravity to PS 1 is projected to reach capacity prior to 2060. This segment is at capacity based on the 2000 flows. Since the length is short, there are likely limited operational impacts related to this interceptor segment.

5. Operational Impacts of PS 1 and the CTFM

PS 1 can pump to either PS 2 or PS 6. The current practice is to pump to PS 2 except for one nighttime pumping cycle when flows are pumped to PS 6 to exercise those pumps and prevent gas accumulation in the force main. Therefore, the operation of PS 1 and the utilization of the CTFM play a significant role in determining the flows seen at PS 2.

Although PS 6 has adequate capacity, PS 7 has been identified as in need of an upgrade or, possibly, a parallel station (proposed PS 18). Pumping all flow from PS 1 to PS 2 assists in reducing the flow to PS 7 (particularly during storm events). However, when a new PS 18 paralleling PS 7 is operational, it may become advantageous or reduce operating costs if more flow is directed from the PS 1 Service Area to PS 6 and subsequently to PS 7 or PS 18.

6. System and Electrical Redundancy Review-*Review MMSD Emergency Response Manual*

Electrical Power for PS 1 is fed from two separate MG&E substations from circuit ID numbers *BLD 1304 and RKN 1337.* PS 1 does not have permanent standby power (no onsite generator). However, a circuit breaker location for a future generator was included in the switchgear (2004 upgrade) for use if standby power is at some point deemed necessary. According to the March 13, 2007, MMSD Emergency Response Manual, during a service outage there is adequate storage in the interceptors and local sewers upstream of this pump station to prevent any basement back-ups or sewer overflows for one hour under normal flow conditions and 30 minutes under high flow conditions. .

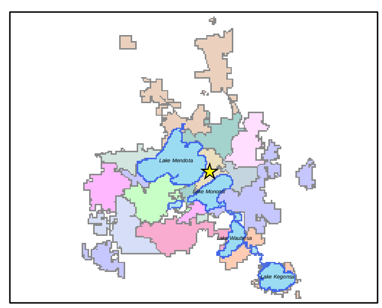
At this time, MMSD does not have a portable generator large enough to power the largest pumps at PS1; however, MMSD does have a portable generator large enough to power PS1C and PS1D. Both of these pumps pump to PS6. Most likely, due to the size of the two pumps (150 hp), the backup generator from PS 17 would have to be used for this purpose since it is likely to be the only unit that could run either of these two pumps. A portable generator set connection is not presently available so the generator would have to be hardwired into the drives or starters for the pumps or alternatively, into the motor control center (MCC) at Pumping Station 1, taking care not to back-feed into MG&E's system (the breaker feeding the MCC must be disengaged from the bus stabs).

S:\@SAI\501--550\547\001\Data\GIS\Figures\Figure 2.03-1 PS1.mxd

PS 1-First Street
104 North First Street, Madison

Originally Constructed	1950
Upgrades	2004
Capacity (2008)	
Maximum (mgd)	38.3
Firm (mgd)	35.3

Pumps	Q (gpm)	H (ft)	Motor (HP)	Year On-Line
1A	14,100	134	600	2005
1B	14,100	134	600	2005
1C	10,375	31	150	1950
1D	12,500	41	150	1950



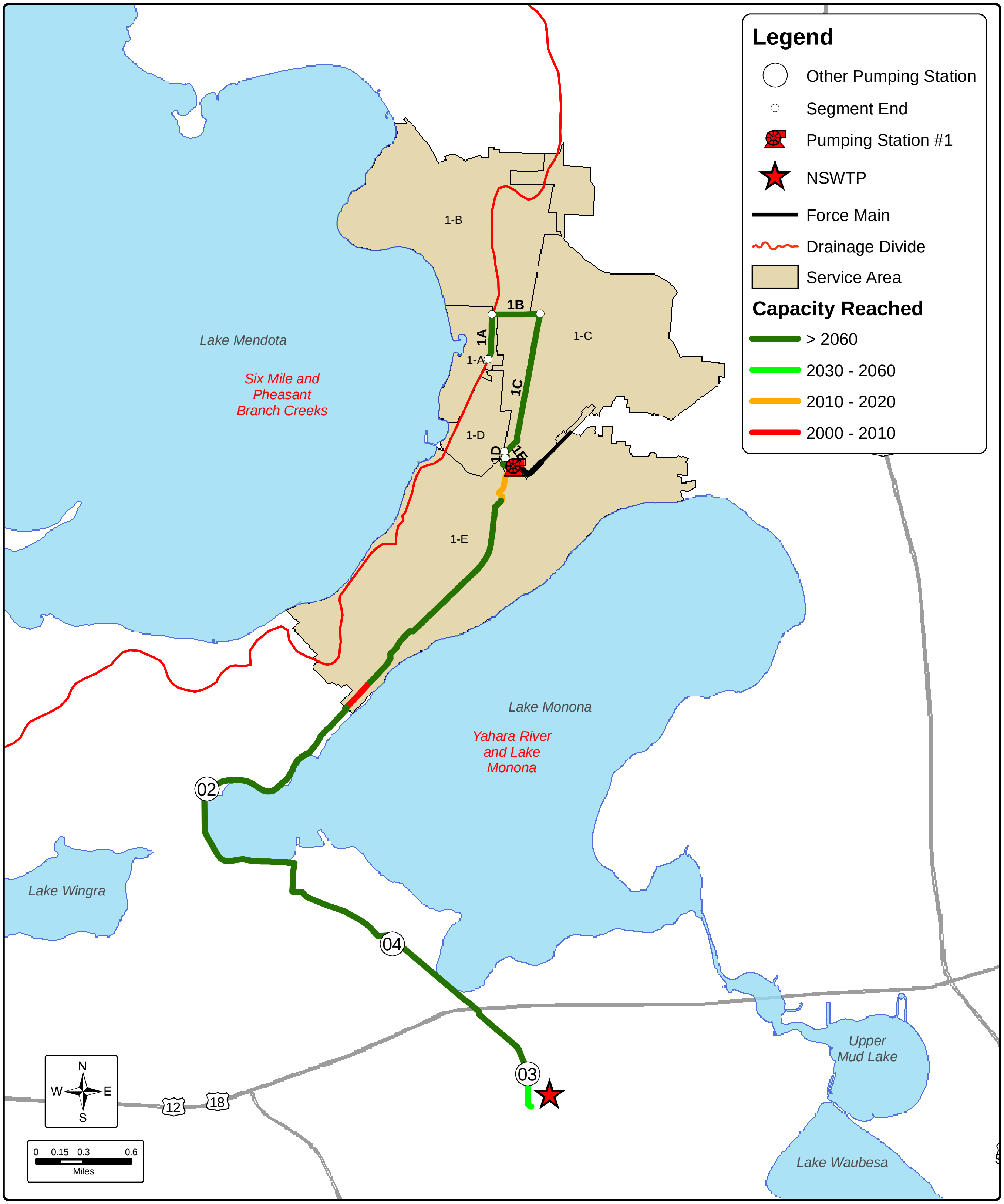
PS 1 - 104 NORTH FIRST STREET
CITY OF MADISON

CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT

STRAND ASSOCIATES, INC.® ENGINEERS

FIGURE 2.03-1
1-547.001

S:\@SAI\501--550\547\001\Data\GIS\Figures\Figure 2.03-2 PS 1 Drainage Area 8x11.mxd



PS 1 - 104 NORTH FIRST STREET
CITY OF MADISON

INTERCEPTOR AND FORCE MAIN ANALYSIS
CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT

STRAND ASSOCIATES, INC.® ENGINEERS

FIGURE 2.03-2
1-547.001

2.04 PS 2–833 WEST WASHINGTON AVENUE (BRITTINGHAM PARK)

A. Areas Served by Pumping Station

PS 2, located in Brittingham Park, is a key station in the overall system and one of the pumping stations that discharges directly to the NSWTP. Figure 2.04-1 shows the location of the station and highlights some of its features. Figure 2.04-2 shows the area that drains by gravity to PS 2 and the area of gravity drainage for PS 1 that contributes to PS 2 via the Cross Town Force Main (CTFM). The PS 2 gravity drainage area is the near west and central portion of the City of Madison, including a portion of the University of Wisconsin campus. This pumping station is the termination point for the West Interceptor (WI) and the Madison Brittingham Interceptor. Gravity drainage accounted for 50 percent of the average daily flow at this pumping station based on flows between 2005 and 2007. PS 2 flows are discharged through a force main that ends at the NSWTP. PS 3 and 4 also contribute flows to this force main.

Following completion of the PS 1 renovation and replacement of the CTFM, PS 2 receives pumped flow from PS 1 through the CTFM. The current (2008) practice is to send wastewater from PS 1 to PS 6 during one nighttime pumping cycle (to keep the FM to PS 6 flushed) and to PS 2 via the CTFM at all other times. Figure 2.04-2 shows the force main and interceptor analysis for PS 2 gravity drainage.

B. Description of Pumping Station

1. History of Station

PS 2, located at 833 West Washington Avenue (Brittingham Park), Madison, was constructed and placed into service in 1964 with two pumps relocated from the old PS 2. Modifications to this station include the following:

a. Revisions to two pumping units and electrical equipment in 1980.
b. Electrical and duct replacement in 1983.
c. Installation of a new telemetry system in 1984.
d. Radios for telemetry system in 1990 (replaced in 2000).
e. Revisions to pump electrical controls in 1991.
f. Surge tank level control automation in 1999.
g. A major upgrade in 2004 that included removal of bar screens, new electrical, HVAC, and new pumps, motors, and variable frequency drives to allow for additional pumping from PS 1 via the CTFM.

2. Current Design Capacities and Limitations

The firm pumping capacity of PS 2 following the 2004 renovation is 41 mgd. Average daily flow pumped at this station for 2005 through 2007 (after renovation of PS 1 and completion of the new CTFM) was 8.9 mgd.

The 2008 Condition Assessment rated this pumping station with an overall rating of 13.00, making its ordinal ranking 10 out of 17. All elements of the station were rated as excellent with the exception of Mechanical Condition, which was rated as good to excellent. Pumping Station 2 has a criticality factor of 2.0.

The actual ranking for each element was as follows:

Adequacy	Rating
Maximum Flow Capacity	Excellent
Firm Flow Capacity	Excellent
Power System Redundancy	Excellent
Mechanical Condition	Good to Excellent
Building and Structural Condition	Excellent
Electrical Condition	Excellent
Criticality Factor	2.00
Overall Score	13.00

The drainage area for PS 2 is entirely developed, so any large increase in flow to the pumping station will come from PS 1 or a significant increase in population density within the PS 2 or PS 1 Service Area. The service area of PS 1 would likely only see a flow increase from increased population density. A gravity diversion exists between PS 2 and PS 8 that could be used to assist in alleviating capacity issues at PS 2 or PS 8.

3. Additional Near-Term Planned Improvements (2010-2020)

With the 2004 renovation, this pumping station is in excellent structural condition and has new electrical controls, new pumps, new ventilation, and better electrical redundancy. No major projects are anticipated in the near term (2010-2020). In the 2008 Condition Assessment, this station received a priority ranking of 10 (of 17) indicating it is one of the stations least likely to need near-term improvements.

4. Long-Term Considerations (2020-2060)

a. Pumping Station: No additional capacity improvements are required for this pumping station prior to the year 2060. The pumping station's last major rehabilitation occurred in 2004. Based on typical lives of equipment and facilities, it is anticipated that the following rehabilitation projects would occur prior to 2060:

(1)	Roofing (25-Year Life)	2029 and 2054
(2)	Electrical (25-Year Life)	2029 and 2054
(3)	HVAC Equipment (20-Year Life)	2024 and 2044

b. Force Mains: The combined force main shared by PSs 2, 3, and 4 is projected to reach its capacity downstream of PS 3 by the year 2057 under high flow projections only. The capacity projection is based on a maximum force main velocity of 8 fps. The actual need to modify this force main will depend on the pump capacities in PS 2, 3, and 4 and the respective operating conditions for each pumping station should the force main reach its projected capacity of 36.5 mgd, which is only 0.4 mgd less than the projected peak hourly combined flow of 36.9 mgd.

c. Interceptors: Only a short section of the West Interceptor on Regent Street will reach capacity before 2060.

5. Operational Impacts of Gravity Diversion from PS 2 to PS 8

A gravity diversion link exists between PS 2 and PS 8. This link, consisting of a portion of the SWI along Haywood Street that normally conveys wastewater to PS 2, can deliver wastewater from PS 2 to PS 8 should enough differential head develop to drive the flow backwards through the interceptor. At present, 6 mgd can be diverted to PS 8 without surcharging the gravity sewer system to the point of causing basement backups. Such a diversion allows flow to be diverted from PS 2 to the SWI upstream of PS 8 during high flow events, which consequently provides capacity relief to PS 2, or during emergency or planned service interruptions at PS 2. The SWI Haywood rehabilitation or replacement scheduled between 2011 and 2020 could provide additional capacity relief to PS 2. The potential to provide additional diversion capacity will be investigated during the planning phases of the project.

6. System and Electrical Redundancy Review

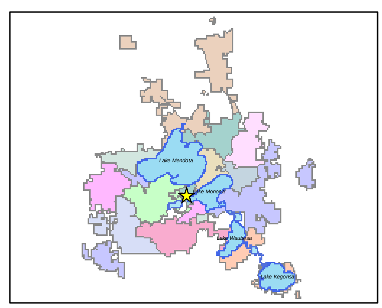
Electrical power for PS 2 is fed from two separate MG&E substations from circuit ID numbers ECA 1311 and WGA 1313. PS 2 does not have permanent standby power (no onsite generator). Additionally, MMSD does not presently have a portable generator large enough to power any of the pumps at PS2. However, a circuit breaker location for a future generator was included in the switchgear (2004 upgrade) for use if standby power is at some point deemed necessary. According to the March 13, 2007, MMSD Emergency Response Manual, during a service outage there is adequate storage in the interceptors and local sewers upstream of this pump station to prevent any basement back-ups or sewer overflows for one hour under normal flow conditions and 30 minutes under high flow conditions. A portion of the flow, estimated at 6 mgd, may be routed to PS 8 if capacity is available at PS 8. The estimated hours are based on PS 1 pumping to PS 2. An alternative option that would provide for additional outage time would be to divert PS 1 flows to PS 6, if capacity is available at PS 6 and, subsequently, PS 7.

S:\@SAI\501--550\547\001\Data\GIS\Figures\Figure 2.04-1 PS2.mxd

PS 2-Brittingham Park
Brittingham Park, Madison

Originally Constructed	1964
Upgrades	2004
Capacity (2008)	
Maximum (mgd)	
Firm (mgd)	41

Pumps	Q (gpm)	H (ft)	Motor (HP)	Year On-Line
2A	16,500	108	600	2004
2B	16,500	108	600	2004
2C	16,500	108	600	2004
2D	16,500	108	600	2004



PS 2 - 833 W. WASHINGTON - BRITTINGHAM PARK
CITY OF MADISON

CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT

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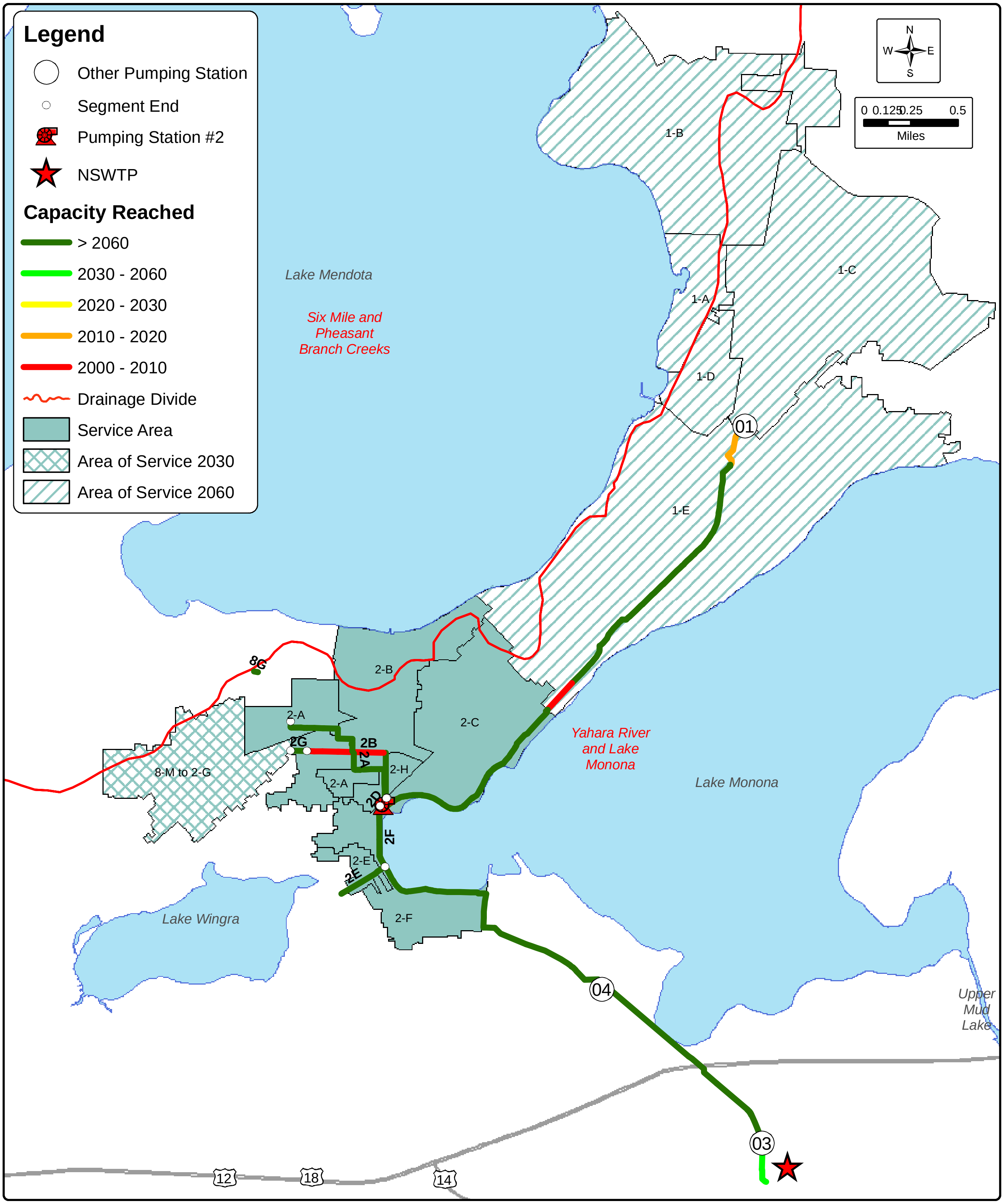
FIGURE 2.04-1
1-547.001

S:\@SAI\501--550\547\001\Data\GIS\Figures\Figure 2.04-2 PS 2 Drainage Area 8x11.mxd



PS 2 - 833 W. WASHINGTON - BRITTINGHAM PARK
CITY OF MADISON

INTERCEPTOR AND FORCE MAIN ANALYSIS
CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT

STRAND ASSOCIATES, INC.® ENGINEERS

FIGURE 2.04-2
1-547.001

2.05 PS 3–NINE SPRINGS

A. Areas Served by Pumping Station

PS 3 is located at the NSWTP near the Effluent Pumping Station. PS 3 exclusively conveys gravity drainage from sewers within its service area. Areas of the District that comprise the PS 3 Service Area include small portions of the Cities of Fitchburg, Madison, and Monona and the Town of Madison near the NSWTP. The Rimrock Interceptor is the only interceptor that drains to this pumping station, which pumps to a force main that also accepts discharges from PS 2 and PS 4. Figure 2.05-1 shows an aerial view of PS 3 and technical data for this pumping station. Figure 2.05-2 shows the area of the District served by PS 3 and the network of pumping stations, force mains, and interceptors associated with PS 3.

B. Description of Pumping Station

1. History of Station

PS 3, MMSD's smallest pumping station, was acquired from the City of Monona and placed into service in 1959. This pumping station is located at the NSWTP. Revisions to the pumping station, including the addition of new pumps, took place in 1980. In 1984 a new telemetry system was installed and in 1990, radios were added to the telemetry system (the radios were subsequently replaced in 2000). In 1998 the electrical and control systems were replaced.

2. Current Design Capacities and Limitations

The firm pumping capacity of PS 3 is approximately 1.51 mgd. The average daily flow pumped by this station was 0.36 mgd from 1996 through 2007. The drainage area for PS 3 is largely developed, and no District pumping stations are tributary to its service area. As a result, only a significant change in the land use resulting in an increase in population density tributary to PS 3 would increase the flows pumped by this station.

The 2008 Condition Assessment rated this pumping station with an overall rating of 14.00, making its ordinal ranking 9 out of 17. The actual rating for each element was as follows:

Adequacy	Rating
Maximum Flow Capacity	Good
Firm Flow Capacity	Good
Power System Redundancy	Adequate
Mechanical Condition	Good
Building and Structural Condition	Poor
Electrical Condition	Excellent
Criticality Factor	1.00
Overall Score	14.00

3. Additional Planned Near-Term Improvements (2010-2020)

The 2002 *Collection System Facilities Plan* recommended a major rehabilitation project for PS 3 primarily based on the type and age of the station rather than its capacity. This is currently scheduled for completion in 2019 based on the MMSD 2009 Capital Projects Budget.

None of the effluent diversion alternatives would impact this pumping station since the station only receives gravity drainage from its service area, which is entirely in the Yahara River basin.

4. Long-Term Considerations (2020-2060)

   a. Pumping Station: PS 3 is not projected to reach its rated firm capacity of 1.51 mgd prior to 2060.

   b. Force Mains: The combined force main shared by PSs 2, 3, and 4 is projected to reach its capacity downstream of PS 3 by the year 2057 under high flow projections only. The capacity projection is based on a maximum force main velocity of 8 fps. The actual need to modify this force main will depend on the pump capacities in PSs 2, 3, and 4 and the respective operating conditions for each pumping station should the force main reach its projected capacity of 36.5, which is only 0.4 mgd less than the projected peak hourly combined flow of 36.9 mgd.

   c. Interceptors: Two short segments (3i and 3ii) of intercepting sewer immediately upstream of PS 3 from the juncture between the interceptor from Monona and the Rimrock interceptor were projected to be at capacity in 2000. The projected capacity deficits (for 2060) are 0.32 mgd and 0.40 mgd, respectively. The need, if any, for any additional capacity depends on the potential for excessive surcharging in the upstream sewers.

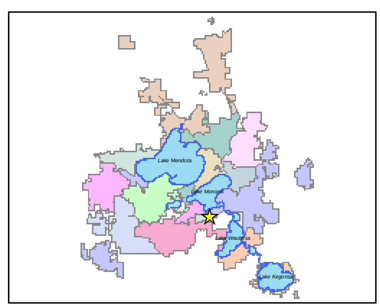
5. System and Electrical Redundancy Review

Electrical power for PS 3 is fed from a single service from the MG&E's Nine Springs substation, circuit ID number NSP 1320. Since this station is located at the NSWTP, any standby power or redundant power provisions would be incorporated with the NSWTP. In addition, because of its relatively low hourly flow, a system of hauling from the station in the event of a prolonged power outage is an acceptable alternative. In addition, the District has a 208 V 3-phase portable generator available to power this station and a quick connect generator receptacle and transfer switch at the pumping station. The Nine Springs substation also has a standby generator owned by MG&E that can provide power to this substation if a power outage occurs at the substation. According to the March 13, 2007, MMSD Emergency Response Manual, during a service outage there is adequate storage in the interceptors and local sewers upstream of this pump station to prevent any basement back-ups or sewer overflows for three hours under normal flow conditions and one hour under high flow conditions.

S:\@SAI\501--550\547\001\Data\GIS\Figures\Figure 2.05-1 PS3.mxd

PS 3-Nine Springs
NSWTP

Originally Constructed				1959
Upgrades				1998
				2004
Capacity (2008)				
Maximum (mgd)				1.51
Firm (mgd)				1.51
Pumps	Q (gpm)	H (ft)	Motor (HP)	Year On-Line
3A	1,050	60	30	1980
3B	1,050	60	30	1980



PS 3 - NSWTP
CITY OF MADISON

CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT

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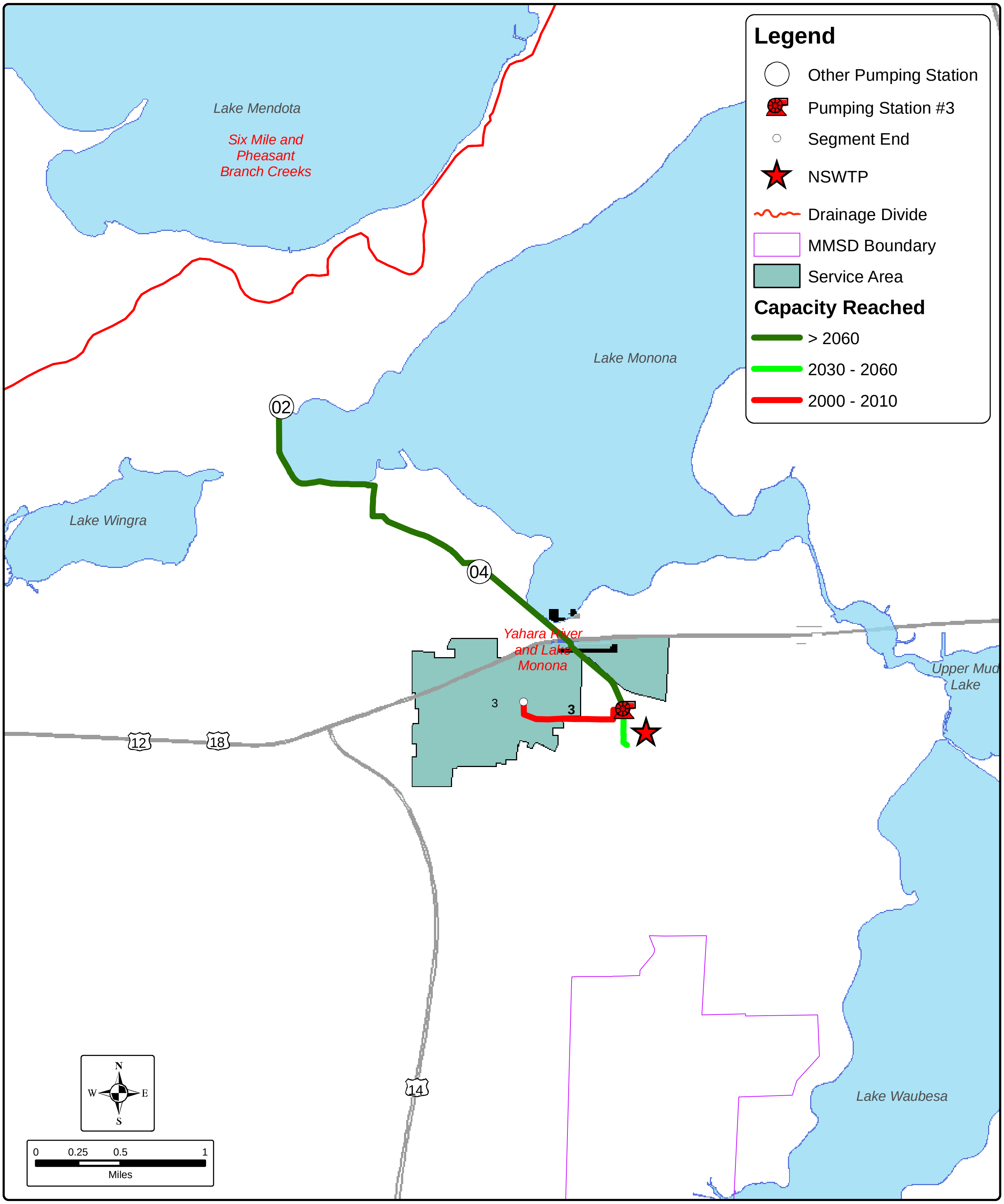
FIGURE 2.05-1
1-547.001

S:\@SAI\501--550\547\001\Data\GIS\Figures\Figure 2.05-2 PS 3 Drainage Area 8x11.mxd



PS 3 - NSWTP
CITY OF MADISON

INTERCEPTOR AND FORCE MAIN ANALYSIS
CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT

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FIGURE 2.05-2
1-547.001

2.06 PS 4–620 JOHN NOLEN DRIVE, MADISON

A. Areas Served by Pumping Station

PS 4 conveys gravity drainage from sewers within its service area. Figure 2.06-1 shows the location of the pumping station and highlights some of its features. Areas of the District that comprise the PS 4 Service Area include portions of the Town of Madison, all of which will be incorporated into the City of Madison by 2020, and the south side of the City of Madison. This pumping station collects drainage from the SI and pumps to a force main shared by PSs 2 and 3 that terminates at the NSWTP. Figure 2.06-2 shows the gravity drainage area of the District served by PS 4 and the network of pumping stations, force mains, and interceptors associated with PS 4. The interceptor and force main analysis for this pumping station are also presented in Figure 2.06-2.

B. Description of Pumping Station

1. History of Station

PS 4, located at 620 John Nolen Drive in Madison, was constructed and placed into service in 1967. Modifications to this pumping station include removal of the comminutor in 1975, a new telemetry system in 1984, radios for the telemetry system in 1990 (replaced in 2000), a roof replacement in 1994, and the installation of a manual transfer switch for a generator in 1996. Additionally, MMSD negotiated an agreement with MG&E to install an automatic transfer switch at the pumping station site to provide a second feed to the station site (installed in 2005).

2. Current Design Capacities and Limitations

The firm pumping capacity of PS 4 is 4.2 mgd. Average daily flows pumped by this station from 1996 to 2007 show a steady average flow rate for PS 4 of 1.1 mgd. The drainage area for PS 4 is largely developed, and no District pumping stations are tributary to its service area. As a result, only a significant change in the land use resulting in a significant increase in population density or the addition of a large industrial or commercial water user would increase the flows pumped by this station.

The 2008 Condition Assessment rated this pumping station with an overall rating of 17.60, making its ordinal ranking 6 out of 17. Improvements are scheduled to be made between 2015 and 2020. The actual rating for each element was as follows:

Adequacy	Rating
Maximum Flow Capacity	Adequate
Firm Flow Capacity	Adequate
Power System Redundancy	Adequate
Mechanical Condition	Good
Building and Structural Condition	Good
Electrical Condition	Adequate
Criticality Factor	1.10
Overall Score	17.60

3. Additional Planned Near-Term Improvements (2010-2020)

The 2009 Capital Projects Fund Budget includes a 2011 project to replace a portion of the SI near Baird Street.

An improvement project for PS 4 is scheduled for completion in 2019 based on the 2009 MMSD Capital Projects Budget.

None of the effluent diversion alternatives would impact this pumping station since the station only receives gravity drainage from its service area, all of which is near the NSWTP in the Yahara River Basin.

4. Long-Term Considerations (2020-2060)

   a. Pumping Station: PS 4 is projected to reach its firm pumping capacity by about the year 2047 based on the high flow projections. Sufficient capacity exists in PS 4 under low flow projections through the year 2060. The projected peak hourly deficit is 0.09 mgd.

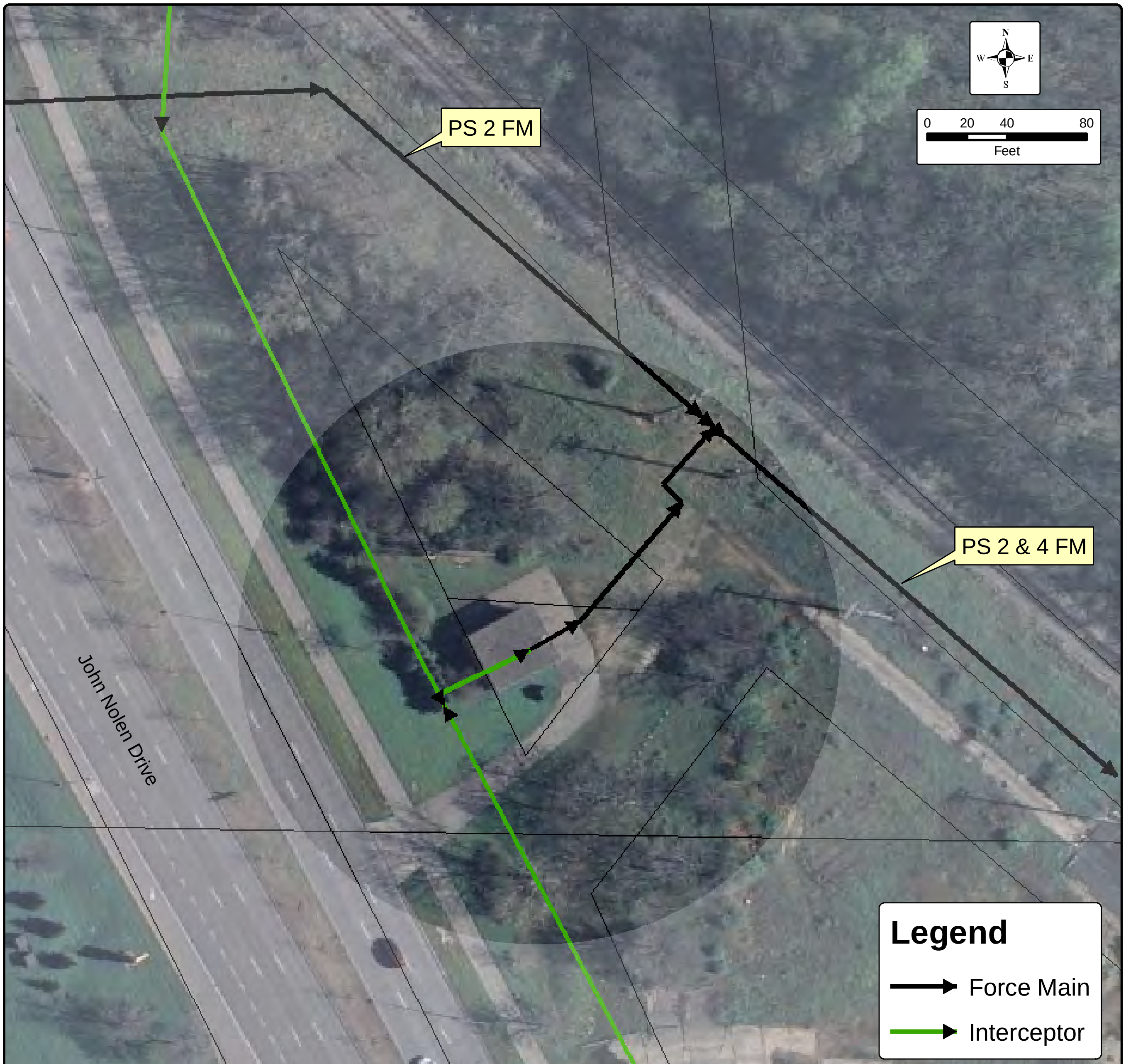
   b. Force Mains: The capacity of the force main for PS 4 upstream of the combined force main for PSs 2 and 4 is not expected to reach its projected capacity of 36.5 mgd prior to the year 2060. The combined force main shared by PSs 2, 3, and 4 is projected to reach its capacity downstream of PS 3 by the year 2050 under high flow projections only. The capacity projection is based on a maximum force main velocity of 8 fps. The actual need to modify this force main will depend on the pump capacities in PSs 2, 3, and 4 and the respective operating conditions for each pumping station should the force main reach its projected capacity of 36.5 which is only 0.4 mgd less than the projected peak hourly combined flow of 36.9 mgd.

   c. Interceptors: No interceptors tributary to PS 4 are expected to reach their capacity prior to the year 2060.

5. System and Electrical Redundancy Review

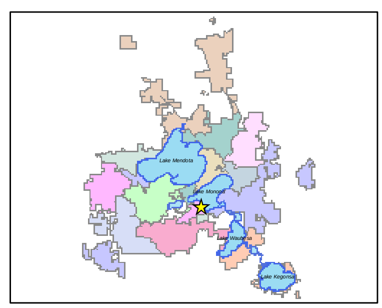
Pumping Station 4 is served from two separate circuits off of separate busses from a single substation, the Nine Springs Substation owned by MG&E. Circuits NSP 1317 and NSP 1318 provide redundant feeds to the station. A 480 V, 3-phase portable generator is also available to provide standby power at this pumping station, which is equipped with a quick connect generator receptacle and transfer switch. According to the March 13, 2007, MMSD Emergency Response Manual, during a service outage there is adequate storage in the interceptors and local sewers upstream of this pump station to prevent any basement back-ups or sewer overflows for three hours under normal flow conditions and one hour under high flow conditions.

S:\@SAI\501--550\547\001\Data\GIS\Figures\Figure 2.06-1 PS4.mxd



PS 4-John Nolen
620 John Nolen Drive, Madison

Originally Constructed				1967
Upgrades				1996
				2004
Capacity (2008)				
Maximum (mgd)				4.2
Firm (mgd)				4.2
Pumps	Q (gpm)	H (ft)	Motor (HP)	Year On-Line
4A	2,000	47	40	1967
4B	2,900	95	100	1967
4C	2,900	95	100	1967



PS 4 - 620 JOHN NOLEN DRIVE
CITY OF MADISON

CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT

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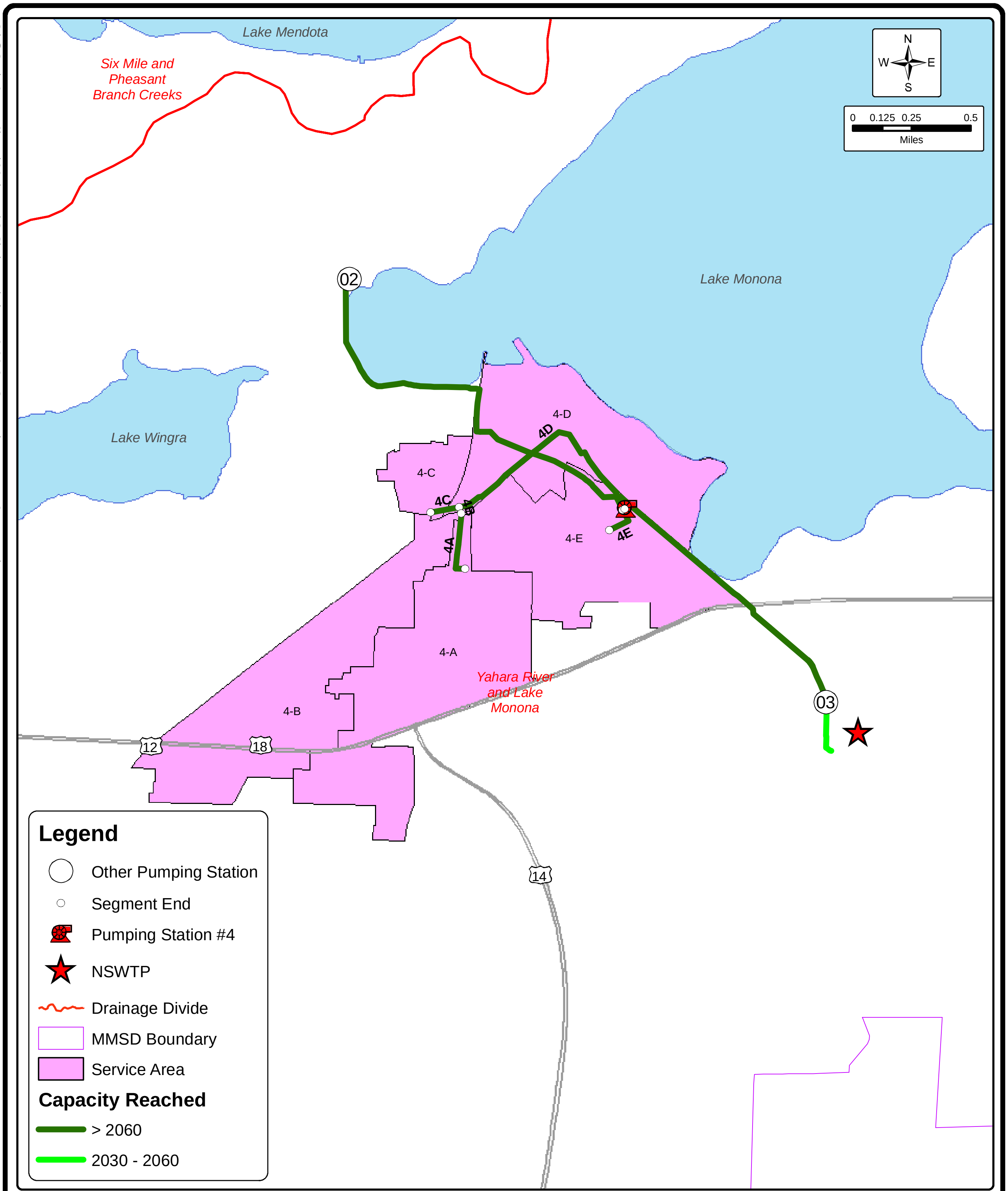
FIGURE 2.06-1
1-547.001



PS 4 - 620 JOHN NOLEN DRIVE
CITY OF MADISON

INTERCEPTOR AND FORCE MAIN ANALYSIS
CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT

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FIGURE 2.06-2
1-547.001

2.07 PS 5–5221 LAKE MENDOTA DRIVE (SPRING HARBOR), MADISON

A. Areas Served by Pumping Station

PS 5 primarily pumps gravity drainage from sewers within its respective service area, but as an alternative operating mode, it can also pump a limited amount of flow from either the PS 15 Service Area or the PS 16 Service Area. Figure 2.07-1 shows the location of this station and highlights some of its features, which include restrooms for the adjacent City park. Areas of the District that comprise the PS 5 Service Area include portions of the west side of the City of Madison and east side of the City of Middleton. This pumping station pumps to the WI. Figure 2.07-2 shows the gravity drainage area of the District served by PS 5 and the interceptors and force main associated with this station.

B. Description of Pumping Station

1. History of Station

PS 5, located in Spring Harbor Park in Madison, was constructed and placed into service in 1996. This pumping station replaced the old PS 5 in its entirety. A new telemetry radio was installed in 2000 as part of the radio system upgrade.

2. Current Design Capacities and Limitations

The firm pumping capacity of PS 5 is 3.6 mgd. Average daily flow pumped by this station from 1996 through 2007 was 0.64 mgd.

The 2008 Condition Assessment assessed the condition of all MMSD conveyance pumping stations. PS 5 was assigned an overall rating of 7.20, making its ordinal ranking (priority) 17 of 17 or the station least likely in need of renovation since it was only placed in service in 1996. The actual rating for each element was as follows:

Adequacy	Rating
Maximum Flow Capacity	Excellent
Firm Flow Capacity	Excellent
Power System Redundancy	Excellent
Mechanical Condition	Excellent
Building and Structural Condition	Excellent
Electrical Condition	Excellent
Criticality Factor	1.20
Overall Score	7.20

Anticipated growth on Madison's west side should not significantly affect this pumping station, which has a largely developed service area and no tributary pumping stations.

3. Additional Near-Term Planned Improvements (2010-2020)

The 2009 Capital Projects Fund Budget identified a project to replace a portion of the WI upstream of PS 5 in the year 2010. This would be interceptor segment 5A. One option being considered is to reroute this flow from PS 5 to PS 15.

None of the effluent diversion alternatives would impact this pumping station since normally, the station only receives gravity drainage from its service area, all of which lies in the Yahara River Basin. If flow from the PS 15 Service Area is routed to PS 5, a small amount of flow would be generated in the Black Earth Creek portion of the Wisconsin River watershed. However, no effluent diversion alternatives are planned for this watershed.

4. Long-Term Considerations (2020-2060)

The interceptors tributary to PS 5, the pumping station capacity, and the combined force main capacity for PS 15 and 5 will not reach their respective capacities prior to the year 2060.

5. Operational Impacts of Diversion from PS 16 to PS 5.

A connection between PS 16 Service Area and the gravity drainage service area of PS 5 exists but would likely only be used in emergency conditions since both the PS 16 bypass line and PS 5 have limited capacity relative to PS 16 flows.

6. System and Electrical Redundancy Review

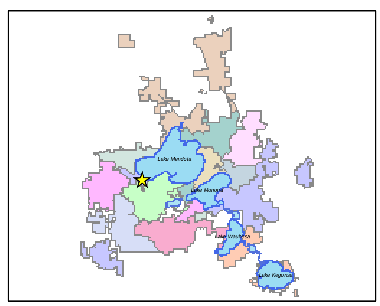
PS 5 is served from two separate circuits, BLK 1332 and BLK 1335, on separate busses, from a single MG&E substation. A 480 V 3-phase portable standby generator is available for use at this pumping station, which is equipped with a quick connect generator receptacle and interlocked circuit breakers. According to the March 13, 2007, MMSD Emergency Response Manual, during a service outage there is adequate storage in the interceptors and local sewers upstream of this pump station to prevent any basement back-ups or sewer overflows for two hours under normal flow conditions and one hour under high flow conditions.

S:\@SAI\501--550\547\001\Data\GIS\Figures\Figure 2.07-1 PS5.mxd

PS 5-Spring Harbor
Spring Harbor Park, Madison

Originally Constructed	1996
Upgrades	-
Capacity (2008)	
Maximum (mgd)	3.6
Firm (mgd)	3.6

Pumps	Q (gpm)	H (ft)	Motor (HP)	Year On-Line
5A	1,800	75	50	1996
5B	1,800	75	50	1996
5C	1,800	75	50	1996



PS 5 - SPRING HARBOR PARK
CITY OF MADISON

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MADISON METROPOLITAN SEWERAGE DISTRICT

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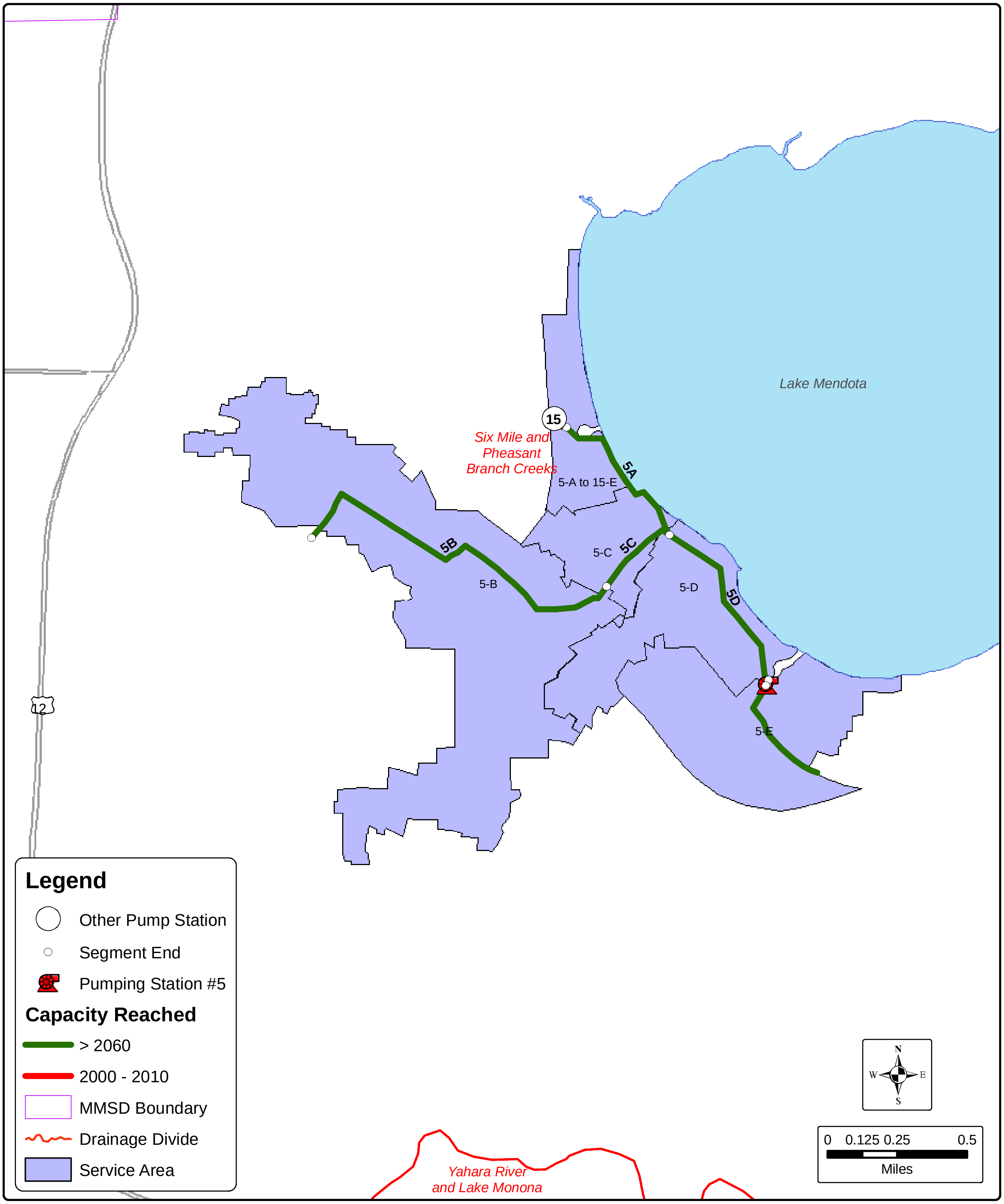
FIGURE 2.07-1
1-547.001

S:\@SAI\501--550\547\001\Data\GIS\Figures\Figure 2.07-2 PS 5 Drainage Area 8x11.mxd



PS 5 - SPRING HARBOR PARK
CITY OF MADISON

INTERCEPTOR AND FORCE MAIN ANALYSIS
CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT

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FIGURE 2.07-2
1-547.001

2.08 PS 6-402 WALTER STREET (OLBRICH PARK), CITY OF MADISON

A. Areas Served by Pumping Station

PS 6 pumps primarily gravity drainage from sewers tributary to the pumping station, but does receive pumped flow from PS 1. Figure 2.08-1 shows the location of this pumping station and highlights some of its features. Areas of the District that comprise the PS 6 gravity drainage area include a portion of the east side of the City of Madison and Utility District 2 of the Town of Blooming Grove. This pumping station is on the EI, and therefore collects drainage from the East Interceptor upstream of the pumping station and pumps into the East Interceptor downstream of the pumping station. Figure 2.08-2 shows the gravity drainage area of PS 6 as well as the service area of PS 1**.** This figure also shows the network of pumping stations, force mains, and interceptors associated with PS 6 including the PS 1 Force Main connection.

Gravity drainage accounted for 92 percent of the average daily flow pumped at this pumping station after May 2005. Prior to the completion of the PS 1 project and the CTFM replacement, this station had a much lower percentage of gravity drainage flow.

Under normal conditions in 2010 all flows during the day are routed from PS 1 through the CTFM to PS 2. To exercise the two PS 1 pumps that pump to PS 6 and to keep the sewage fresh in the PS 1 Force Main to PS 6, one of the pumps operates for one wet well cycle each night pumping to PS 6. The total volume pumped in such a cycle varies from 150,000 to 200,000 gallons.

B. Description of Pumping Station

1. History of Station

PS 6, located at 402 Walter Street in Madison, was constructed and placed into service in 1950. Modifications to this station include:

a. New telemetry system installed in 1984.
b. Improvement of the electrical and control systems in 1987.
c. Radios added to the telemetry system in 1990 (replaced in 2000).
d. Roof replacement in 1992.
e. 2009 Rehabilitation project included four new pumps and motors, one equipped with a new variable frequency drive, that increased firm capacity, provided new electrical services and controls, HVAC modifications and structural modifications.

2. Current Design Capacities and Limitations

The maximum pumping and firm pumping capacity of PS 6 after the 2009 renovation is 24.2 mgd. Average daily flow pumped by PS 6 for 2005 to 2007 was 2.1 mgd, which was a dramatic decrease from the average daily flow pumped from 1996 to 2004 (7.4 mgd). This decrease resulted from rerouting PS 1 flow from PS 6 to PS 2 through the CTFM. The drainage area for PS 6 is largely developed, so any large increase in flow to the pumping station will come from another pumping station (PS 1 or potentially PS 10) or a change in land use tributary to this station that would result in a significant increase in population density within the service area thereby increasing the flows pumped by this station. The only tributary pumping station at this point is PS 1, which now typically pumps to the CTFM. Like PS 6, the drainage area served by PS 1 will not experience much growth because of its central location within the City of Madison unless population density increases. The only major source of future capacity limitation comes from a potential link with PS 10. This link would potentially divert flow from PS 10 to PS 6 to provide limited capacity relief and redundancy for PS 10.

The 2008 Condition Assessment assessed the condition of all MMSD conveyance pumping stations. PS 6 based on the current upgrade, was assigned an overall rating of 7.80, which meant an ordinal ranking (priority) of 15 out of 17. The actual rating for each element was as follows:

Adequacy	Rating
Maximum Flow Capacity	Excellent
Firm Flow Capacity	Excellent
Power System Redundancy	Excellent
Mechanical Condition	Excellent
Building and Structural Condition	Excellent
Electrical Condition	Excellent
Criticality Factor	1.30
Overall Score	7.80

3. Additional Near-Term Planned Improvements (2010-2020)

The only interceptor repair project associated with the gravity drainage areas of PS 6 is the replacement or lining of the East Monona Interceptor at Fair Oaks Avenue because of cracked pipe sections. This project is scheduled for completion in 2012 according to the 2009 MMSD Capital Projects Budget.

None of the effluent diversion alternatives would impact this pumping station since the station only receives gravity drainage from its service area located in the Yahara River Basin.

3. Long-Term Considerations (2020-2060)

a. Pumping Station: No additional capacity improvements are required for this pumping station prior to the year 2060. A major rehabilitation project at this pumping station is currently under construction. Based on typical lives of equipment and facilities, it could be anticipated that the following rehabilitation projects would occur prior to 2060:

(1) Roofing (25-Year Life) 2035 and 2060
(2) Electrical (25-Year Life) 2035 and 2060
(3) HVAC Equipment (20-Year Life) 2030 and 2050

b. Force Mains: The force main is not expected to reach its capacity prior to the year 2060.

c. Interceptors: None of the interceptors in the gravity drainage service area for PS 6 are expected to reach capacity prior to the year 2060.

d. Other Projects: A potential link between PS 6 and PS 10 to provide operational flexibility is under consideration by the District but no specific project timetable has been set.

5. PS 1 and PS 10 Operational Impacts on PS 6

PS 1 is designed to pump flows to either PS 2 (normal operation) or alternatively, PS 6 (nightly cycle, high flows, and emergency operation). PS 10 is not currently connected to PS 6; however, the District has considered a force main or gravity sewer link from PS 10 to PS 6 that could be used to divert flow in either direction. This link is included in the projects listed above.

6. Potential System-wide Impacts of a Link Between PS 6 and PS 10

The proposed link between PS 6 and PS 10 could provide relief to PS 10 during high flows and is economically somewhat attractive because of the relative proximity of PS 10 and PS 6. Use of the CTFM to divert flow away from PS 6, and therefore PS 7, has provided more capacity at PS 6 to convey flow diverted there by this proposed link from PS 10. Such a link, however, would not provide relief to PS 7 because PS 6 is also tributary to PS 7. Once PS 18 is constructed and placed in service, the ability to route flows from PS 10 to PS 6 may be desirable to take advantage of the potential available capacity at PS 7. This force main, or gravity connector (which could allow transfer from PS 10 to PS 6 or PS 6 to PS 10) could be sized to divert normal flows from PS 10 to PS 6 and hence to PS 7 allowing maintenance of PS 10 or PS 18 or vice versa.

7. System and Electrical Redundancy Review

The 2010 PS upgrade included providing redundant feeds from two circuits, *MIL 443 and RYS 443*, originating from two different substations. There is no current provision for standby power at this station; however, a provision, a circuit breaker with a keyed interlock, was included in the main switchgear for a future generator connection should an on-site generator be deemed necessary. Although a portable generator connection has not been installed at PS 6, MMSD does have a portable generator large enough to power any single pump at PS 6. (Note: the portable generator at PS 17 could power more than one pump at a time at this station.) A portable generator would have to be hardwired into the drive or starters for the pumps. As with any time a generator is used, extreme caution must be taken to prevent any possible back feeding of power into the utility's system. According to the March 13, 2007, MMSD Emergency Response Manual, during a service outage there is adequate storage in the interceptors and local sewers upstream of this pump station to prevent any basement back-ups or sewer overflows for one hour under normal flow conditions and 30 minutes under high flow conditions with PS 1 pumping to PS 6. With PS 1 flows pumping to PS 2 as in normal operation, allowable outages are anticipated to be substantially greater.

S:\@SA\501--550\547\001\Data\GIS\Figures\Figure 2.08-1 PS6.mxd

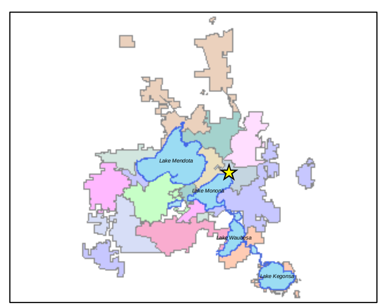
PS 6-Walter Street
402 Walter Street, Madison

Originally Constructed	1950
Upgrades	2009

Capacity (2008)

Maximum (mgd)	24.2
Firm (mgd)	24.2

Pumps	Q (gpm)	H (ft)	Motor (HP)	Year On-Line
6A	7,700	45	125	2009
6B	7,700	45	125	2009
6C	7,700	45	125	2009
6D	7,700	45	125	2009



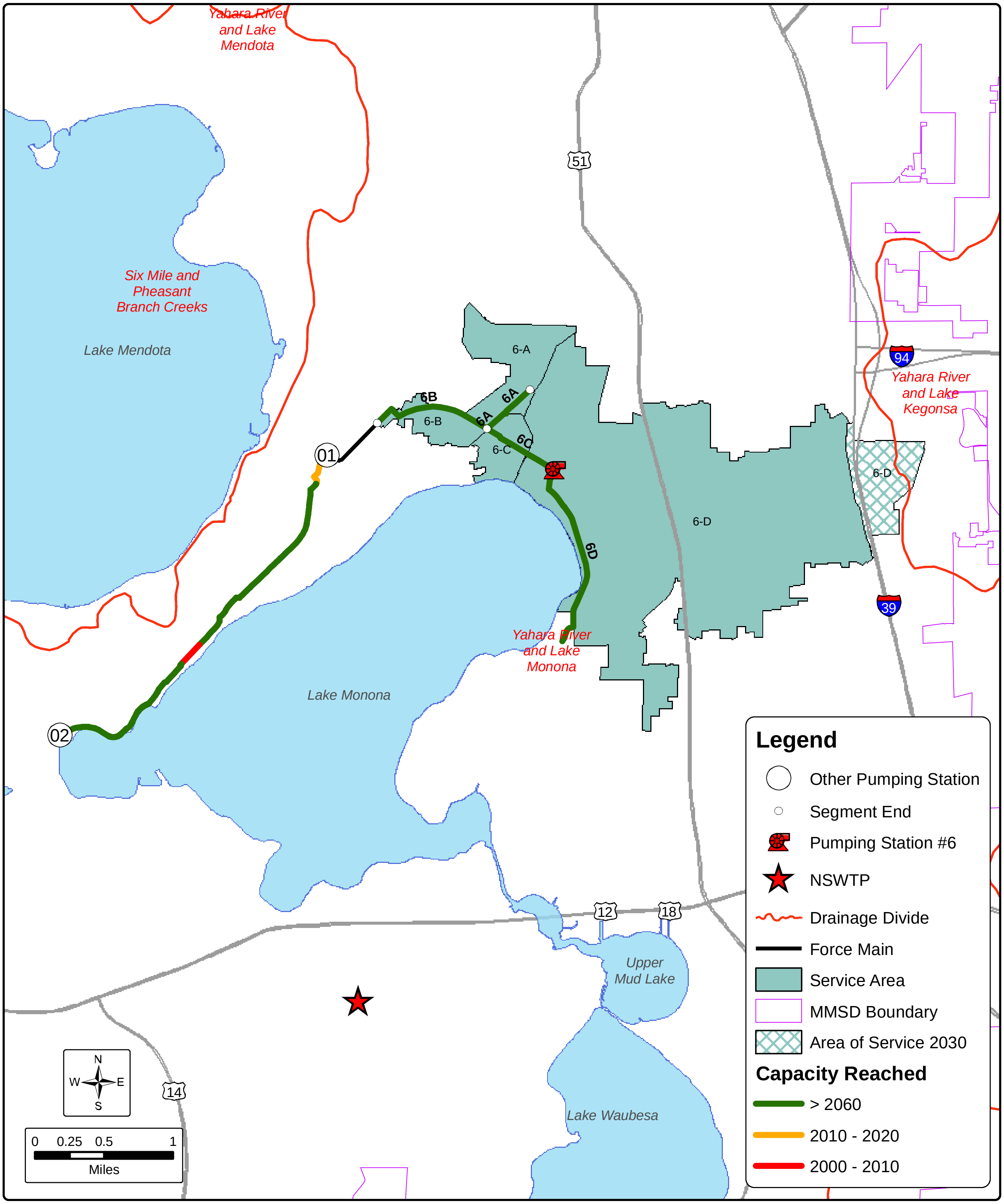
PS 6 - 402 WALTER STREET
CITY OF MADISON

CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT

STRAND ASSOCIATES, INC.® ENGINEERS

FIGURE 2.08-1
1-547.001

S:\@SAI\501--550\547\001\Data\GIS\Figures\Figure 2.08-2 PS 6 Drainage Area 8x11.mxd



PS 6 - 402 WALTER STREET
CITY OF MADISON

INTERCEPTOR AND FORCE MAIN ANALYSIS
CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT

STRAND ASSOCIATES, INC.® ENGINEERS

FIGURE 2.08-2
1-547.001

2.09 PS 7-6300 METROPOLITAN LANE, MONONA

A. Areas Served by Pumping Station

PS 7 conveys gravity drainage from adjacent sewers and pumped flows from upstream pumping stations. Figure 2.09-1 shows the location and highlights the features of this pumping station. Areas of MMSD that comprise the PS 7 gravity drainage service area include portions of the City of Monona, City of Madison, Village of McFarland, Village of Cottage Grove, and Town of Blooming Grove. Four main interceptor systems, the EI, SEI, Northeast Interceptor (NEI), and Far East Interceptor (FEI), drain to this pumping station. Figure 2.09-2 shows the area of MMSD served first by PS 7 as well as the other areas ultimately served by PS 7 including the gravity drainage areas of PSs 14, 13, 10, 9, and 6. Gravity drainage accounts for 20 percent of the average daily flow at this pumping station based on the preferred operating mode for PS 1 in 2008 (normally, PS 1 pumps most of its flow to PS 2 through the CTFM). This figure also shows the capacity needs for the gravity drainage area served by PS 7.

PS 7 receives pumped flows directly from PSs 6, 9, and 10. PSs 13, and 14 are tributary to PS 10, and their flows ultimately flow through PS 7. PS 1 is tributary to PS 6; however, most of its flow is typically pumped to PS 2. Figure 2.09-3 shows the network of pumping stations, force mains, and interceptors associated with PS 7.

B. Description of Pumping Station

1. History of Station

PS 7 is one of MMSD's largest and most critical stations. It was constructed at 6300 Metropolitan Lane in Monona and placed in service in 1950. Major rehabilitation projects occurred in 1963 and 1992. The 1963 improvements included a second 36-inch diameter force main and associated piping revisions at the pumping station in addition to some major electrical modifications. The 1992 improvements included replacing three pumps, rebuilding one pump, replacing and upgrading the electrical system, adding a new surge system, ball valves, and screens, and modifying the roof and building exterior. Other lesser modifications to the pumping station included installation of a new telemetry system in 1984 and the addition of radios for telemetry communication in 1990 (the radios were subsequently replaced in 2000). The bar screens at this pumping station were removed in 2007 after new screening facilities were constructed at Nine Springs.

2. Current Design Capacities and Limitations

Because of its location within the collection system and its proximity to NSWTP, approximately 40 percent of all wastewater in MMSD passes through PS 7 as the system was operated in 2008. The maximum pumping capacity of the station is approximately 45 mgd, whereas the firm capacity is 39 mgd. Data from the last decade shows an average flow rate range for PS 7 of 15 to 20 mgd. Current flows are at the lower end of this range since most of the flows from PS 1 have been diverted from this pumping station to PS 2.

Based on the flows projected in the 2008 MMSD Collection System Evaluation prepared by CARPC, the station has reached or will reach its firm capacity between 2005 and 2011 and will reach its maximum capacity between 2012 and 2027. Growth on Madison's east side and growth in the Waunakee and Northern Urban Service Areas will cause flows to the pumping station to increase beyond the current available capacity.

As a result of the current need for additional capacity at PS 7, MMSD has included a new PS 18 scheduled for completion by 2015 based on the 2009 MMSD Capital Projects Budget. PS 18 will provide firm capacity relief to PS 7 by intercepting flows from the NEI and potentially the SEI.

The 2008 Condition Assessment assessed the condition of all MMSD conveyance pumping stations. PS 7 was assigned an overall rating of 30.00, making its ordinal ranking (priority) 1 out of 17, or the station most in need of upgrade or modification. The actual rating for each element was as follows:

Adequacy	Rating
Maximum Flow Capacity	Poor
Firm Flow Capacity	Poor
Power System Redundancy	Adequate to Good
Mechanical Condition	Good
Building and Structural Condition	Excellent
Electrical Condition	Good to Excellent
Criticality Factor	2.00
Overall Score	30.00

3. Additional Planned Near-Term Improvements (2010-2020)

The construction of PS 18 and its associated force main is a major planned near-term improvement that would affect PS 7. The addition of this station near the existing PS 7 would both account for the anticipated increase in future flows and provide redundancy at this location in the collection system. A force main would be required between PS 18 and NSWTP and a gravity sewer would be required to connect the pumping station to either the 60-inch SEI or to the NEI. To provide greater redundancy, a connection from the new pumping station's wet well to the SEI may be more advantageous, but it is also likely to be more difficult to construct and will be more expensive since the NEI from the FEI to the SEI will have to be relieved regardless of the final connection to the pumping station.

The capacity of PS 18 has not yet been determined. It will be affected by the determination of how to serve areas in the eastern portion of the gravity drainage area (City of Madison and Village of Cottage Grove). Based on preliminary evaluations, MMSD staff would project that PS 18 would have a firm capacity similar to PS 7 (39 mgd). This would provide a firm capacity of 78 mgd or about 5.73 mgd greater than the peak flow that the 2060 high flow projections indicate.

Several projects within the service area of PS 7 were included in the MMSD 2009 Capital Projects Budget. These include:

a. NEI – FEI junction to SEI junction (MMSD Project 839-00-79)
b. FEI - Cottage Grove Extension Lining (MMSD Project 841-00-57)
c. PS 7 Improvements in Conjunction with PS 18 (MMSD Project 857-00-70)
d. PS 18 (MMSD Project 868-00-51)
e. PS 18 Force Main (MMSD Project 868-00-52)
f. PS 7 Backup Power (MMSD Project 440-00-20)

The following segments tributary to PS 7 were indicated to be at capacity prior to the year 2010:

a. 7Fi (MH 07-932 to MH 07-313)-This is a short (14-foot) section that connects the FEI with the NEI.
b. 7Fii (MH 07-313 to MH 07-215). This segment will be modified as a part of the PS 18 Project (MMSD Project 868-00-52) listed above.
c. 7Mii (07-211 to PS 7). PS 18 will provide capacity relief for this interceptor segment. This segment will function as the interconnection between PS 7 and PS 18 to allow flows to be balanced between the two stations.

The following segments tributary to PS 7 were indicated to be at capacity between the years of 2010 and 2020:

a. 7B (MH 07-437 to 07-426)
b. 7Ci (MH 07-734 to MH 07-728) High Flow Projections Only
c. 7Dii (MH 07-425 to MH 07-416) High Flow Projections Only
d. 7Ji (MH 07-249 to MH 07-242) High Flow Projections Only
e. 7Mi (MH 07-215 to MH 07-211) High Flow Projections Only

Pumping Station 18 will provide capacity relief for Segment 7Mi, like 7Mii above (pre-2010 needs). Also like 7Mii above, this segment will function as the gravity interconnection between PS 7 and PS 18 to allow flows to be balanced between the two stations.

4. Long-Term Considerations (2020-2060)

a. Pumping Station: PS 18 improvements will provide relief for the firm capacity deficit for PS 7. In addition there will be improvements scheduled for PS 7 concurrent with the PS 18 construction. Per the 2009 Capital Improvement Plan, these projects are scheduled for completion in 2014. Based on typical lives of equipment and facilities, it could be anticipated that the following rehabilitation projects would occur prior to 2060:

(1)	Roofing (25-Year Life)	2039
(2)	Electrical (25-Year Life)	2039
(3)	HVAC Equipment (20-Year Life)	2034 and 2054

b. Force Mains: The PS 7 Force Main is expected to reach its rated pressure capacity between the years 2024 and 2050. Construction of PS 18 and its force main will alleviate the need for expanded capacity. A consideration for the PS 18 Force Main would be to provide a connection between this force main and the PS 7 dual force mains to allow use of the PS 7 Force Main with PS 18 or the PS 18 Force Main with PS 7.

c. Interceptors: The following segments of interceptors upstream of PS 7 are expected to reach their capacity between 2020 and 2060.

(1) 7Ai (MH 07-955 to 07-954) High Flow Projections Only
(2) 7Cii (MH 07-728 to MH 07-723)
(3) 7Ciii (MH 07-723 to MH 07-707)

(4) 7Civ (MH 07-707 to MH 07-426)
(5) 7Di (MH 07-426 to MH 07-425) High Flow Projections Only
(6) 7Diii (MH 07-416 to MH 07-415) High Flow Projections Only
(7) 7E (MH 07-415 to MH 07-932) High Flow Projections Only
(8) 7Hii (MH 07-610 to MH 07-609)
(9) 7Jii (MH 07-242 to MH 07-231)
(10) 7Jiii (MH 07-231 to MH 07-228)
(11) 7Ki (MH 07-228 to MH 07-224) High Flow Projections Only
(12) 7Kii (MH 07-222 to MH 07-222)
(13) 7Kiii (MH 07-222 to MH 07-218) High Flow Projections Only
(14) 7Kiv (MH 07-218 to MH 07-215) High Flow Projections Only

5. PSs 1, 6, 9, 10, 13, and 14 Operational Impacts on PS 7

As can been seen in Figure 2.09-2, PSs 6, 9, and 10 pump directly to PS 7. In addition, PS 13 and PS 14 pump to PS 10, and PS 1 pumps some of its flow to PS 6 during nighttime operation. Therefore, all wastewater passing through PSs 6, 9, 10, 13, and 14 and a portion of PS 1's flow eventually arrives at PS 7. PS 10 is the largest pumping station tributary to PS 7 and has the most potential impact on the existing pumping station and future PS 18.

6. Potential Flow Diversion from PSs 7 or 18–Stoughton WWTP

Flow balancing between watersheds may prove to be an important consideration for MMSD in the future to meet watershed-related objectives. Therefore, it may become desirable to route flow to a treatment facility other than NSWTP, which could in turn provide relief to PS 7 and PS 18. One possible option for this type of diversion from PS 7 and PS 18 would be to utilize the existing Stoughton WWTP located to the southeast of the PS 9 drainage area. The following potential service areas tributary to the overall service area for PS 7 could be considered for rerouting to the Stoughton WWTP: 7B, 7H (2030), 7J (2030-2060), 9A. Figure 2.09-4 highlights these potential service areas.

Rerouting the above flows from PS 7 to the Stoughton WWTP would potentially eliminate the need for the following capacity-related projects:

a. 7B (MH 07-437 to 07-426)
b. 7Di (MH 07-426 to MH 07-425) High Flow Projections Only
c. 7Dii (MH 07-425 to MH 07-416) High Flow Projections Only
d. 7Diii (MH 07-416 to MH 07-415) High Flow Projections Only
e. 7E (MH 07-415 to MH 07-932) High Flow Projections Only
f. 7Ji (MH 07-249 to MH 07-242) High Flow Projections Only
g. 7Jii (MH 07-242 to MH 07-231)
h. 7Jiii (MH 07-231 to MH 07-228)
i. 7Ki (MH 07-228 to MH 07-224) High Flow Projections Only
j. 7Kii (MH 07-222 to MH 07-222)
k. 7Kiii (MH 07-222 to MH 07-218) High Flow Projections Only
l. 7Kiv (MH 07-218 to MH 07-215) High Flow Projections Only

These eliminated capacity improvement segments are noted in Figure 2.09-5.

7. Impact of CTFM Operation on PS 7

The 30-inch CTFM provides relief to PS 6 and PS 7. During high flow events, up to 21 mgd from PS 1 is pumped to PS 2. Prior to the replacement of the CTFM in 2003, this flow would have been pumped to PS 6 and then to PS 7. As discussed above, PS 7 now handles approximately 40 percent of all flow to the NSWTP. Utilization of the CTFM allows flow to be routed to the NSWTP via PS 2, flow that would otherwise pass through PS 7.

8. Potential Diversion from PS 7 to Mendota Treatment Plant

Flow balancing between watersheds or location within a watershed may prove to be an important consideration for MMSD in the future to meet watershed-related objectives. Therefore, it may become desirable to route flow to a treatment facility other than the NSWTP, which could in turn provide relief to PS 7 and PS 18. Another possible option for this type of diversion from PS 7 and PS 18 would be to construct a wastewater treatment plant near the Yahara River north of Lake Mendota and divert flow from PS 14 to this new facility. This diversion would also reduce the peak flows from PS 13 and PS 10 in addition to PS 7 and/or PS 18. The length of time required for obtaining approval for a Lake Mendota discharge is longer than the time available to the MMSD for the needed capacity improvements downstream of PS 14.

If constructed, this facility would only serve to relieve the NSWTP capacity need of 3 mgd of additional secondary treatment capacity by the year 2060. Technical Memo 7 (Development of Alternatives) discusses this alternative project in more detail.

9. Potential Diversion from PS 7 to Starkweather Creek WWTP

Flow balancing between watersheds or location within a watershed may prove to be an important consideration for MMSD in the future to meet watershed-related objectives. Therefore, it may become desirable to route flow to a treatment facility other than the NSWTP, which could in turn provide relief to PS 7 and PS 18. Another possible option for this type of diversion from PS 7 and PS 18 would be to construct a wastewater treatment plant near PS 13 (Truax Field) east of Lake Mendota and divert flow from PS 13 to this new facility. This diversion would also reduce the peak flows to PS 10 in addition to PS 7 and/or PS 18. The length of time required for obtaining approval for a Starkweather Creek discharge is longer than the time available to the MMSD for the needed capacity improvements downstream of PS 13.

If constructed, this facility would only serve to relieve the NSWTP capacity need of 3 mgd of additional secondary treatment capacity by the year 2060. Technical Memo 7 (Development of Alternatives) discusses this alternative project in more detail.

10. Potential Diversion from PS 7 to PS 13 and PS 14 WWTP

Flow balancing between watersheds or location within a watershed may prove to be an important consideration for MMSD in the future to meet watershed-related objectives. Therefore, it may become desirable to route flow to a treatment facility other than the NSWTP, which would in turn provide relief to PS 7 and PS 18. Another possible option for this type of diversion from PS 7 and PS 18 would be to construct a wastewater treatment plant located near PS 13 (Truax

Field) east of Lake Mendota and divert flow from PS 13 and PS 14 to this new facility. This diversion would also reduce the peak flows to PS 10 in addition to PS 7 and/or PS 18. The length of time required for obtaining approval for a PS 13 & 14 WWTP discharge is longer than the time available to the MMSD for the needed capacity improvements downstream of PS 13.

If constructed, this facility would only serve to relieve the NSWTP capacity need of 3 mgd of additional secondary treatment capacity by the year 2060. Technical Memo 7 (Development of Alternatives) discusses this alternative project in more detail.

11. Potential Diversion from PS 7 to Sun Prairie WWTP

Flow balancing between watersheds may prove to be an important consideration for MMSD in the future to meet watershed-related objectives. Therefore, it may become desirable to route flow to a treatment facility other than NSWTP, which could in turn provide relief to PS 7 and PS 18. Another possible option for this type of diversion from PS 7 and PS 18 would be to utilize the existing Sun Prairie WWTP located to the northeast of the PS 7 drainage area. The following potential service areas tributary to the overall service area for PS 7 could be considered for rerouting to the Sun Prairie WWTP: 7C (2030 to 2060) Figure 2.09-6 highlights these potential service areas.

Rerouting the flows from PS 7 to the Sun Prairie WWTP would potentially eliminate the need for the following capacity-related projects:

a. 7Ci (MH 07-734 to MH 07-728)
b. 7Cii (MH 07-728 to MH 07-723)
c. 7Ciii (MH 07-723 to MH 07-707)
d. 7Civ (MH 07-707 to MH 07-426)

These eliminated capacity improvement segments are noted in Figure 2.09-7.

12. System and Electrical Redundancy Review

The 2009 PS 7 Electrical Service upgrade included providing a redundant feed from another MG&E circuit in addition to circuits NSP 1309 and PFL 1306, which both fed the pumping station prior to the upgrade. NSP 1309 and PFL 1306 are located along Bridge Road and although from separate directions, are also located on the same pole line. The new service, NSP 1311 was routed to the station from the southeast along Metropolitan Lane and feeds an MG&E switch located near the outdoor transformer enclosure at the pumping station. The new circuit and PFL 1306 now typically feed the pumping station with NSP 1309 located at the MG&E switch, available to feed the station in the event both other feeds are lost. To switch to NSP 1309 will require a manual switching operation by MG&E.

There is no provision for standby power at Pumping Station 7; however, a portable generator set connection was installed in 2009 to facilitate connecting a portable generator in the event of a catastrophic power failure. MMSD presently has only one portable generator set with the ability to power any of the pumps at Pumping Station 7; however, that generator is the backup generator for Pumping Station 17. Rental units of the size necessary are typically available from FABCO, but generally, would not be available in a timely enough manner during an emergency event. Still, the portable generator connection is available to facilitate maintenance work and mitigate the damage that could be caused by a long-term outage.

According to the March 13, 2007, MMSD Emergency Response Manual, during a service outage there is adequate storage in the interceptors and local sewers upstream of this pump station to prevent any basement back-ups or sewer overflows for one hour under normal flow conditions and 30 minutes under high flow conditions.

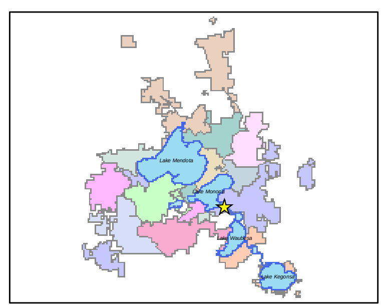
The PS 18 proposed project as well as the PS 7 improvements project will likely address the potential needs for redundant and/or standby power to serve these stations. At the present time, the Nine Springs (NSP) substation does have a manual standby generator owned by MG&E that could serve as a standby should power from the supply grid be lost to that substation and the Pflaum Road substation (PFL). It is also possible that power could temporarily be routed from the MG&E owned Femrite substation should the power serving the site from the two other substations fail.

S:\@SAI\501--550\547\001\Data\GIS\Figures\Figure 2.09-1 PS7.mxd

PS 7-Metropolitan Lane (Black Bridge)
6300 Metropolitain Lane, Monona, WI

Originally Constructed	1950
Upgrades	1992
Capacity (2008)	
Maximum (mgd)	45
Firm (mgd)	39

Pumps	Q (gpm)	H (ft)	Motor (HP)	Year On-Line
7A	10,000	47	250	1950
7B	15,200	53	250	1992
7C	19,400	59	350	1992
7D	19,400	59	350	1992



PS 7 - 6300 METROPOLITAN LANE
CITY OF MONONA

CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT

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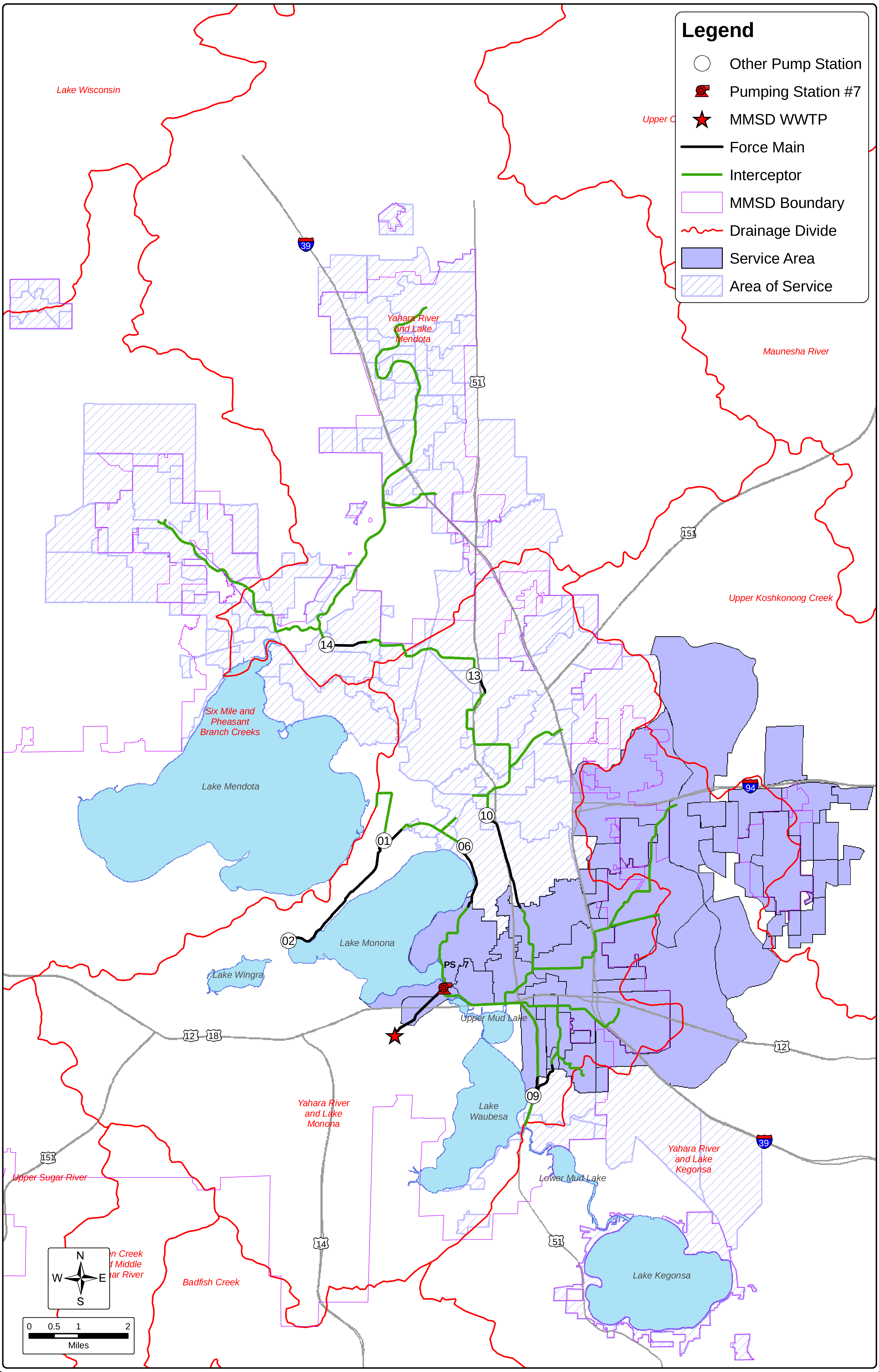
FIGURE 2.09-1
1-547.001



PS 7 - 6300 METROPOLITAN LANE
CITY OF MONONA

OVERALL SERVICE AREA
CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT

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FIGURE 2.09-2
1-547.001

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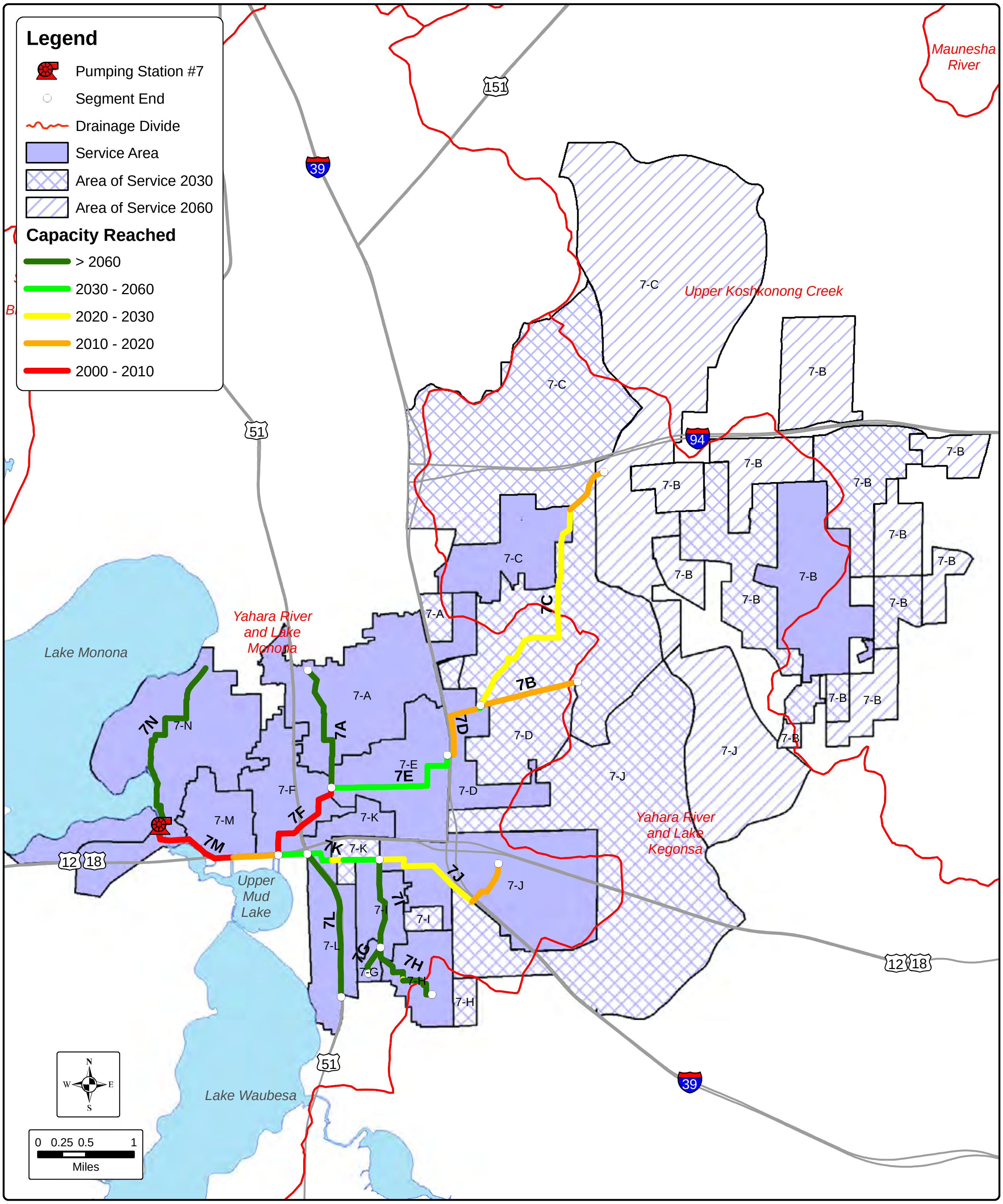

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PS 7 - 6300 METROPOLITAN LANE
CITY OF MONONA

INTERCEPTOR AND FORCE MAIN ANALYSIS
CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT

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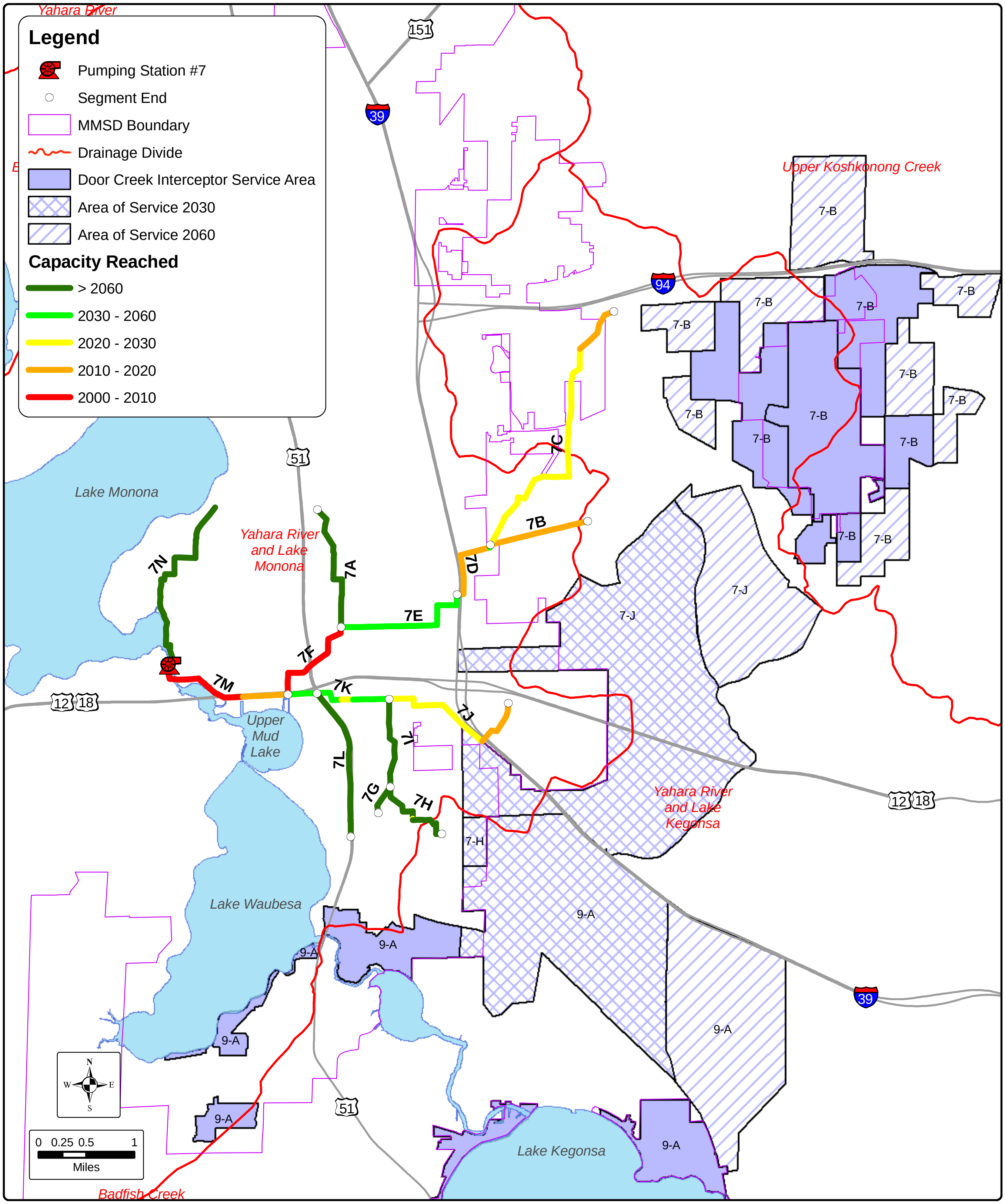
FIGURE 2.09-3
1-547.001

S:\MAD\1500--1599\1547\001\Data\GIS\Figures\Figure 2.09-4 PS 7 Drainage Area 8x11.mxd

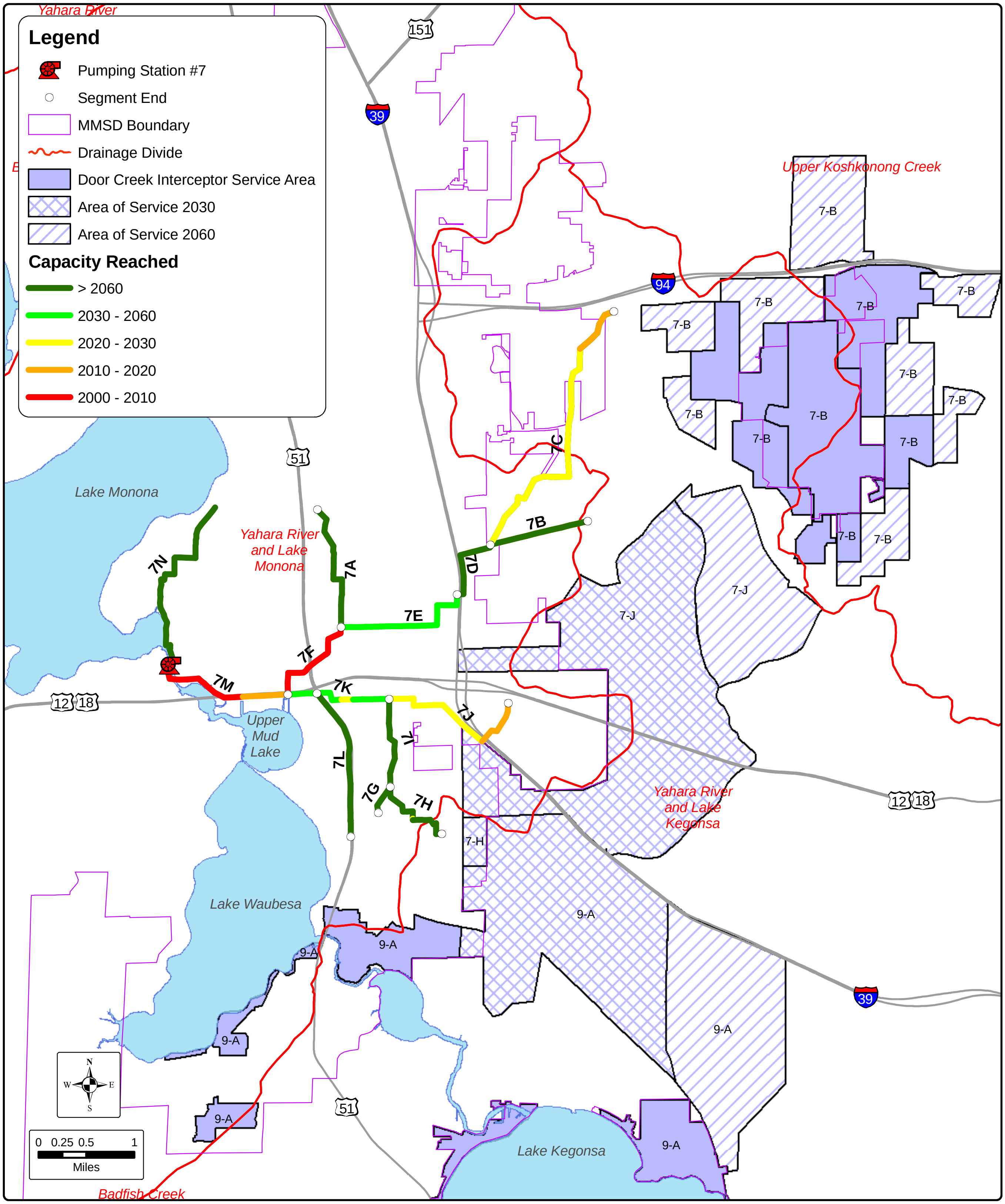


PS 7 - 6300 METROPOLITAN LANE
CITY OF MONONA

DOOR CREEK INTERCEPTOR POTENTIAL SERVICE AREA
CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT

STRAND ASSOCIATES, INC.® ENGINEERS

FIGURE 2.09-4
1-547.001



PS 7 - 6300 METROPOLITAN LANE
CITY OF MONONA

DOOR CREEK AREA INTERCEPTOR
CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT

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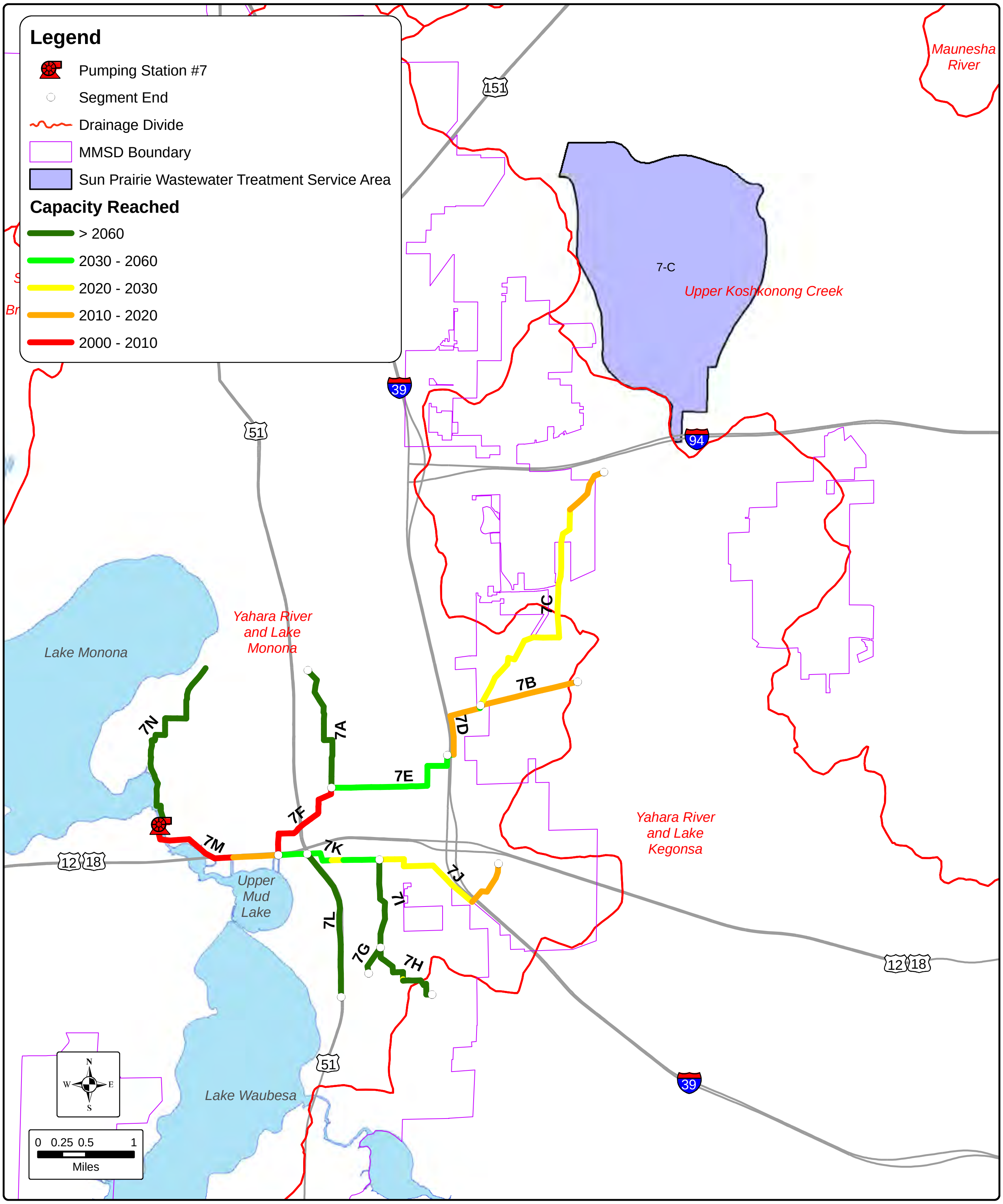
FIGURE 2.09-5
1-547.001

S:\MAD\1500--1599\1547\001\Data\GIS\Figures\Figure 2.09-6 PS 7 Drainage Area 8x11.mxd



PS 7 - 6300 METROPOLITAN LANE
CITY OF MONONA

SUN PRAIRIE WWTP POTENTIAL SERVICE AREA
CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT

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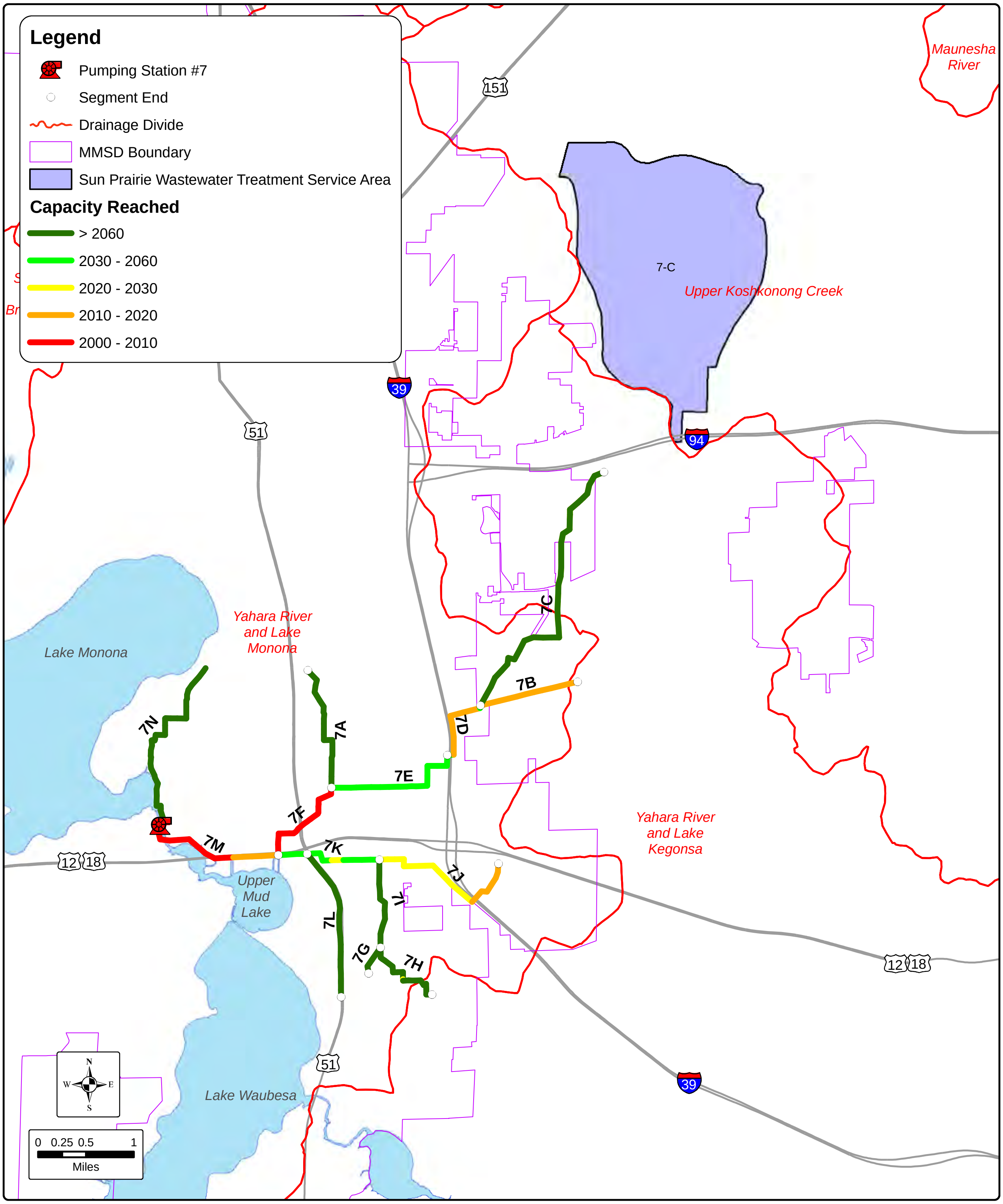
FIGURE 2.09-6
1-547.001



S:\MAD\1500--1599\1547\001\Data\GIS\Figures\Figure 2.09-7 PS 7 Drainage Area 8x11.mxd

PS 7 - 6300 METROPOLITAN LANE
CITY OF MONONA

SUN PRAIRIE WWTP
CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT

STRAND ASSOCIATES, INC.® ENGINEERS

FIGURE 2.09-7
1-547.001

2.10 PS 8–967 PLAENERT DRIVE, CITY OF MADISON

A. Areas Served by Pumping Station

PS 8 conveys both gravity drainage from adjacent sewers and pumped flows from upstream pumping stations. Figure 2.10-1 shows the location of this pumping station and highlights the features of this station. Areas of the District that comprise the PS 8 gravity drainage service area include the Village of Shorewood Hills and portions of the west side of the City of Madison and the Town of Madison. This pumping station receives flow from the SWI and the WI. Figure 2.10-2 shows the gravity service area of the District served by PS 8 as well as the gravity drainage areas of other pumping stations that ultimately discharge to PS 8 including PS 5 and PS 15. Gravity drainage accounts for 76 percent of the average daily flow (2000) at this pumping station.

PS 8 receives pumped flow from PS 5 and, under routine operation, PS 15. Provisions also exist for gravity drainage from PS 2 to PS 8, in the event that PS 2 needs relief for a high flow or emergency condition, and from the PS 16 Service Area to PS 5, which pumps into the WI and flows to PS 8. The gravity interconnection with PS 2 can also be used to relieve some of the flow to PS 8 in the event of an emergency or high flow condition. Figure 2.10-3 shows the force main and interceptor subbasins in the PS 8 gravity drainage area. The capacity assessment for interceptors and force mains for PS 8 is also shown in this figure.

B. Description of Pumping Station

1. History of Station

PS 8, located at 967 Plaenert Drive in Madison, was constructed and placed into service in 1964. This pumping station has undergone several minor repair projects since its original construction. In 1995, a bar screen was replaced with a channel grinder, which was subsequently removed in 2004. In 2000, the roof was replaced and modifications to the power system were made. These modifications included a new underground service from MG&E.

Improvements to PS 8 for condition rehabilitation and an increase in capacity are in progress and should be complete by 2010. The improvements include electrical system replacement and upgrade, a new HVAC system, and an upgrade of the overall hydraulic capacity using rebuilt pumps (C and D) from both PS 8 and PS 6. No screening or comminution facilities are included in this station although the two influent channels could again be used for this purpose if there is a future need.

2. Current Design Capacities and Limitations

The firm pumping capacity of PS 8 after the current upgrade will be 34 mgd, whereas the maximum capacity will be 34.1 mgd. Data from the last decade shows an average flow rate for PS 8 of approximately 8 mgd.

The 2008 Condition Assessment assessed the condition of all MMSD conveyance pumping stations. PS 8, based on the current upgrade, was assigned an overall rating of 11.40, making its ordinal ranking (priority) 11 out of 17. The actual rating for each element was as follows:

Adequacy	Rating
Maximum Flow Capacity	Excellent
Firm Flow Capacity	Excellent
Power System Redundancy	Excellent
Mechanical Condition	Excellent
Building and Structural Condition	Excellent
Electrical Condition	Excellent
Criticality Factor	1.90
Overall Score	11.40

The PS 8 service area is largely developed, although population density may increase over time. Thus, the increase in flow to PS 8 from within its service area will likely be minimal. Anticipated growth on Madison's west side in the PS 15 service area, however, will increase flows to PS 8.

Four phases of improvements of the WI were scheduled prior to 2010. These improvements provided both capacity relief and relief because of the physical condition of the sewers. Work has been completed from the intersection of Randall Avenue and Dayton Street to the Walnut Street overpass on Campus Drive. Design of the relief sewers west of this point is ongoing.

The SWI South Leg and North Leg sewers were relined in 2007. An evaluation of their capacity indicates that although some capacity deficit may exist in these sewers prior to 2060, that capacity deficit is likely to be minimal and may not require attention during the planning period. Ownership of these sewers and the SWI from their junction down to a location near the Vilas Zoo may at some point be transferred from MMSD to the City of Madison since the sewers serve the City of Madison almost exclusively.

3. Additional Near-Term Planned Improvements (2010-2020)

The following segments tributary to PS 8 were indicated to be at capacity prior to the year 2010:

a. 8Aiii (MH 02-545 to MH 02-238)
b. 8Aiv (MH 02-538 to MH 02-536)
c. 8Di (MH 02-531A to 02-519)
d. 8Diii (MH 02-518 to MH 20-516)
e. 8Div (MH 08-215 to MH 08-228
f. 8Ei

The following segments tributary to PS 8 were indicated to be at capacity between the years of 2010 and 2020:

a. 8Av (MH 02-536 to MH 02-535) – High Flow Projections Only
b. 8Avi (MH 02-535 to MH 02-532) – High Projections Flow Only
c. 8Iii (MH 02-238 to MH 02-234) – High Flow Projections Only
d. 8Ji (MH 02-232 to MH 02-513) – High Flow Projections Only

None of the effluent diversion alternatives would impact this pumping station since the station only receives gravity drainage from its service area, almost all of which lies in the Yahara River Basin. Two small areas are outside of the Yahara River Basin, an area in the Sugar River Basin and one in the Black Earth Creek Basin (Wisconsin River Basin), tributary to PS 15. No diversion alternatives to the Wisconsin River Basin are under consideration.

4. Long-Term Considerations (2020-2060)

a. Pumping Station: No additional capacity improvements are required for this pumping station prior to the year 2060. A major rehabilitation project at this pumping station is currently under construction. Based on typical lives of equipment and facilities, it is anticipated that the following rehabilitation projects will occur prior to 2060:

(1)	Roofing (25-Year Life)	2035 and 2060
(2)	Electrical (25-Year Life)	2035 and 2060
(3)	HVAC Equipment (20-Year Life)	2030 and 2050

b. Force Mains: The force main is not expected to reach its capacity prior to the year 2060.

c. Interceptors: The following segments of interceptors upstream of PS 8 are expected to reach their capacity between 2020 and 2060.

(1) 8C (MH 02-531I to MH 02-531A)
(2) 8Ii (MH 02-041 to MH 02-038) – High Flow Projections Only
(3) 8Jii (MH 02-032 to MH 02-513)
(4) 8R (02-150 to 02-145) – High Flow Projections Only
(5) 8Sii (MH 02-142 to MH 02-136) – High Flow Projections Only
(6) 8Xiii (MH 08-121 to MH 08-120)

5. PS 5 and PS 15 Operational Impacts on PS 8

As can been seen in Figure 2.10-2, PS 5 and PS 15 pump wastewater to gravity interceptors that ultimately flow to PS 8. Therefore, under the current preferred operating mode, all wastewater from the PS 5 service area must also be pumped by PS 8. Additionally, PS 15 may be allowed to pump directly to PS 16, although at present this is not the station’s routine operating mode. The projected increase in average daily flow from PS 5 and PS 15 will not stress the firm capacity of PS 8, which will be upgraded in the current project.

6. Impact of Gravity Diversion from PS 2 to PS 8 or from PS 8 to PS 2

PS 2 and PS 8 are both centrally located within the City of Madison, near in proximity to each other, and near the NSWTP. Both pumping stations also have tributary pumping stations, and both pump directly to the NSWTP through two independent force mains. Consequently, the gravity diversion sewer connecting PS 2 and PS 8 can be advantageous in terms of routing flow through these important pumping stations. Should a condition develop where one pumping station is experiencing high flows and the other is not, the gravity diversion could relieve the stressed pumping station by balancing out the overall flow to both pumping stations. Heterogeneous rainfall patterns (which are common), nonuniform water use, upstream pumping station operation, pumping station outages, and force main breaks all could contribute to a situation in which the PS 8-PS 2 gravity diversion would be of use.

Separate gravity sewers link PS 2 and PS 8. A portion of the SWI along Haywood Street, which normally conveys wastewater to PS 2, can deliver wastewater from PS 2 to PS 8 should enough differential head develop to drive the flow backwards through the interceptor. At present, the diversion capacity of this sewer under high flows is approximately 6 mgd.

The same portion of the SWI along Haywood Street described above, from MH 8-106 to PS 2, can be used as a diversion from PS 8 to PS 2. This diversion, performed by allowing the wet well level to rise sufficiently in PS 8, can be used during an outage of PS 8 or during force main repairs. At present, the diversion capacity of this sewer under high head is approximately 5.6 mgd.

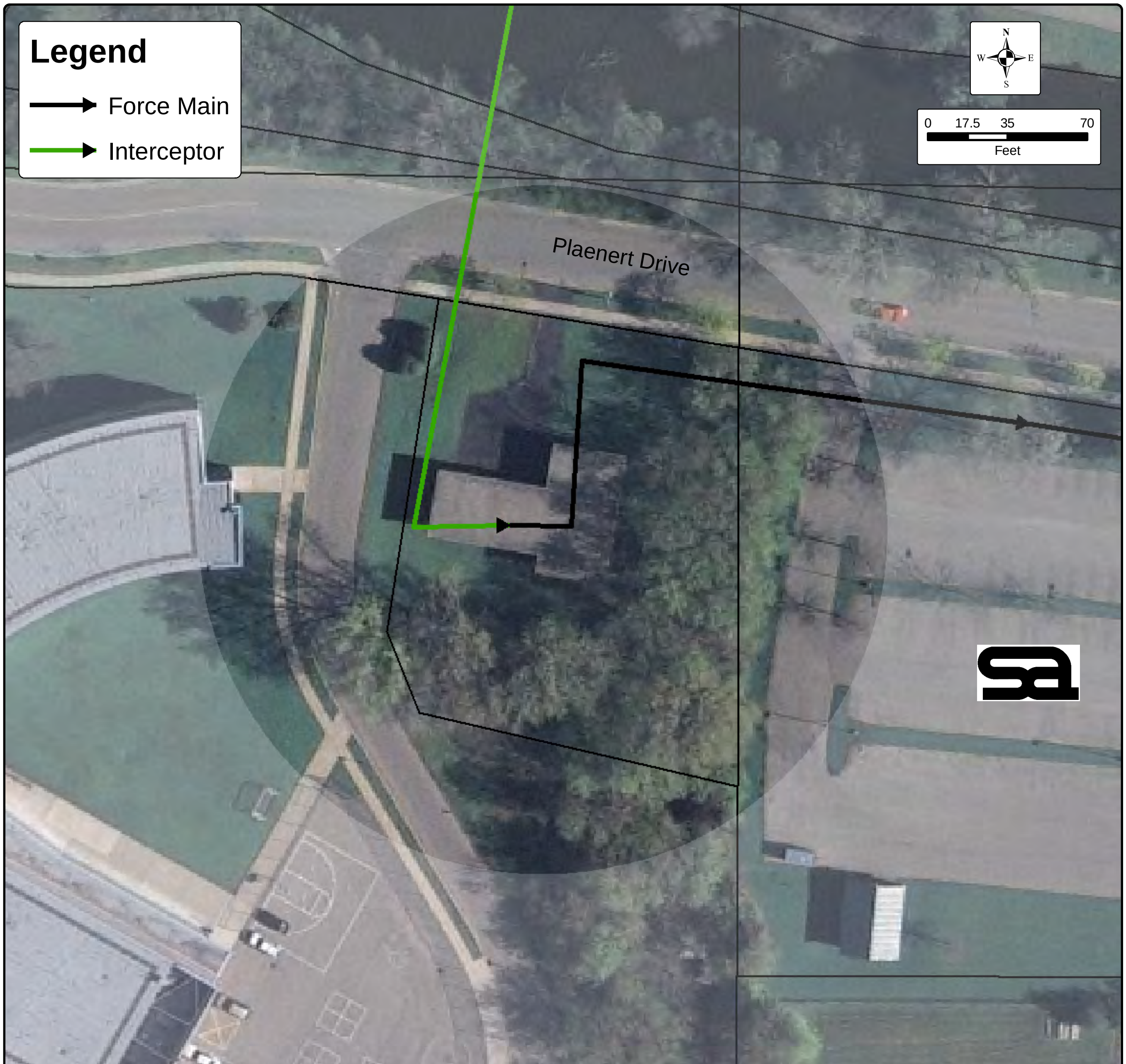
7. System and Electrical Redundancy Review

The 2010 PS upgrade included providing redundant feeds from two circuits, WGA 1313 and WGA 1316, from opposite busses of a single substation, MG&E's Wingra substation. The routing of the feeds to the pumping station is from two separate directions, one being along Fish Hatchery Road and the second from Park Street and then westerly along Plaenert Drive. MG&E can also switch power from other areas in the event of an extended outage of the entire Wingra substation.

There is no current provision for standby power at this station; however, a provision, a circuit breaker with a keyed interlock, was included in the main switchgear for a future generator connection should an on-site generator be deemed necessary. Although a portable generator connection has not been installed at PS 8, MMSD does have a portable generator large enough to power any single pump at PS 8. In normal use, however, that portable generator is the backup on-site generator for PS 17. The portable generator would have to be hardwired into the drive or starters for the pumps. As with any time a generator is used, extreme caution must be taken to prevent any possible back-feeding of power into the utility's system.

According to the March 13, 2007, MMSD Emergency Response Manual, during a service outage there is adequate storage in the interceptors and local sewers upstream of this pump station to prevent any basement back-ups or sewer overflows for two hours under normal flow conditions and one hour under high flow conditions. Flows of up to 5.6 mgd could be routed through PS 2 provided capacity is available at this pumping station.

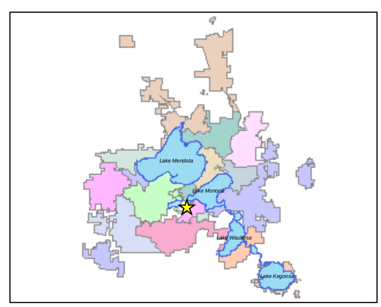
S:\@SAI\501--550\547\001\Data\GIS\Figures\Figure 2.10-1 PS8.mxd



PS 8-Plaenert
967 Plaenert Drive, Madison

Originally Constructed	1964
Upgrades	2009
Capacity (2008)	
Maximum (mgd)	34.1
Firm (mgd)	34

Pumps	Q (gpm)	H (ft)	Motor (HP)	Year On-Line
8A	12,800	58	250	2009
8B	12,800	58	250	2009
8C	13,900	60	300	2009
8D	13,900	60	300	2009



PS 8 - 967 PLAENERT DRIVE
CITY OF MADISON

CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT

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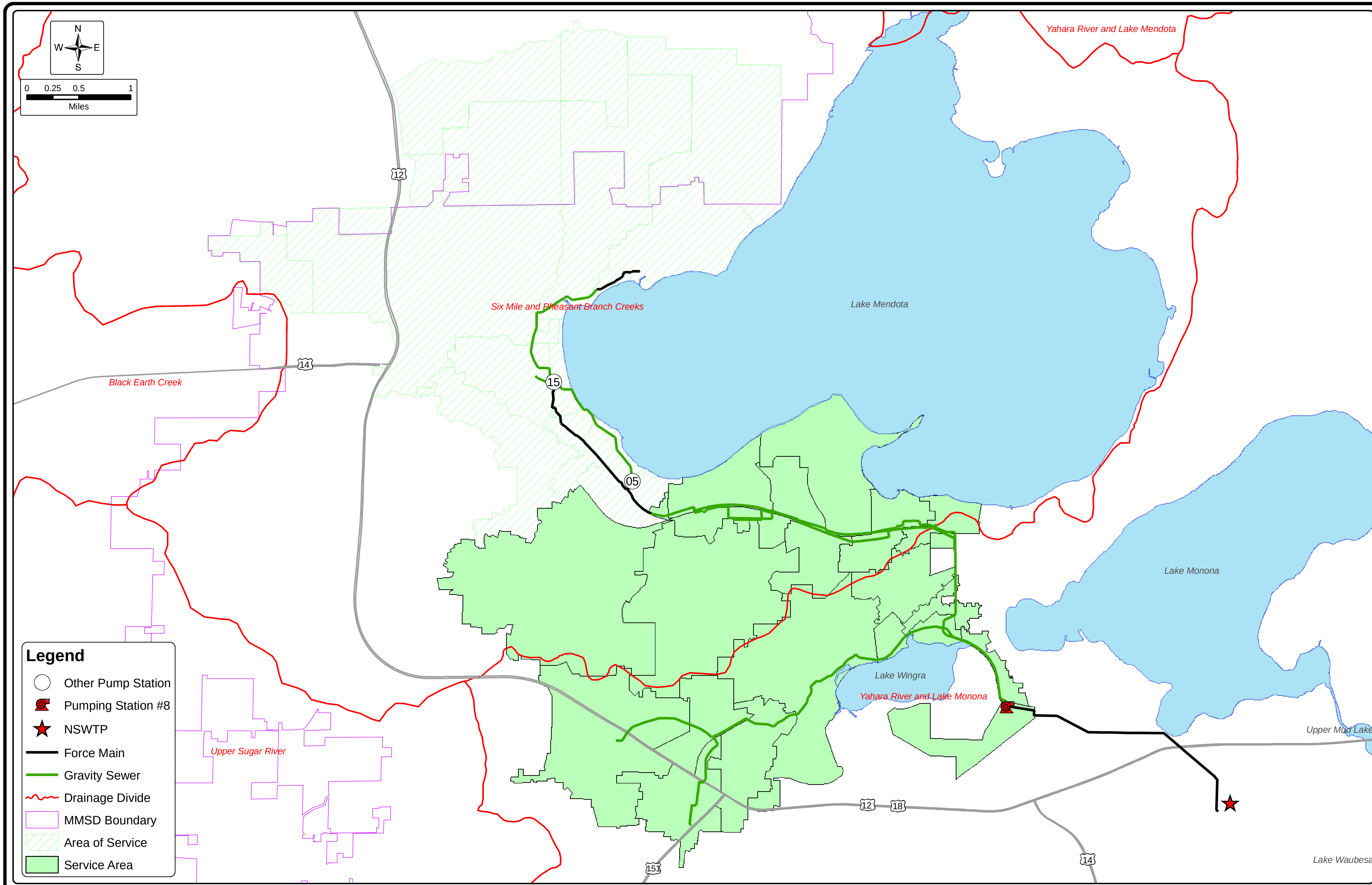
FIGURE 2.10-1
1-547.001



PS 8 - 967 PLAENERT DRIVE
CITY OF MADISON

OVERALL SERVICE AREA
CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT

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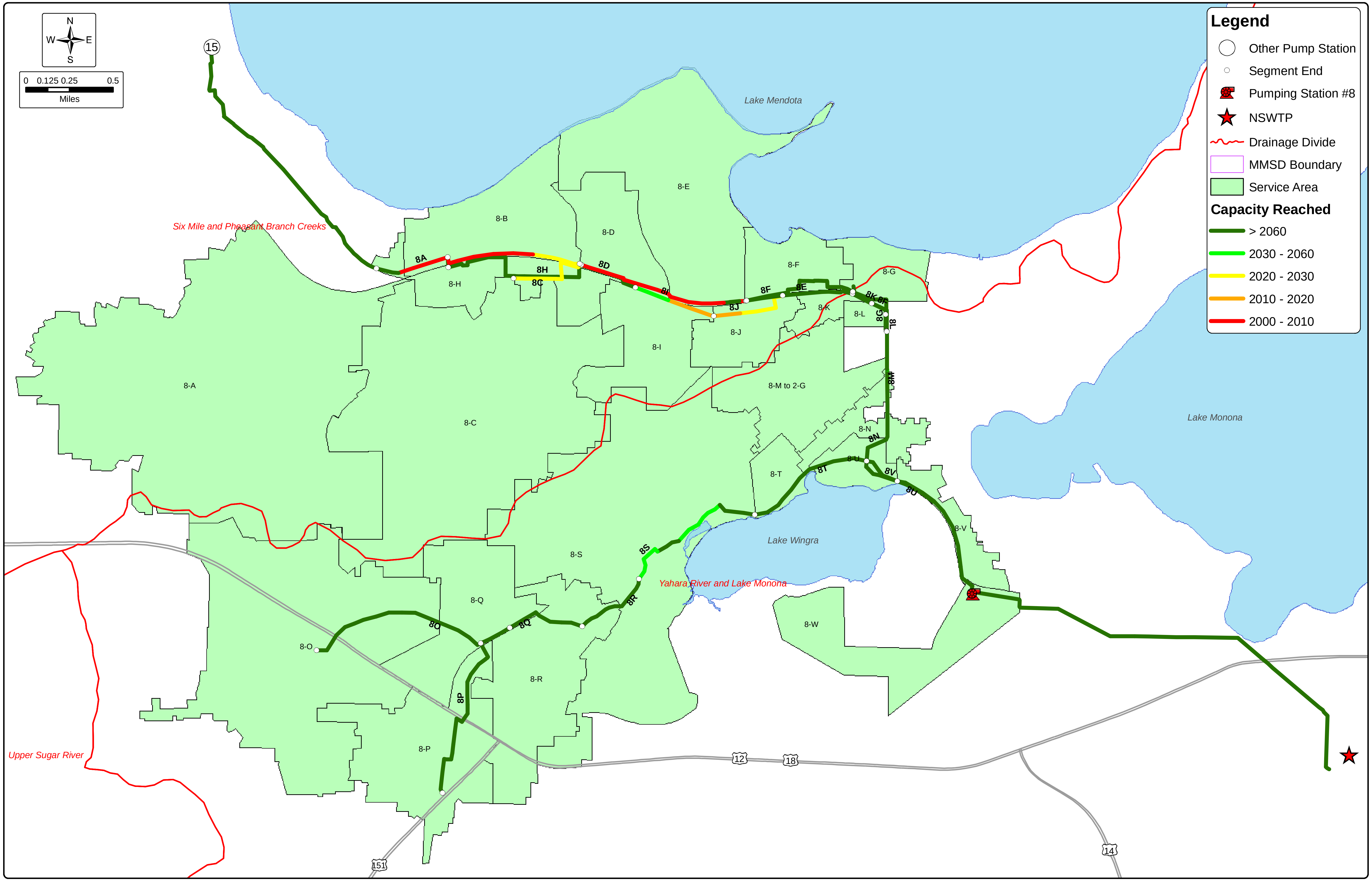
FIGURE 2.10-2
1-547.001

S:\@SAI\501--550\547\001\Data\GIS\Figures\Figure 2.10-2 PS 8 Drainage Area 11X17.mxd



PS 8 - 967 PLAENERT DRIVE
CITY OF MADISON

INTERCEPTOR AND FORCE MAIN ANALYSIS
CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT

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FIGURE 2.10-3
1-547.001

S:\@SAI\501--550\547\001\Data\GIS\Figures\Figure 2.10-3 PS 8 Drainage Area 11X17.mxd

2.11 PS 9-4612 LARSEN BEACH ROAD, MCFARLAND

A. Areas Served by Pumping Station

PS 9 conveys both gravity drainage from adjacent sewers and pumped flows from upstream pumping stations. Figure 2.11-1 shows the location of this pumping station and highlights the features of this station. Areas of the District that comprise the PS 9 service area include the Village of McFarland, the Town of Pleasant Springs, the Town of Dunn, and the Kegonsa Sanitary District. This pumping station is on the SEI. Specifically, it collects drainage from the Highway 51 Leg of the SEI and can pump both to the Highway 51 Leg and the McFarland Relief Leg. In normal operation, it pumps into the McFarland Relief Leg. Figure 2.11-2 shows the area of the District served first by PS 9 as well as the capacity analysis for the interceptors tributary to PS 9. PS 9 receives pumped flows from the Kegonsa Sanitary District and Town of Dunn Sanitary District 3 pump stations.

B. Description of Pumping Station

1. History of Station

PS 9, located at 4612 Larsen Beach Road in McFarland, was constructed and placed into service in 1963. Modifications to this station include:

a. Installation of a new telemetry system in 1984.
b. Radio for telemetry system in 1990 (replaced in 2000).
c. Roof replacement in 1997.
d. New Pump C with motor in 2002.
e. New Pump A with motor in 2004.
f. A major electrical and control system replacement in 2004. (This included installation of redundant power feeds.)
g. New Pump B with motor in 2007.

2. Current Design Capacities and Limitations

The maximum pumping capacity of PS 9, which is equal to the station's firm capacity, is approximately 4.5 mgd. Data from the last decade shows an average flow rate for PS 9 of approximately 0.8 mgd. As can be seen by these numbers, the average flow falls well below the station's firm capacity.

The 2008 Condition Assessment gave this pumping station an overall rating of 7.35, making its ordinal ranking (priority) the sixteenth highest priority among the District's 17 pumping stations. The rating for each pumping station element was as follows:

Adequacy	Rating
Maximum Flow Capacity	Excellent
Firm Flow Capacity	Excellent
Power System Redundancy	Excellent
Mechanical Condition	Excellent

Building and Structural Condition	Good
Electrical Condition	Excellent
Criticality Factor	1.05
Overall Score	7.35

3. Additional Near-Term Planned Improvements (2010-2020)

MMSD in its 2009 Capital Improvement Plan identified a potential project, rehabilitation of PSs 3, 4, and 9 (MMSD project 843-00-50), scheduled to begin in the year 2017 with completion scheduled for 2018. This project would address any current deficiencies in PS 9 including the need, if any, for additional firm capacity.

There are no capacity-related needs for PS 9, interceptors tributary to PS 9 or the PS 9, force main prior to the year 2020.

4. Long-Term Considerations (2020-2060)

a. Pumping Station: The planned PS rehabilitation project (2018) will address all deficiencies at this station. If the plan updates the roofing, electrical, and HVAC based on typical lives of equipment and facilities, it could be anticipated the following rehabilitation projects would occur prior to 2060:

(1) Roofing (25-Year Life) 2043
(2) Electrical (25-Year Life) 2043
(3) HVAC Equipment (20-Year Life) 2038 and 2058

Based upon the high flow projections only, there will be a need to expand capacity in about 2022. It is assumed that this deficit would be addressed at the time of the pumping station rehabilitation project. The projected 2060 firm pumping capacity deficit is 1.89 mgd.

b. Force Mains: A short segment (14 feet) of force main is at the nominal 8 fps rating in the year 2000. The remainder of the force main does not have any capacity needs prior to 2060.

c. Interceptors: The following segments of interceptors upstream of PS 9 are expected to reach their capacity between 2020 and 2060.

(1) 9A (MH09-108 to MH09-104) – High Flow Projections Only
(2) 9Bi (MH 09-104 to MH 09-101) – High Flow Projections Only
(3) 9Bii (MH 09-101 to PS 9)

5. Kegonsa Sanitary District and Town of Dunn Sanitary District 3 Pumping Stations Operational Impacts on PS 9

The Kegonsa Sanitary District and Town of Dunn Sanitary District 3 Pumping Stations, which are not owned or operated by MMSD, pump to PS 9. PS 9 is projected to reach its firm pumping capacity by the year 2022 only under high flow projections. Rerouting flow from the Kegonsa Sanitary District and Pleasant Springs Sanitary District to the Stoughton WWTP, discussed below, could potentially result in the Kegonsa Sanitary District and Pleasant Springs Pumping Stations pumping directly to the Stoughton WWTP, which would eliminate any firm capacity deficit at PS 9.

6. Potential Flow Diversion from PSs 7 or 18–Stoughton WWTP

Flow balancing between watersheds may prove to be an important consideration for MMSD in the future to meet watershed-related objectives. Therefore, it may become desirable to route flow to a treatment facility other than NSWTP, which would in turn provide relief to PS 9. One possible option for this type of diversion from PS 9 would be to utilize the existing Stoughton WWTP located to the southeast of the PS 9 drainage area.

Rerouting flows from service area 9A would potentially eliminate the need for the following capacity related projects:

a. 9A (MH09-108 to MH09-104) High Flow Projections Only
b. 9Bi (MH 09-104 to MH 09-101) High Flow Projections Only
c. 9Biii (MH 09-101 to PS 9) High Flow Projections Only
d. PS 9

These eliminated capacity improvement segments are shown on Figure 2.11-2.

7. System and Electrical Redundancy Review

PS 9 is served from two separate feeds from Alliant Energy (CODN 7523 and MCFN 1112). This facility also has provisions for a portable generator connection for standby power. MMSD's portable generators are capable of powering any of the pumps at this site.

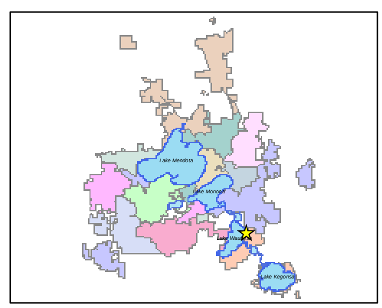
According to the March 13, 2007, MMSD Emergency Response Manual, during a service outage there is adequate storage in the interceptors and local sewers upstream of this pump station to prevent any basement back-ups or sewer overflows for two and one-half hours under normal flow conditions and one hour under high flow conditions.

S:\@SAI\501--550\547\001\Data\GIS\Figures\Figure 2.11-1 PS9.mxd

PS 9-McFarland
4612 Larson Beach Road, McFarland

Originally Constructed	1962
Upgrades	1997
Capacity (2008)	
Maximum (mgd)	4.5
Firm (mgd)	4.5

Pumps	Q (gpm)	H (ft)	Motor (HP)	Year On-Line
9A	2,300	51	40	2003
9B	2,300	51	40	2007
9C	2,300	51	40	2002

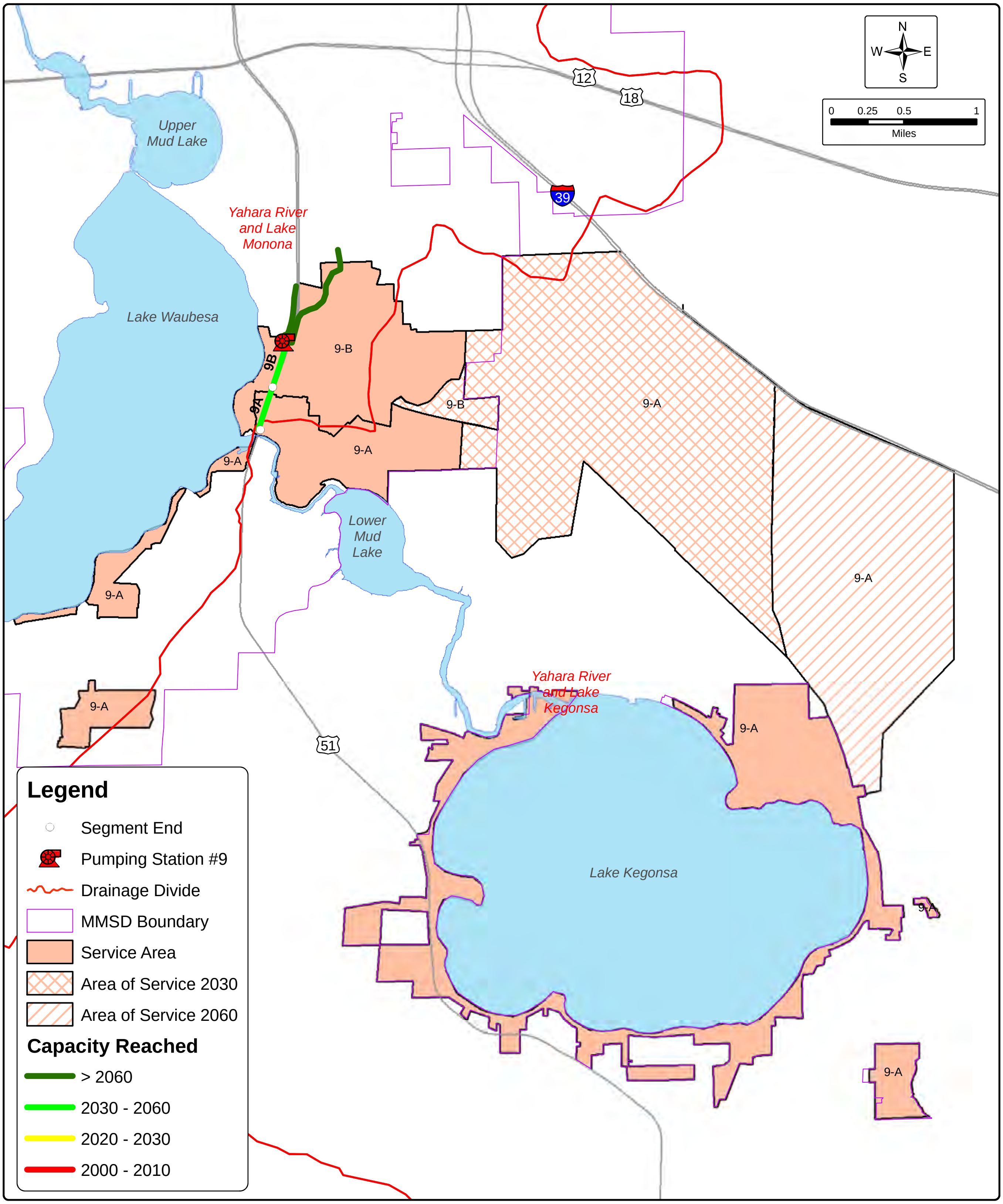


PS 9 - 4612 LARSON BEACH ROAD
VILLAGE OF MCFARLAND

CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT

STRAND ASSOCIATES, INC.® ENGINEERS

FIGURE 2.11-1
1-547.001



PS 9 - 4612 LARSEN BEACH ROAD
VILLAGE OF MCFARLAND

INTERCEPTOR AND FORCE MAIN ANALYSIS
CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT

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FIGURE 2.11-2
1-547.001

2.12 PS 10-110 REGAS ROAD (MAIN POST OFFICE), MADISON

A. Areas Served by Pumping Station

PS 10 conveys gravity drainage from adjacent sewers and pumped flows from upstream pumping stations. Figure 2.12-1 shows the location of this pumping station and highlights some of its features. Areas of the District that comprise the PS 10 service area include a portion of the east side of the City of Madison, Utility District 10 of the Town of Blooming Grove, and Utility Districts 2 and 6 of the Town of Burke. This pumping station is on the NEI, and therefore both collects drainage from the interceptor and pumps to it. Figure 2.12-2 shows the area of the District served first by PS 10 as well as PS 14 and PS 13, which ultimately discharges to PS 10. Gravity drainage accounts for 42 percent of the average daily flow at this pumping station.

PS 10 receives pumped flows from PS 13 through the NEI. PS 13 receives pumped flows from PS 14. All of the areas served by PS 13 and 14 ultimately flow through PS 10. Figure 2.12-3 shows the network of pumping stations, force mains, and interceptors associated with PS 10.

B. Description of Pumping Station

1. History of Station

PS 10, located at 110 Regas Road in Madison, was constructed and placed into service in 1965. Improvements to this station include:

a. Installation of a new telemetry system in 1984.
b. The replacement of the smallest original pump with a new pump in 1987 to add capacity. Modifications to the control system in 1990.
c. Radio for the telemetry system in 1990 (replaced in 2000)
d. Roof replacement in 1998.
e. Major upgrade and rehabilitation project in 2004 included new pumps, motors and variable frequency drives that increased firm pumping capacity, replaced electrical power supply and controls, improved power redundancy, and upgraded the station HVAC.

2. Current Design Capacities and Limitations

The firm pumping capacity of PS 10 with the 2004 upgrade is 42 mgd. Data from 1996 through 2007 shows an average flow rate range for PS 10 of 8 to 11 mgd with an overall average of 9.4 mgd. The present capacity will provide sufficient firm pumping capacity in this station for the entire master planning period of 2010 through 2060.

The 2008 Condition Assessment gave this pumping station an overall rating of 10.40, making its ordinal ranking (priority) the thirteenth highest priority among the District's 17 pumping stations.. The rating for each pumping station element was as follows:

Adequacy	Rating
Maximum Flow Capacity	Excellent
Firm Flow Capacity	Excellent
Power System Redundancy	Excellent
Mechanical Condition	Good to Excellent
Building and Structural Condition	Excellent
Electrical Condition	Excellent
Criticality Factor	1.60
Overall Score	10.40

The following interceptor segments are projected to be at capacity prior to year 2010:

a. 10Bi (MH 10-121 to MH 10-118)
b. 10Bii (MH 10-118 to MH 10-201)
c. 10Ei (MH 10-201 to MH 10-105)
d. 10Eii (MH 10-105 to MH 10-104A)
e. 10Eiii (MH 10-104A to 10-102 A)
f. 10G (MH 10-102A to MH 10-101)
g. 10H (MH 10-101 to PS 10)

As a result of these capacity needs and based on the condition of the PS 10 interceptor segments, MMSD in its 2009 Capital Improvement Plan (MMSD Project 839-00-78) identified a project to provide relief for these sewer segments.

3. Additional Near-Term Planned Improvements (2010-2020)

The following segments tributary to PS 10 were indicated to be at capacity between the years of 2010 and 2020:

a. 10A (MH10-145 to MH 10-121) – High Flow Projections Only

4. Long-Term Considerations (2020-2060)

a. Pumping Station: No additional capacity improvements are required for this pumping station prior to the year 2060. The pumping station's last major rehabilitation occurred in 2004. Based on typical lives of equipment and facilities, it is anticipated the following rehabilitation projects would occur prior to 2060:

(1)	Roofing (25-Year Life)	2029 and 2054
(2)	Electrical (25-Year Life)	2029 and 2054
(3)	HVAC Equipment (20-Year Life)	2024 and 2044

b. Force Mains: Unless flows are reduced through the construction of satellite plants, the force main for PS 10 is projected to reach its capacity about the year 2040 only under high flow projections.

c. Interceptors: No additional interceptor segments are anticipated to have capacity needs between 2020 and 2060 since the capacity needs would have been met prior to the year 2020.

5. PS 13 and PS 14 Operational Impacts on PS 10

As shown in Figure 2.12-2, PS 13 pumps directly to PS 10, and PS 14 pumps to PS 13. Therefore, all wastewater passing through PS 13 and PS 14 eventually arrives at PS 10. The firm and maximum capacities of PS 13 are 20.0 mgd and 20.2 mgd respectively, which are less than the firm pumping capacity of PS 10. Addition of the peak hourly flow from the gravity drainage service area will not exceed the firm capacity of PS 10 in the year 2010 even if PS 13 were to operate at maximum flow for an extended period of time.

6. Potential Options for Rerouting Flow from PS 14 to Mendota WWTP, Starkweather Creek WWTP or PS 13 and PS 14 WWTP

Flow balancing between watersheds may prove to be an important consideration for the District in the future to meet watershed-related objectives. Therefore, it may become desirable to route flow from the PS 14 service area and/or PS 13 service area to a treatment facility other than NSWTP, which could in turn provide relief to PS 13 and PS 10. One option for this type of diversion from PS 13 would be to construct a Mendota WWTP on the north side of Lake Mendota. Flow from PS 14 would then be rerouted to the Mendota WWTP instead of NSWTP, thereby returning treated effluent closer to its point of origin. Other options for this type of diversion from PS 10 would be to construct either a Starkweather Creek WWTP or a PS 13 and PS 14 WWTP.

7. Potential Impacts for a Link Between PS 6 and PS 10

One potential future improvement that would affect PS 10 is to link PS 6 and PS 10. This could provide relief to PS 10 during high flows and is economically attractive because of the proximity of PS 10 and PS 6. Use of the CTFM to divert PS 1 flow away from PS 6, and therefore PS 7, has provided more capacity at PS 6, allowing it to potentially convey flow diverted there if a link from PS 10 were constructed. Such a link, however, would not currently provide relief to PS 7, because PS 6 is also tributary to PS 7. Once PS 18 is constructed and placed in service, the ability to route flows from PS 10 to PS 6 may be desirable to take advantage of the potential available capacity at PS 7. This force main or gravity connector (which could allow transfer from PS 10 to 6 or 6 to 10) could be sized to convey normal flows from PS 10 to PS 6 and hence to PS 7, allowing maintenance of PS 10 or PS 18. The same capability in the opposite direction could allow maintenance of PS 6 or PS 7.

8. System and Electrical Redundancy Review

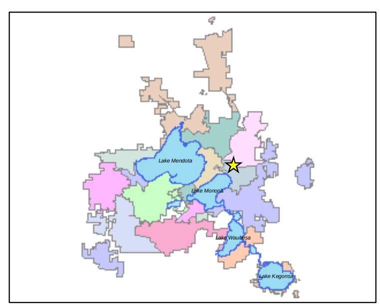
PS 10 is served from two separate feeds from separate MG&E substations (RVS 1312 and RKN 1338). PS 10 does not have permanent standby power (no onsite generator). Additionally, MMSD does not presently have a portable generator large enough to power any of the pumps at PS10. However, a circuit breaker location for a future generator was included in the switchgear (2004 upgrade) for use if standby power is at some point deemed necessary.

According to the March 13, 2007, MMSD Emergency Response Manual, during a service outage there is adequate storage in the interceptors and local sewers upstream of this pump station to prevent any basement back-ups or sewer overflows for two and one-half hours under normal flow conditions and one hour under high flow conditions.

S:\@SAI\501--550\547\001\Data\GIS\Figures\Figure 2.12-1 PS10.mxd

PS 10-Regas Road
110 Regas Road, Madison

Originally Constructed				1965
Upgrades				2005
Capacity (2008)				
Maximum (mgd)				42.2
Firm (mgd)				42.2
Pumps	Q (gpm)	H (ft)	Motor (HP)	Year On-Line
10A	18,900	94	600	2005
10B	18,900	94	600	2005
10C	18,900	94	600	2005



PS 10 - 110 REGAS ROAD
CITY OF MADISON

CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT

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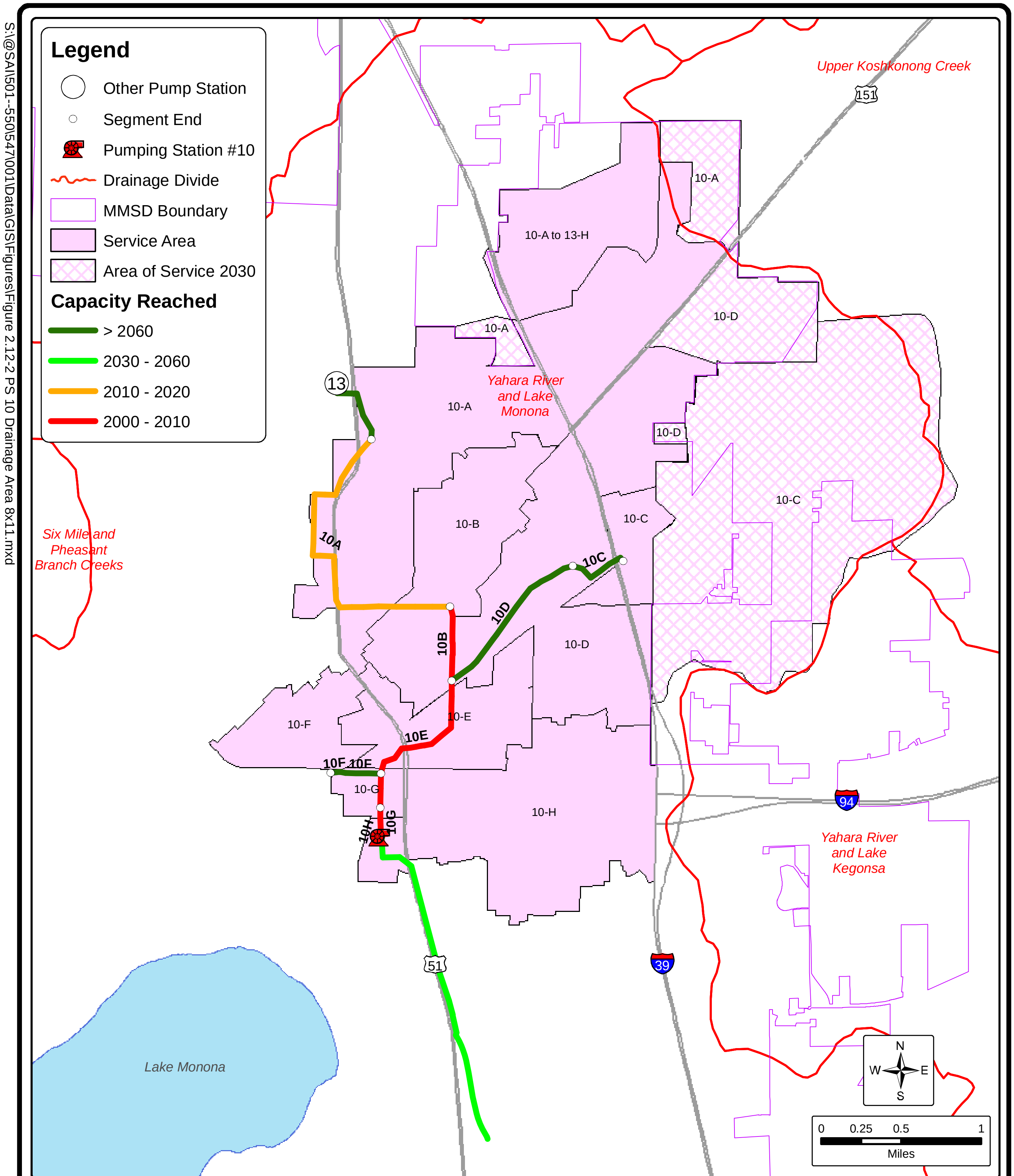
FIGURE 2.12-1
1-547.001



PS 10 - 192 REGAS ROAD
CITY OF MADISON

INTERCEPTOR AND FORCE MAIN ANALYSIS
CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT

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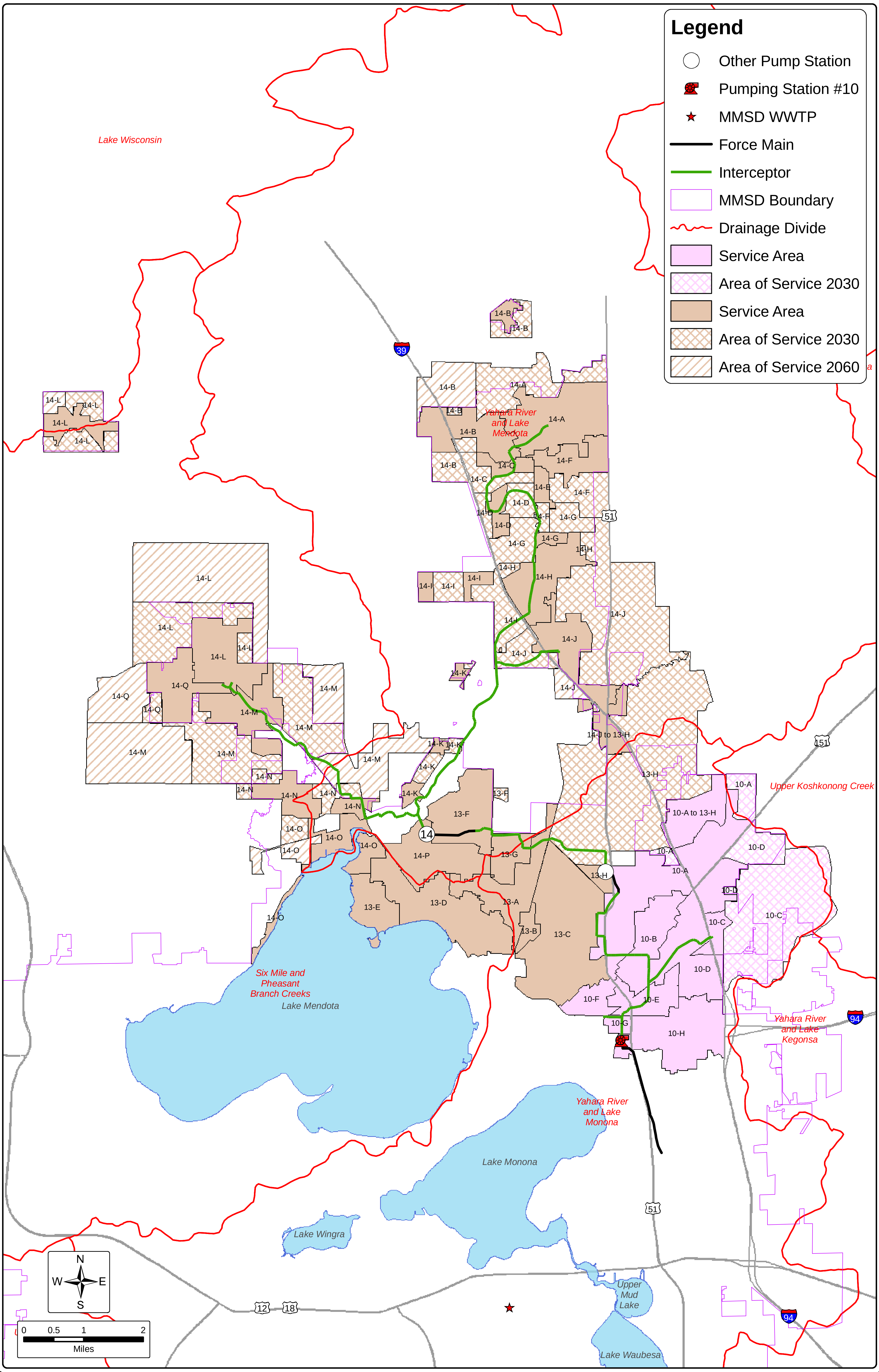
FIGURE 2.12-2
1-547.001



PS 10 - 192 REGAS ROAD
CITY OF MADISON

INTERCEPTOR AND FORCE MAIN ANALYSIS
CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT

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FIGURE 2.12-3
1-547.001

S:\@SAI\501--550\547\001\Data\GIS\Figures\Figure 2.12-3 PS 10 Drainage Area 11x17.mxd

2.13 PS 11–4760 EAST CLAYTON ROAD, TOWN OF DUNN

A. Areas Served by Pumping Station

PS 11 conveys gravity drainage from adjacent sewers and pumped flows from upstream pumping stations. Figure 2.13-1 shows the location of the pumping station and highlights some of its features. Areas of the District that comprise the PS 11 service area include portions of the Town of Madison, the Town of Dunn, the City of Fitchburg, and the City of Madison. This pumping station collects drainage from the NSVI and pumps directly to the NSWTP. Figure 2.13-2 shows the area of the District served by PS 11 as well as the areas served by tributary PSs 12, 16, and 17. Gravity drainage accounts for 39 percent of the average daily flow at this pumping station.

PSs 12, 16, and 17 are tributary to PS 11 and their pumped flow is routed through PS 11. In addition, PS 15 flow may also be routed through PS 16 and subsequently to PSs 12 and 11. Figure 2.13-3 shows the network of pumping stations, force mains, and interceptors associated with PS 11. This figure also highlights components included in the PS 11 overall service area that are projected to reach capacity prior to 2060.

The gravity drainage surface area of PS 11 lies almost entirely within the Yahara Lakes watershed. Nearly all the gravity drainage surface areas for PS 12 (only Area 12A is not in the Sugar River watershed) and all the gravity drainage for PS 17 lie in the Sugar River watershed. The gravity drainage surface area for PS 16 is located almost entirely in the Yahara Lakes watershed although a significant area of potential growth in the Wisconsin River watershed could also be served by PS 16.

B. Description of Pumping Station

1. History of Station

PS 11, located at 4760 East Clayton Road in the Town of Dunn, was constructed and placed into service in 1966. Improvements to this station include:

a. Major changes to the pumping equipment in 1983, which involved three new pumps and the replacement of valve operators within the station.
b. Installed new telemetry system in 1984.
c. In 1988, a fourth pump was installed in this station.
d. Radio for telemetry system in 1990 (replaced 2000).
e. In 1990, a second electric power feed was installed.
f. The roof was replaced in 1993.
g. Control system improvements in 2002.
h. The bar screen was removed in 2006.

2. Current Design Capacities and Limitations

The maximum pumping capacity of PS 11 is approximately 31.2 mgd, whereas the firm capacity is 25.5 mgd. Data from the last decade shows an average flow rate range for PS 11 of 6.5 to 8.5 mgd.

The station has potentially already exceeded its firm capacity in the year 2006 under the high flow projections or will exceed them in 2010 for the low flow projections. The maximum pumping capacity would be potentially exceeded by the year 2016 for high flow projections and the year 2026 for low flow projections unless flow is diverted elsewhere, for example from PS 12 to a Sugar River Plant discharge. At its present capacity, the maximum overall projected firm pumping capacity deficit will be 19.32 mgd under the high flow projections for the year 2060.

As noted above, PS 11 handles all the flow from PSs 12, 16, and 17, and sometimes PS 15, as well as flow from its own pumping station service area. These service areas incorporate the outlying developments of the west side of Madison, and the cities of Fitchburg, Middleton and Verona. The potential for growth in each of these service areas is high.

The 2008 Condition Assessment gave this pumping station an overall rating of 27.00, making its ordinal ranking the third highest priority among the District's 17 pumping stations. Improvements to this pumping station are scheduled for construction prior to 2015. The rating for each pumping station element was as follows:

Adequacy	**Rating**
Maximum Flow Capacity	Adequate
Firm Flow Capacity	Poor
Power System Redundancy	Adequate
Mechanical Condition	Good
Building and Structural Condition	Good
Electrical Condition	Poor
Criticality Factor	1.50
Overall Score	27.00

The force main for PS 11 will also reach its projected capacity based on a design velocity of 8 fps by 2025 under high flow projections and 2048 under low flow projections unless flow is diverted to satellite treatment plants upstream of PS 11.

Options available for increasing this capacity include either increasing the size of the pumps or providing an additional force main and increased pumping capacity. Another option is to construct satellite treatment plants that will discharge to the Sugar River basin. The satellite plant(s) would treat wastewater generated in the Sugar River basin and would reduce the peak hourly flow requirements for PS 11 and 12.

3. Additional Near-Term Planned Improvements (2010-2020)

As discussed above, various elements of PS 11 have been identified as being in adequate to poor overall condition. Areas in need of attention in the near term include the electrical system and the firm pumping capacity of the station. MMSD in its 2009 Capital Improvement Plan identified a project to address the deficiencies of PS 11 (MMSD project 861-00-52). This project is scheduled to begin in 2013 and be completed in 2016.

The following segments of the NSVI are projected to reach capacity between 2010 and 2020:

a. 11Aiii (MH 11-169 to MH 11-167)
b. 11Fii (MH 11-111A to MH 11-106A)
c. 11Fiii (MH 11-106A to MH 11-104)
d. 11Fiv (MH 11-104 to PS 11)

4. Long-Term Considerations (2020-2060)

a. Pumping Station: Based on the current configuration of the MMSD conveyance system, the firm capacity for this pumping station may have already been exceeded around the year 2006 under high flow projections or will be around the year 2010 for low flow projections. The projected capacity needs will be addressed by MMSD Project 861-00-52 described above.

b. Force Mains: The force main for PS 11 is projected to reach its capacity about the year 2025 under high flow projections and 2048 under low flow projections, unless flows are reduced through the construction of satellite plants.

c. Interceptors: The following interceptor segments are projected to reach capacity between 2020 and 2060:

(1) 11Aii(MH 11-171 to MH 11-169)
(2) 11Aiv (MH 11-167 to MH 11-161E)
(3) 11Avi (MH 11-161A to MH 11-159)
(4) 11Bi (MH 11-159 to MH 11-158)
(5) 11Biii (MH 11-156 to MH 11-151A)
(6) 11C (MH 11-151A to MH 11-145)
(7) 11Di (MH 11-145 to MH 11-141) High Flow Projections Only
(8) 11Dii (MH 11-141 to MH 11-137) High Flow Projections Only
(9) 11Diii (MH 11-137 to MH 11-129)
(10) 11Div (MH 11-129 to MH 11-127)
(11) 11Dv (MH 11 -127 to MH 11-116A)
(12) 11Fi (MH 11-116A to MH 11-111A)

d. No allowance has been provided for a future Village of Oregon connection to MMSD. This connection would likely occur at the Syene Road Interceptor upstream of PS 11. The projected 2060 population for Oregon is about 17,275. At an average flow contribution of 100 gpcd, this would result in an additional peak flow of 6.34 mgd to the Syene Road Interceptor by the year 2060.

5. PSs 12, 15, 16, and 17 Operational Impacts on PS 11

As shown in Figure 2.13-2, PS 12 pumps directly to PS 11, and PSs 16 and 17 pump to PS 12. Also, PS 15 can pump to PS 16. Therefore, all wastewater passing through PSs12, 16, and 17 eventually reaches PS 11 and flow from PS 15 can reach PS 11 if diverted from its normal route, which is to pump its flow to PS 8.

6. Potential Options for Rerouting Flow from PSs 12 and 17 to Sugar River WWTP

Flow balancing between watersheds may prove to be an important consideration for the District in the future to meet watershed-related objectives. Therefore, it may become desirable to route flow in Verona and on the west side of Madison to a treatment facility other than NSWTP, thereby providing relief to PS 12 and to PS 11. Possible options for this type of diversion from PS 11 would be to provide new WWTP facilities in the Sugar River Basin. Three options are under consideration. The first option would be to provide a Sugar River plant that would discharge treated flow to the Sugar River in the vicinity of the City of Verona. Flow from PS 17 would be rerouted to this facility instead of NSWTP, thereby returning treated effluent closer to its point of origin in the Sugar River watershed. A portion of the flow treated at NSWTP would likely be returned via the Badger Mill Creek force main that currently has a permitted capacity of 3.6 mgd. Based on the flow projections for the PS 17 Service Area, the maximum amount of relief available to PS 11 and its related components would be 11.62 mgd under low flow projections and 14.54 mgd under high flow projections. This would be an adequate reduction in flows to eliminate the requirement for relief sewers on the NSVI for all interceptor segments 11A to 11 F. This would also be a sufficient reduction in flows to eliminate the need to consider additional force main capacity for PS 11. The reduction in flows would reduce the 2060 peak flow deficit for PS 11 firm capacity from 19.32 mgd to approximately 5.75 mgd. The current maximum pumping capacity at PS 11 would not be exceeded under this option.

A second option would be to construct a treatment plant associated with PS 12 that would take flows from areas 12B-12H as well as 17A and 17D. This would provide average daily flow from these areas that would total 6.25 mgd in 2060 or a peak hourly flow of 18.72 mgd.

A third option would be to construct a CTH PD plant north of the City of Verona in the vicinity of the ending point of segment 17B. Effluent from this plant would be routed directly to the Sugar River at CTH PD or to wetlands adjacent to the river. The potential flow relief to PS 11 would be 1.84 mgd or a peak hourly flow of 6.68 mgd based on the MDC.

Construction of both Option 2 and Option 3 would alleviate all NSVI capacity needs as well as PS 11 capacity needs.

The estimated average daily flow generated by the gravity drainage areas of PS 12 (Areas 12B through 12E) and PS 17 that lie almost entirely within the Sugar River Basin would be as follows:

	2030 Low	**2030 High**	**2060 High**
Nesbitt Road WWTP			
12B	0.25	0.25	0.25
12C	0.41	0.48	0.48
12D	0.44	0.56	0.56

	2030 Low	**2030 High**	**2060 High**
12E	0.22	0.40	0.40
12F	0.24	0.26	0.26
12G	1.16	1.35	1.35
12H	0.09	0.12	0.62
	2.81	3.42	3.92
17A	0.52	0.8	1.05
17D	.69	.78	1.38
ADF	4.02	5.00	6.35
CTH PD WWTP			
17B	0.25	0.78	0.78
17C	0.76	1.06	1.06
ADF	1.01	1.84	1.84
Total Sugar River WS	**5.03**	**6.84**	**8.19**

The corresponding peak hourly flow based on the Madison Design Curve would be as follows:

Year	**Peak Hourly Flow (mgd)**
2030 Low	15.59
2030 High	20.19
2060 High	23.50

The above analysis assumes that all peak hourly flow would be transferred to the Sugar River plant.

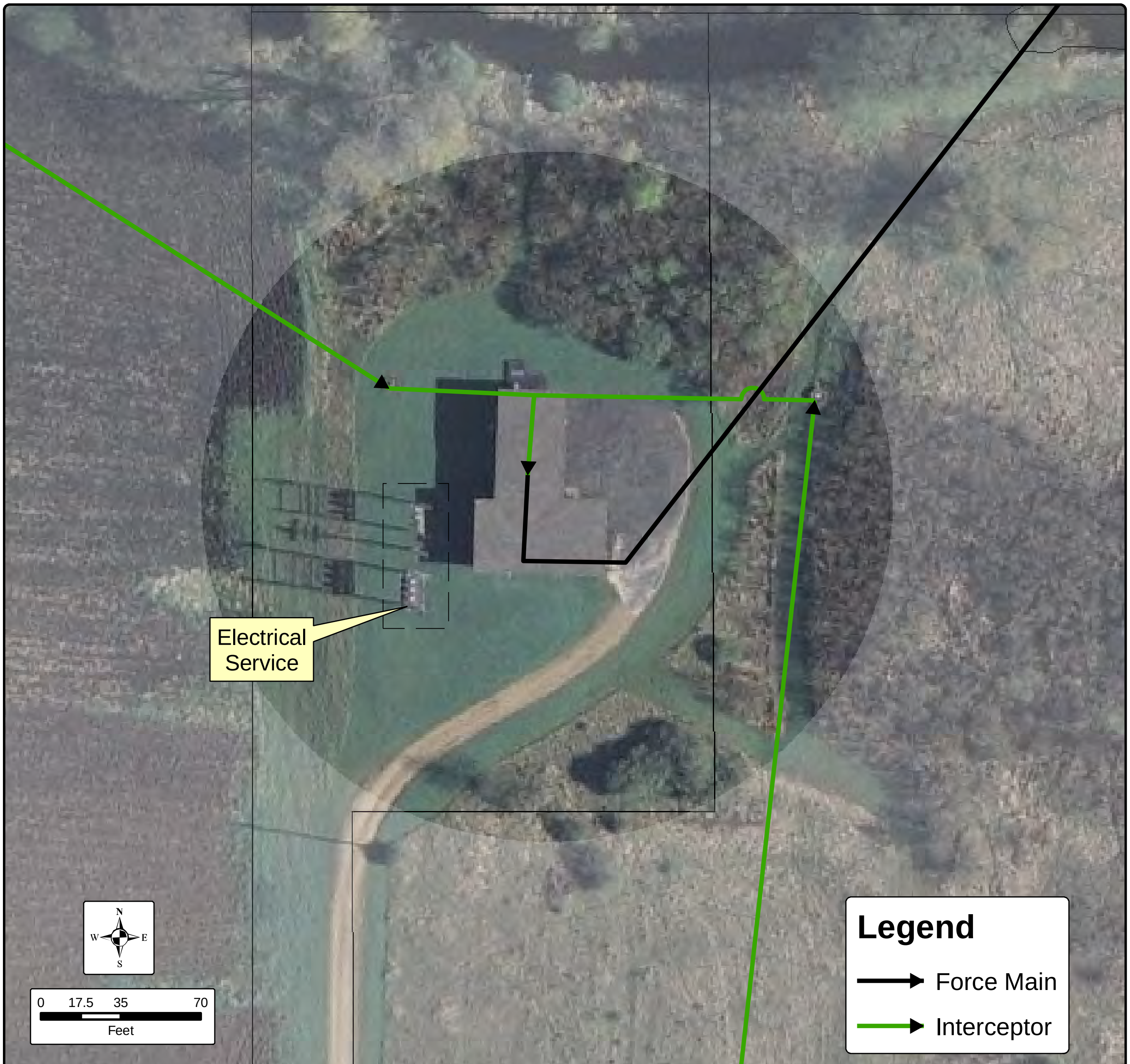
7. System and Electrical Redundancy Review

PS 11 is served from two separate feeds from the Nine Springs Substation (NSP 1319 and NSP 1320). Two separate transformer banks are located at the pumping station to convert 13.8 kV power to the 4,160 volt utilization voltage of the station. Unfortunately, both power feeds are routed to the station on the same pole lines, which route past the pumping station. However, power may be fed from either direction on this pole line.

Although Pumping Station 11 does not have permanent standby power or a connection for a portable generator, one of its pumps, PS 11A, could be powered from one of MMSD's standard portable generator sets in the event of a total power outage. The backup generator set at PS 17, could be used to power a larger pump, PS 11B, if that were found to be necessary. The largest pumps are at 4 kV so powering them from a generator set with the station's current electrical configuration becomes difficult. As with any time a generator is used, extreme caution must be taken to prevent any possible back-feeding of power into the utility's system. A review of the pumping station's electrical configuration, power system redundancy, and standby power needs will be a part of the upcoming pumping station rehabilitation.

According to the March 13, 2007, MMSD Emergency Response Manual, during a service outage there is adequate storage in the interceptors and local sewers upstream of this pump station to prevent any basement back-ups or sewer overflows for five hours under normal flow conditions and three hours under high flow conditions.

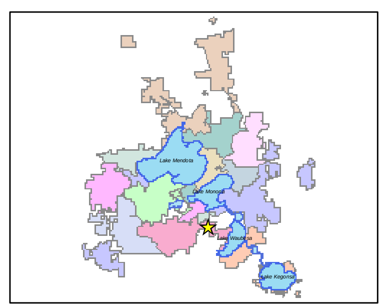
S:\@SAI\501--550\547\001\Data\GIS\Figures\Figure 2.13-1 PS11.mxd



PS 11-Clayton Road
4670 East Clayton Road, Town of Dunn

Originally Constructed	1966
Upgrades	1993
Capacity (2010)	
Maximum (mgd)	31.2
Firm (mgd)	25.5

Pumps	Q (gpm)	H (ft)	Motor (HP)	Year On-Line
11A	6,400	43	125	1950
11B	9,100	49	150	1982
11C	13,300	57	250	1982
11D	13,300	57	250	1982



PS 11 - 4670 E. CLAYTON ROAD
TOWN OF DUNN

CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT

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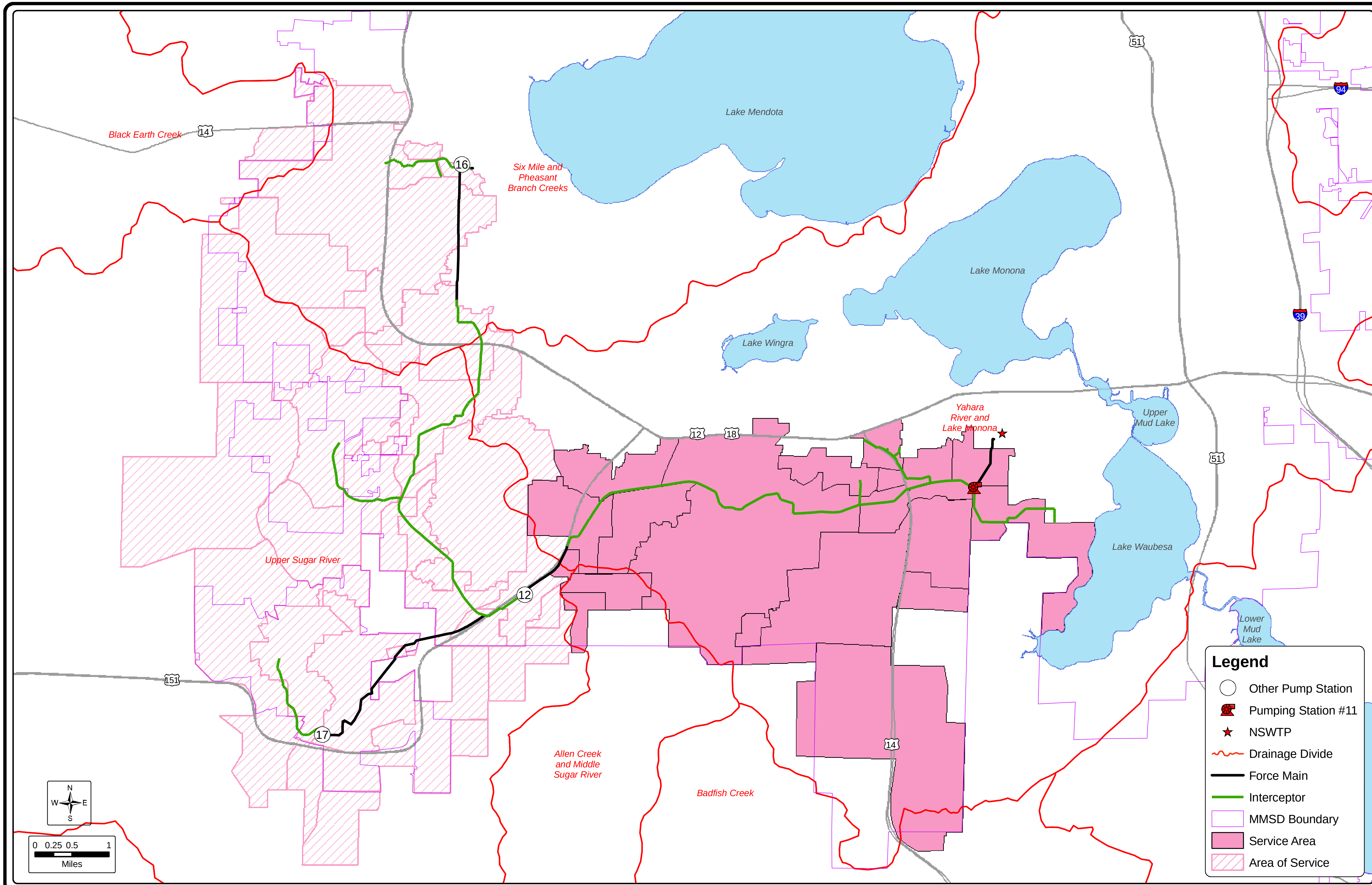
FIGURE 2.13-1
1-547.001



PS 11 - 4760 E. CLAYTON ROAD
TOWN OF DUNN

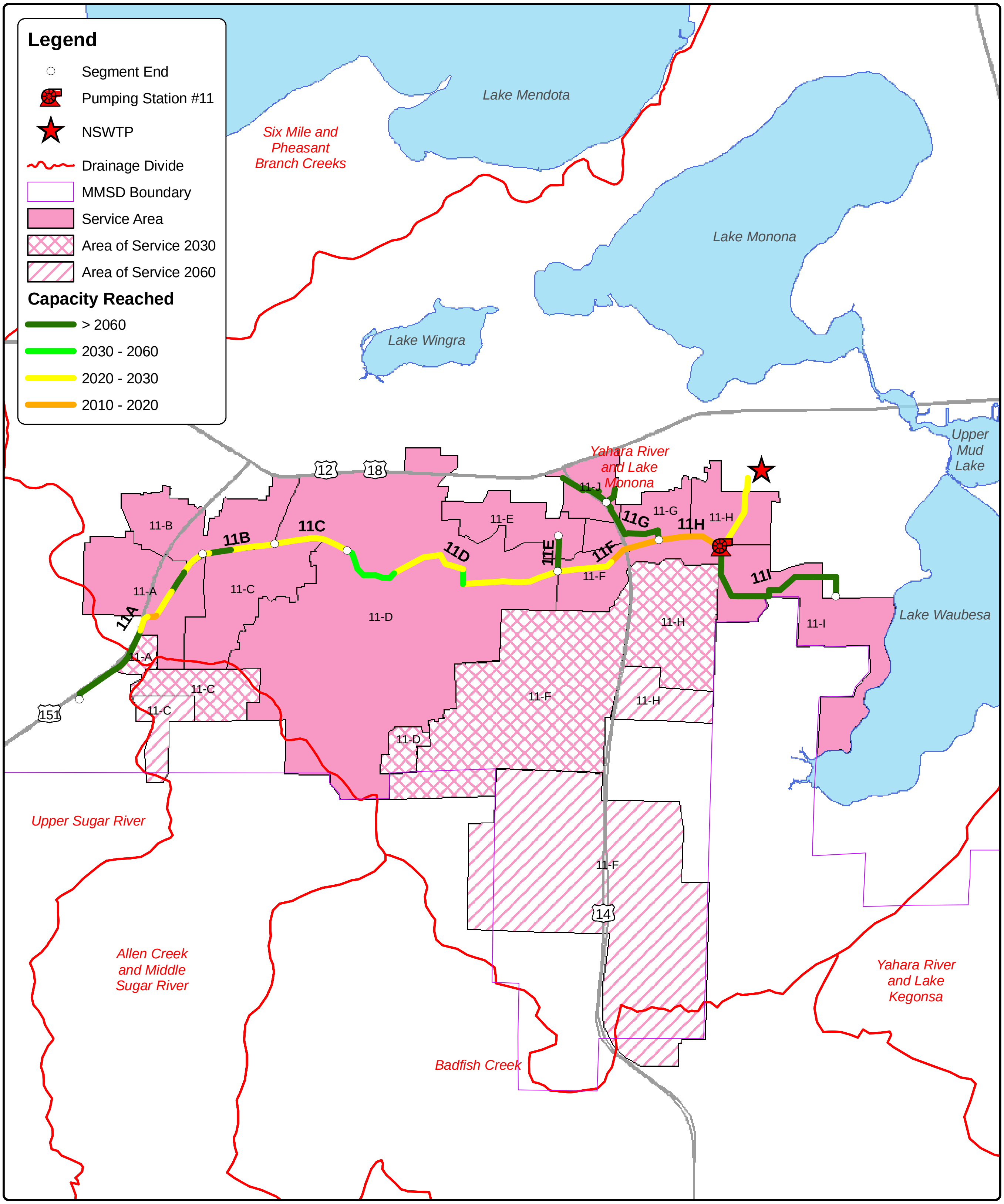
OVERALL SERVICE AREA
CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT

STRAND ASSOCIATES, INC.® ENGINEERS

FIGURE 2.13-2
1-547.001

S:\MAD\1500--1599\1547\001\Data\GIS\Figures\Figure 2.13-2 PS 11 Drainage Area 11x17.mxd

S:\MAD\1500--1599\1547\001\Data\GIS\Figures\Figure 2.13-3 PS 11 Drainage Area 8x11.mxd



PS 11 - 4760 E. CLAYTON ROAD
TOWN OF DUNN

INTERCEPTOR AND FORCE MAIN ANALYSIS
CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT

STRAND ASSOCIATES, INC.® ENGINEERS

FIGURE 2.13-3
1-547.001

2.14 PS 12–2739 FITCHRONA ROAD, TOWN OF VERONA

A. Areas Served by Pumping Station

PS 12 conveys gravity drainage from adjacent sewers and pumped flows from PS 15 (as an alternative operating mode), 16, and 17. Figure 2.14-1 shows the location of this pumping station and highlights some of its features. Areas of the District that comprise the PS 12 service area include a portion of the southwest side of the City of Madison, a portion of the City of Fitchburg and the Town of Verona. This pumping station is on the NSVI, and therefore both collects drainage from the interceptor and pumps to it. Figure 2.14-2 shows the area of the District served first by PS 12 as well as the areas served by PSs 15, 16, and 17. Gravity drainage accounts for approximately 50 percent of the average daily flow at this pumping station. The gravity drainage service area for PS 12 lies almost entirely in the Sugar River watershed except for sewer service area 12 A as shown on Figure 2.14-3. The gravity drainage surface area for PS 17 lies entirely in the Sugar River watershed. The gravity drainage service area for PS 15 lies entirely within the Yahara Lakes watershed while the gravity drainage service area for PS 16 lies mostly within the Yahara Lakes watershed but does have a portion of its service area in the Wisconsin River watershed.

PS 12 receives pumped flows from PS 16 and PS 17 through interceptors tributary to PS 12. In addition, flow from PS 15 can also be routed through PS 16. Figure 2.14-3 shows the network of pumping stations, force mains, and interceptors associated with PS 12. This figure also highlights components included in the PS 12 overall service area that are projected to reach capacity prior to 2060.

B. Description of Pumping Station

1. History of Station

PS 12, located at 2739 Fitchrona Road in the Town of Verona, was constructed and placed into service in 1969. The comminutor was removed in 1980. Improvements to this station include major changes to the pumping equipment in 1983, which involved two new pumps and the replacement of valve operators within the station. Other improvements include installing a new telemetry system in 1984, radio telemetry in 1990 (subsequently replaced in 2000), a roof replacement in 1995, and control system revisions in 2000.

2. Current Design Capacities and Limitations

The maximum pumping capacity of PS 12 is approximately 23.5 mgd, whereas the firm capacity is 16.6 mgd. Data from the last decade shows an average flow rate for PS 12 of 5 mgd.

As noted above, PS 12 handles all the flow from PS 16 and PS 17, and potentially from PS 15, as well as flow from its own pumping station service area. These service areas incorporate the outlying developments of the west side of Madison, the City of Middleton, and the City of Verona. The potential for growth in each of these service areas is high. Based on the current mode of operation (i.e., PS 15 pumps to PS 8), the projected peak hourly flow to PS 12 exceeded the firm capacity in 2008 under low flow projections and 2005 under high flow projections. Maximum pumping capacity would be exceeded by the year 2020 under high flow projections and 2030 under low flow projections. The projected maximum firm capacity deficit under high flow projections for the year 2060 is 15.7 mgd.

The 2008 Condition Assessment established an overall rating score of 29.25 for this pumping station, making its overall ordinal ranking as the second highest priority for improvements among the District's 17 pumping stations. Improvements are scheduled for construction within five years. The rating for each pumping station element was as follows:

Adequacy	Rating
Maximum Flow Capacity	Adequate
Firm Flow Capacity	Very Poor
Power System Redundancy	Poor
Mechanical Condition	Good
Building and Structural Condition	Good
Electrical Condition	Poor to Adequate
Criticality Factor	1.50
Overall Score	29.25

3. Near-Term Planned Improvements (2010-2020)

As discussed above, the overall condition of PS 12 has been identified as below average with respect to the other pumping stations in the District. Areas in need of attention in the near term include the power supply system redundancy and the firm capacity of the station. The 2009 MMSD Capital Projects Budget (MMSD project 861-00-52) includes a major rehabilitation project for this pumping station beginning in 2013 with a projected completion date of 2016.

As noted above, PS 12 currently pumps all the flow from PS 16 and PS 17, and potentially PS 15, as well as flow from its own pumping station service area. These service areas incorporate the outlying developments of the west side of Madison, and the potential for growth in each of these service areas is high. The projected peak hourly flow without diversion will exceed the firm capacity of PS 12 prior to the year 2010 under either low or high flow projections.

4. Long-Term Considerations (2020-2060)

   a. Pumping Station: Based on the current configuration of the MMSD conveyance system, firm capacity for this pumping station was reached by about the year 2005 under high flow projections and about the year 2010 for low flow projections. Maximum pumping capacity will be reached about the year 2019 under high flow projections and 2031 under low flow conditions. The firm capacity deficit under high flow projections in the year 2060 is projected to be 15.7 mgd. Additional firm capacity will be considered as a part of the MMSD project 861-00-52 described above.

   b. Force Mains: The PS 12 force main is not projected to reach its capacity based on 8 fps velocity prior to 2060.

c. Interceptors: The only segments for PS 12 interceptors projected to reach capacity under the normal operating mode of PS 15 pumping to PS 8 would be segments 12 Hi and 12 Hii. These segments are located between the terminus of the PS 17 force main and PS 12. They are projected to be at capacity at the year 2017 under high flow projections and 2028 under low flow projections. The projected capacity deficit is 8.14 mgd for 12Hi and 9.57 mgd for 12 Hii.

5. Potential Options for Rerouting Flow from PS 12 and PS 17 to Sugar River WWTP

Flow-balancing between watersheds may prove to be an important consideration for the District in the future to meet watershed-related objectives. Therefore, it may become desirable to route flow from Verona and the west side of Madison to a treatment facility other than NSWTP. One possible option to achieve this flow-balancing would be to construct a Sugar River WWTP on the southwest side of the Madison area that would discharge into the Sugar River Drainage Basin. A portion of the flow from PS 12 and PS 17 would then be rerouted to the Sugar River WWTP instead of NSWTP, thereby returning treated effluent closer to its point of origin. A byproduct of this change would be to reduce the pumping demands of PS 12, which would no longer receive flow from PS 17. Likely, however, the old force mains and interceptors would be retained since the ability of the Sugar River to accept all of the peak hourly flow may be limited. This would allow routing of peak hourly flows in excess of the design capacity of the Sugar River plant (e.g. flows greater than a 2 to 1 peaking factor) to the NSWTP.

The estimated average daily flow generated by the gravity drainage areas of PS 12 and PS 17 which lie almost entirely within the Sugar River Basin would be as follows (low flow projections only):

Sugar River Plant	2030 Low	2030 High	2060 High
12B	0.25	0.25	0.25
12C	0.41	0.48	0.48
12D	0.44	0.56	0.56
12E	0.22	0.40	0.40
12F	0.24	0.26	0.26
12G	1.16	1.35	1.35
12H	0.09	0.12	0.62
	2.81	3.42	3.92
17A	0.52	0.8	1.05
ADF	3.33	4.22	4.97
Upper Sugar River			
17B	0.25	0.78	0.78
17C	0.76	1.06	1.06

Sugar River Plant	**2030 Low**	**2030 High**	**2060 High**
17D	0.69	0.78	1.38
ADF	1.70	2.62	3.22
Total Sugar River Watershed	**5.03**	**6.84**	**8.19**

The corresponding peak hourly flow based on the Madison Design Curve would be as follows:

Year	**Peak Hourly Flow (mgd)**
2030 Low	15.59
2030 High	20.19
2060 High	23.50

This analysis assumes that all of the peak hourly flow would be transferred to the Sugar River WWTP(s). Under the scenario presented above, construction of a Sugar River Plant for the flows for PS 17 service area would provide sufficient capacity relief for PS 12.

All capacity deficits for PS 12 and the related interceptors would be eliminated if ADF of approximately 2.8 mgd is routed away from PS 12.

6. System and Electrical Redundancy Review

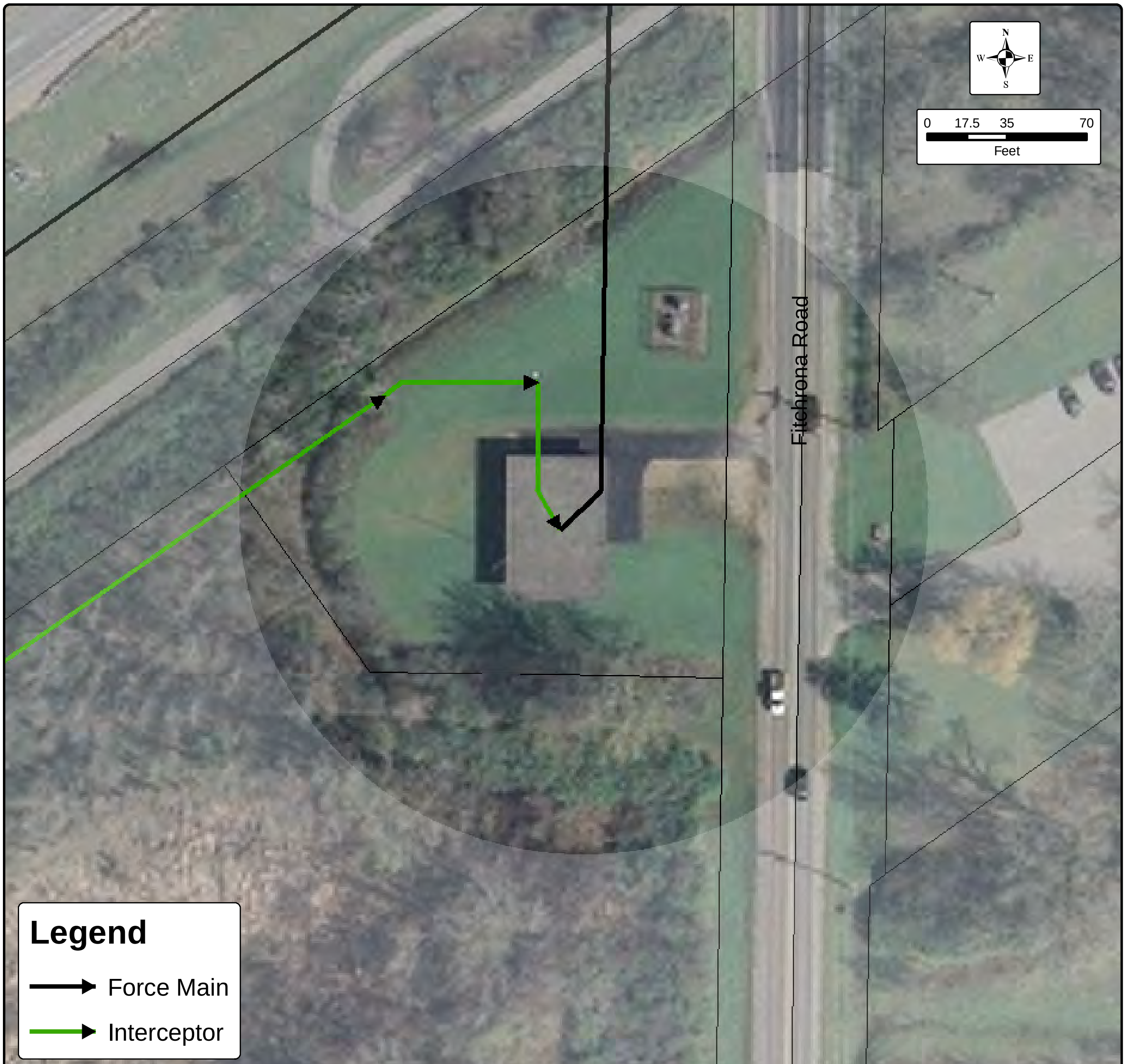
PS 12 is served from two separate feeds from MG&E's Fitchburg Substation (FCH 1319 and FCH 1316).

The two services feed a MMSD owned automatic transfer switch (G&W switch) at the primary (utility) voltage. The switch always seeks an active power feed and in turn feeds a bank of utility transformers that convert the power to the pumping station's utilization voltage of 480 volts. Although redundant feeds power the site, failure of any transformer in the bank of transformers or of the switch will result in a loss of power to the station.

This pumping station is not provided with a standby power generator nor does the station have a permanent connection for a portable generator. However, MMSD does have a portable generator large enough to power the two smallest pumps. The backup generator from PS 17 could be used to power the larger pumps in parallel if it were necessary and that generator set was available (PS 17 had utility power). As with any time a generator is used, extreme caution must be taken to prevent any possible back-feeding of power into the utility's system. The pumping station's power system redundancy and standby power options will be reviewed during the planning and design process for the station's rehabilitation.

According to the March 13, 2007, MMSD Emergency Response Manual, during a service outage there is adequate storage in the interceptors and local sewers upstream of this pump station to prevent any basement back-ups or sewer overflows for two and three-quarters hours under normal flow conditions and one hour under high flow conditions.

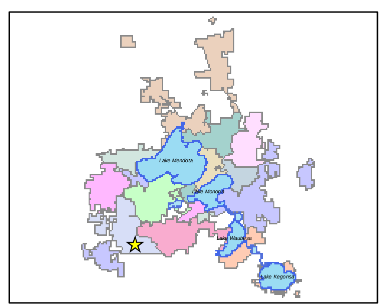
S:\@SAI\501--550\547\001\Data\GIS\Figures\Figure 2.14-1 PS12.mxd



PS 12-Fitchrona Road
2739 Fitchrona Road, Verona

Originally Constructed	1969
Upgrades	2000
Capacity (2008)	
Maximum (mgd)	23.5
Firm (mgd)	16.6

Pumps	Q (gpm)	H (ft)	Motor (HP)	Year On-Line
12A	3,400	44	50	1969
12B	7,200	48	100	1969
12C	9,000	48	150	1982
12D	9,000	48	150	1982



PS 12 - 2739 FITCHRONA ROAD
TOWN OF VERONA

CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT

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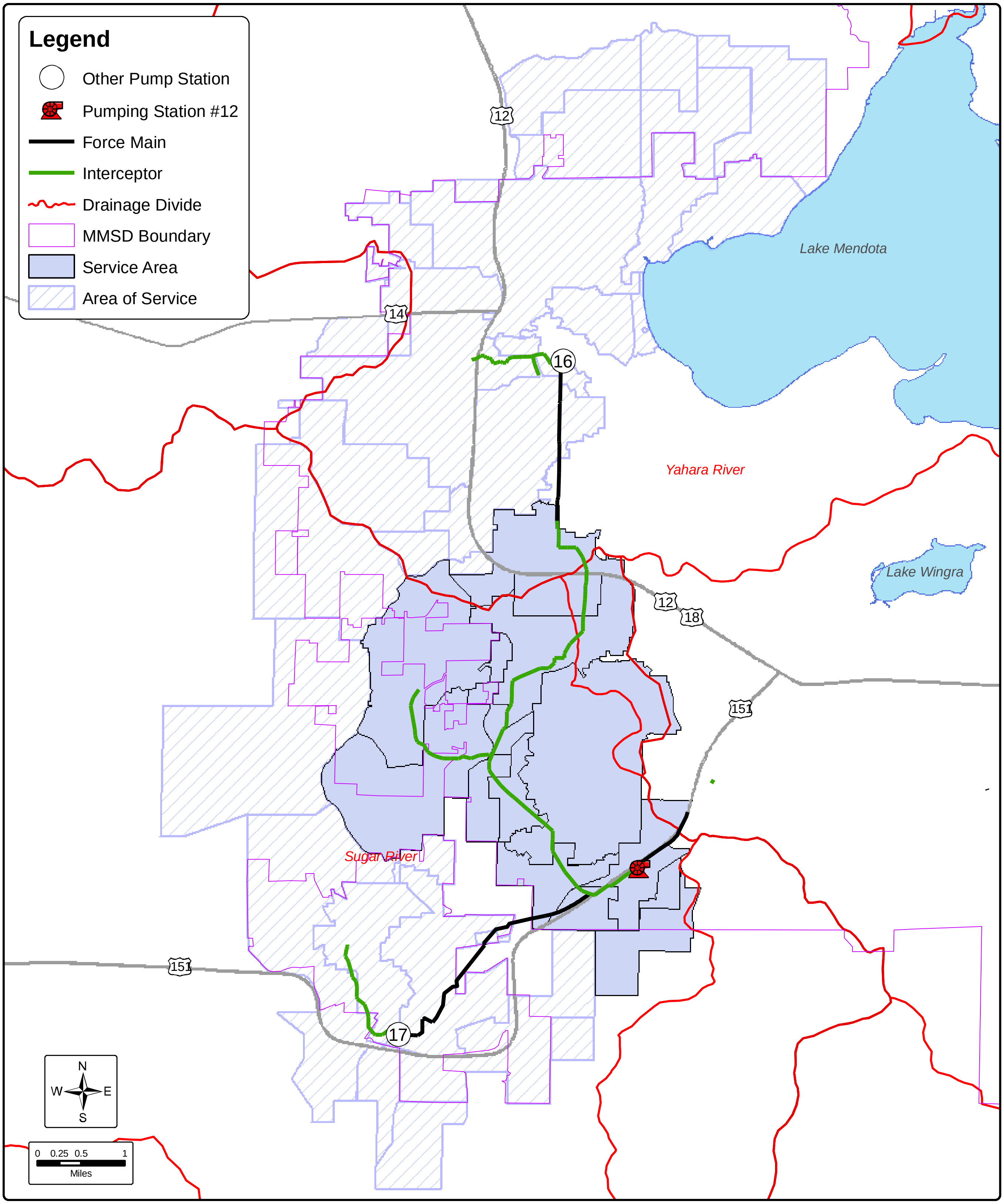
FIGURE 2.14-1
1-547.001

S:\@SAI\501--550\547\001\Data\GIS\Figures\Figure 2.14-2 PS 12 Drainage Area 8x11.mxd



PS 12 - 2739 FITCHRONA ROAD
TOWN OF VERONA

OVERALL SERVICE AREA
CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT

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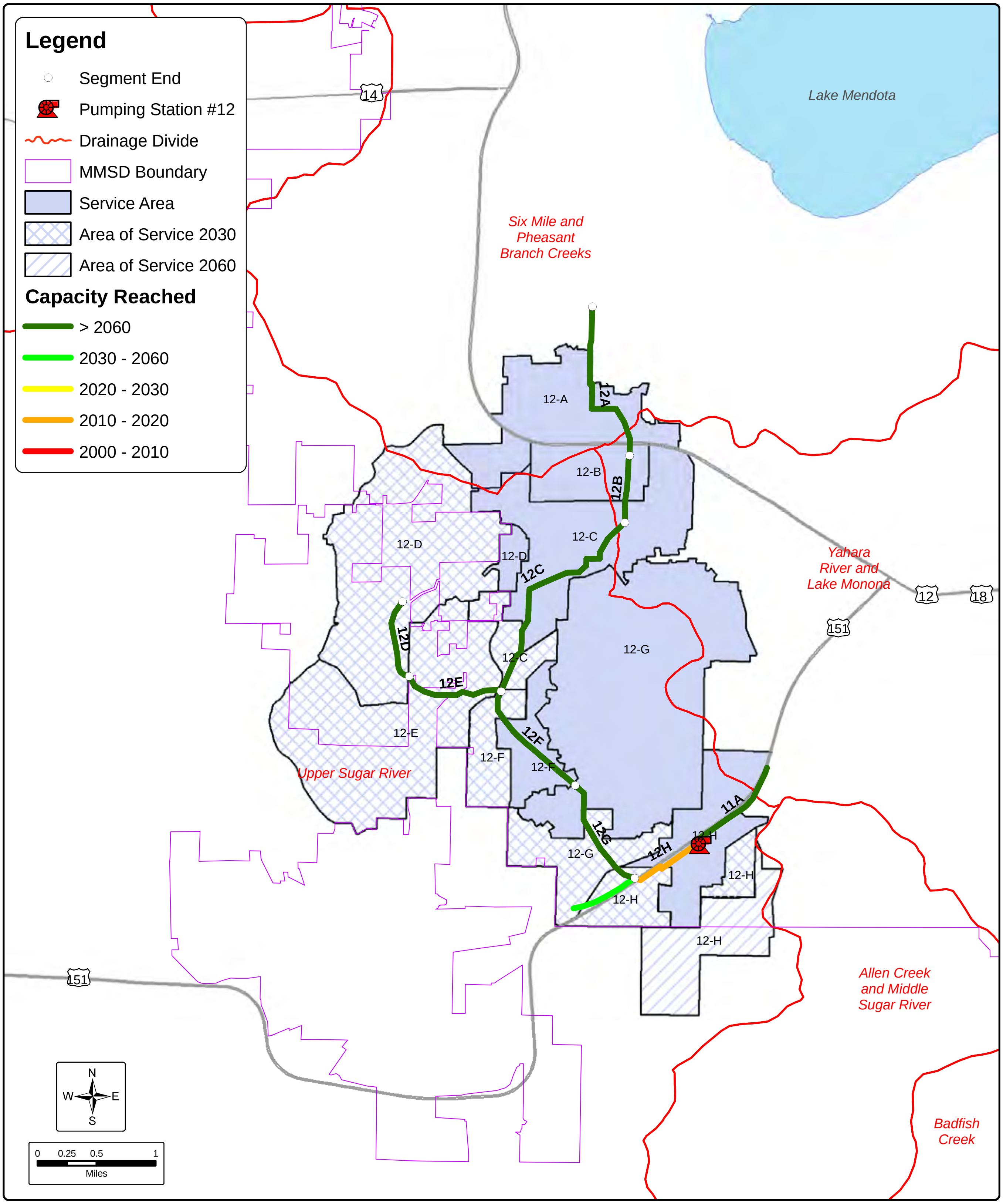
FIGURE 2.14-2
1-547.001

S:\@SAI\501--550\547\001\Data\GIS\Figures\Figure 2.14-3 PS 12 Drainage Area 8x11.mxd



PS 12 - 2739 FITCHRONA ROAD
TOWN OF VERONA

INTERCEPTOR AND FORCE MAIN ANALYSIS
CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT

STRAND ASSOCIATES, INC.® ENGINEERS

FIGURE 2.14-3
1-547.001

2.15 PS 13-3634 AMELIA EARHART DRIVE, MADISON

A. Areas Served by Pumping Station

PS 13 conveys gravity drainage from adjacent sewers and pumped flows from upstream pumping stations. Figure 2.15-1 shows the location of the station and highlights some of its features. Areas of the District that comprise the PS 13 gravity drainage service area include a portion of the northeast side of the City of Madison. This pumping station is on the NEI, and therefore both collects drainage from the interceptor and pumps to it. Figure 2.15-2 shows the area of the District served first by PS 13 as well as the gravity drainage surface area for PS 14 which discharges to PS 13. Gravity drainage accounts for 39 percent of the average daily flow at this pumping station for the years between 1996 and 2007.

PS 13 receives pumped flow from PS 14 directly. Figure 2.15-3 shows the network of pumping stations, force mains, and interceptors associated with PS 13.

B. Description of Pumping Station

1. History of Station

PS 13, located at 3634 Amelia Earhart Drive in Madison, was constructed and placed into service in 1971. The comminutor was removed in 1980. A new telemetry system was installed in 1984 and upgraded with radio telemetry in 1990 (radio subsequently replaced in 2000). The control system was upgraded in 2002 and again modified as part of the 2008 upgrade. The Pump A motor starter was replaced in 2005. An upgrade to firm pumping capacity was completed in 2008.

2. Current Design Capacities and Limitations

The maximum pumping capacity of PS 13 is approximately 20.2 mgd, whereas the firm capacity is 20 mgd. Pumped flows from 1996 through 2007 averaged approximately 5 mgd. In 2008, this pumping station received a firm capacity increase from 12.2 to 20 mgd. With this increase, the firm capacity of this pumping station is projected to be reached between 2010 and 2020.

The 2008 Condition Assessment gave this pumping station an overall rated score of 19.25, making its ordinal ranking the fourth highest priority for improvements among the District's 17 pumping stations. A project is included in the 2009 Capital Projects Budget (MMSD project 863-00-50) that will address the pumping station's deficiencies. The project is scheduled to begin in 2015 and be completed in 2018. The rating for each pumping station element was as follows:

Adequacy	Rating
Maximum Flow Capacity	Adequate
Firm Flow Capacity	Adequate
Power System Redundancy	Poor
Mechanical Condition	Excellent

Adequacy	Rating
Building and Structural Condition	Adequate
Electrical Condition	Poor to Adequate
Criticality Factor	1.10
Overall Score	19.25

3. Additional Near-Term Planned Improvements (2010-2020)

As discussed above, PS 13 has been identified as in need of a major rehabilitation project to address capacity deficiencies, electrical service, and power redundancy. A major rehabilitation project to achieve these items is planned for PS 13 between 2015 and 2018.

4. Long-Term Considerations (2020-2060)

a. Pumping Station: Based on the current configuration of the MMSD conveyance system, firm capacity for this pumping station will be reached by about the year 2010 under high flow projections and about the year 2020 for low flow projections. The firm capacity deficit under high flow projections in the year 2060 is projected to be 9.44 mgd.

b. Force Mains: The force main is not expected to reach its capacity prior to the year 2060.

c. Interceptors: The only segments for PS 13 interceptors projected to reach capacity would be these segments:

(1) 13 A-Ei (MH 13-122A to MH 13-116H).
(2) 13G (MH 13-132 to MH 13-122A) High Flow Projections Only
(3) 13Hi (MH 13-105 A to MH 13-105) High Flow Projections Only
(4) 13Hii (MH 13-105 to PS 13)

5. PS 14 Operational Impacts on PS 13

PS 14 pumps directly to PS 13. Therefore, all wastewater from the PS 14 service area must also be pumped by PS 13. The PS 14 gravity service area from 1996 to 2007 contributed an average daily flow of 3.5 mgd compared to the 1.8 mgd contributed from the gravity drainage area of PS 13. In addition, more growth is likely in the PS 14 service area than in that of PS 13. Due to these factors, any diversion of flow from PS 14 to an alternative location for treatment will have a significant impact on the flows and future capacity needs at PS 13.

6. Potential Options for Rerouting Flow from PS 14 to Mendota WWTP

Flow balancing between watersheds or within watersheds may prove to be an important consideration for the District in the future to meet watershed-related objectives. Therefore, it may become desirable to route flow from Waunakee, DeForest, and/or the north side of Madison to a treatment facility other than NSWTP, which would in turn provide relief to PS 13 and ultimately PS 10 and PS 7. One possible option for this type of diversion from PS 13 would be to construct a Mendota WWTP near the Yahara River where it enters Lake Mendota. Some or all of the flow from PS 14 would then be rerouted to the Mendota WWTP instead of NSWTP, thereby returning treated effluent closer to its point of origin and providing an additional source of base flow to the Yahara River, from just above Lake Mendota and following downstream. Diversion to a Mendota WWTP would serve the purpose of providing relief to PS 13, PS 10 and PS 7. The projected year 2060 average daily flow from PS 14 would be 6.83 mgd. The resulting peak hourly flow using the MDC would be 20.3 mgd. None of the indicated capacity improvements for PS 13 and related interceptors would be required if the peak hourly flow from PS 14 were reduced by 9.44 mgd.

6. System and Electrical Redundancy Review

PS 13 is served from two separate feeds from the separate MG&E substations (AMN 1313 and ETN 1335). The two services feed a MMSD owned automatic transfer switch (G&W switch) at the primary (utility) voltage. The switch always seeks an active power feed and in turn feeds a bank of utility transformers that convert the power to the pumping station's utilization voltage of 480 volts. Although redundant feeds power the site, failure of any transformer in the bank of transformers or of the switch will result in a loss of power to the station.

This pumping station is not provided with a standby power generator nor does the station have a permanent connection for a portable generator. However, MMSD does have a portable generator large enough to power any of the pumps at this pumping station. As with any time a generator is used, extreme caution must be taken to prevent any possible back-feeding of power into the utility's system. The pumping station's power system redundancy and standby power options will be reviewed during the planning and design process for the station's rehabilitation.

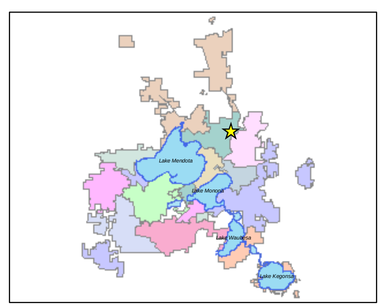
According to the March 13, 2007, MMSD Emergency Response Manual, during a service outage there is adequate storage in the interceptors and local sewers upstream of this pump station to prevent any basement back-ups or sewer overflows for four hours under normal flow conditions and two hours under high flow conditions.

S:\@SAI\501--550\547\001\Data\GIS\Figures\Figure 2.15-1 PS13.mxd

PS 13-Amelia Earhart
3634 Amelia Earhart Drive, Madison

Originally Constructed	1970
Upgrades	2008
Capacity (2008)	
Maximum (mgd)	20.2
Firm (mgd)	20.0

Pumps	Q (gpm)	H (ft)	Motor (HP)	Year On-Line
13A	8,200	16	50	2008
13B	8,200	16	50	1970
13C	14,000	20	100	1970



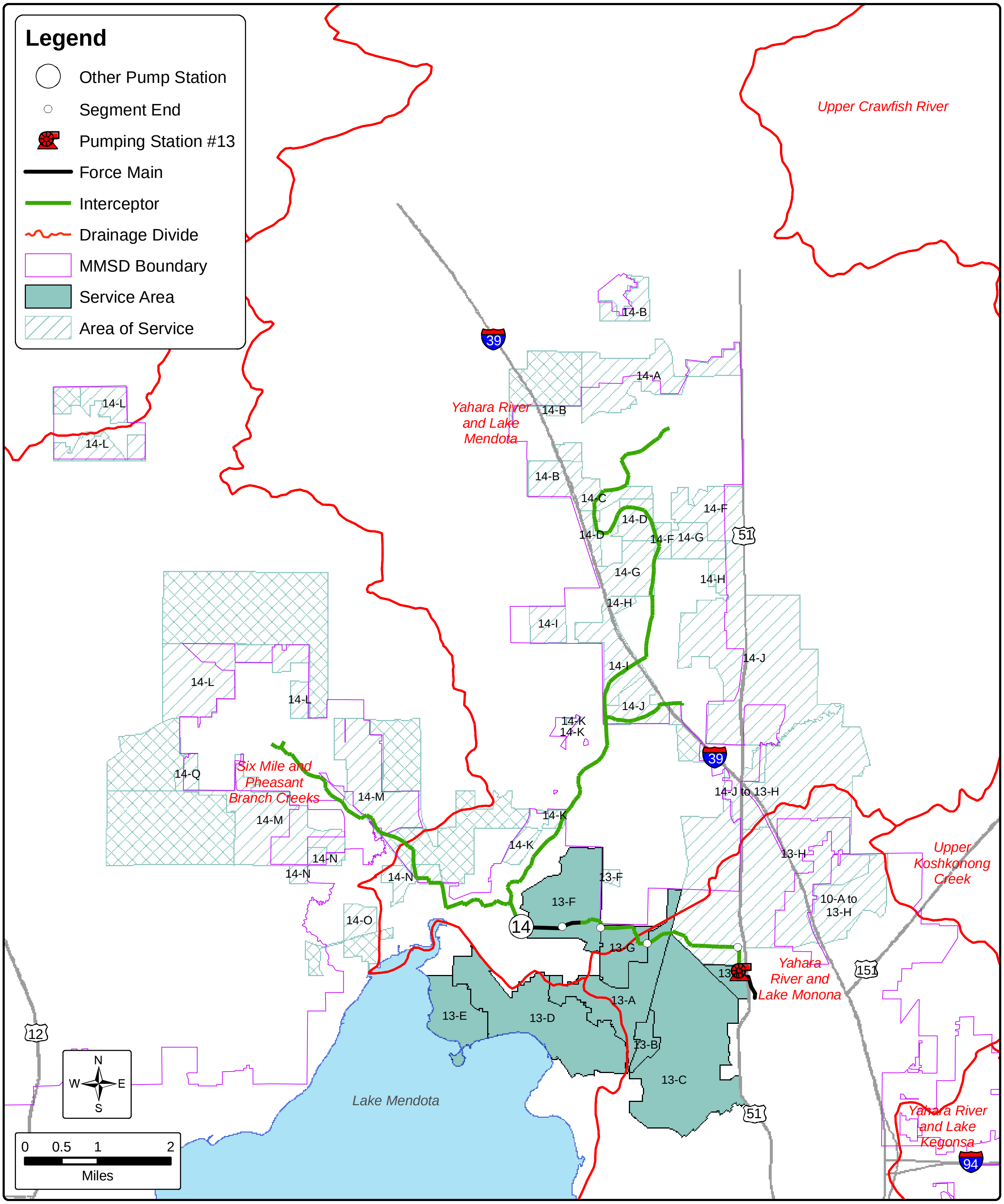
PS 13 - 3634 AMELIA EARHART DRIVE
CITY OF MADISON

CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT

STRAND ASSOCIATES, INC.® ENGINEERS

FIGURE 2.15-1
1-547.001

S:\@SAI\501--550\547\001\Data\GIS\Figures\Figure 2.15-2 PS 13 Drainage Area 8x11.mxd



PS 13 - 3634 AMELIA EARHART DRIVE
CITY OF MADISON

OVERALL SERVICE AREA
CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT

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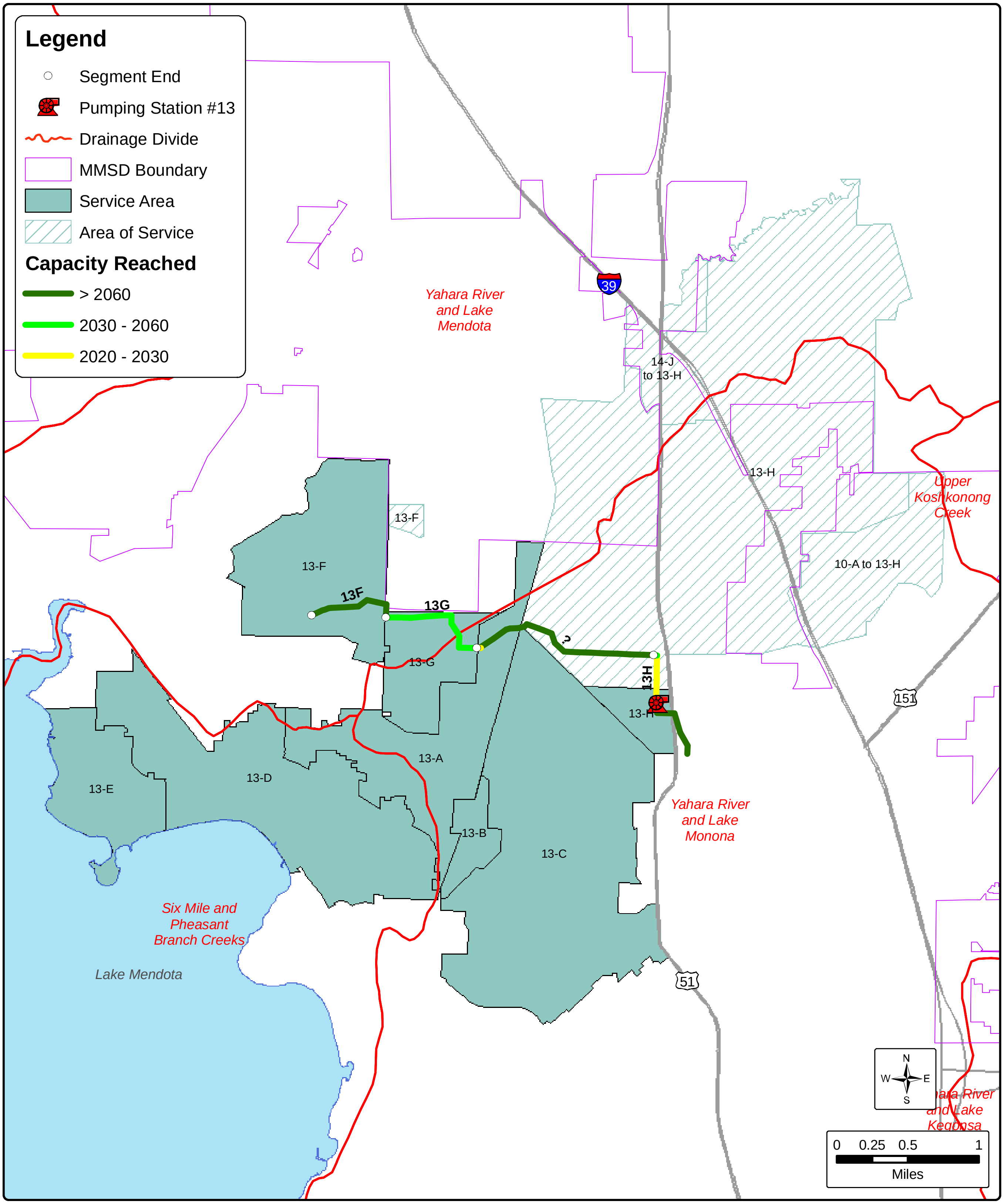
FIGURE 2.15-2
1-547.001

S:\@SAI\501--550\547\001\Data\GIS\Figures\Figure 2.15-3 PS 13 Drainage Area 8x11.mxd



PS 13 - 3634 AMELIA EARHART DRIVE
CITY OF MADISON

INTERCEPTOR AND FORCE MAIN ANALYSIS
CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT

STRAND ASSOCIATES, INC.® ENGINEERS

FIGURE 2.15-3
1-547.001

2.16 PS 14-5000 SCHOOL ROAD, CITY OF MADISON

A. Areas Served by Pumping Station

PS 14 conveys both gravity drainage from adjacent sewers and pumped flows from extra-jurisdictional pumping stations. Figure 2.16-1 shows the location of this pumping station and highlights some of the features of the pumping station. Areas of the District that comprise the PS 14 service area include the following:

(1) Village of Dane
(2) Village of Waunakee
(3) Village of DeForest
(4) Town of Windsor Sanitary District 1
(5) Town of Windsor Lake Windsor Sanitary District
(6) Town of Windsor Hidden Springs Sanitary District
(7) Town of Windsor Oak Springs Sanitary District
(8) Town of Vienna - Illinois Seed Foundation
(9) Town of Windsor Morrisonville Sanitary District
(10) Town of Vienna Utility Districts 1 and 2
(11) A portion of the Town of Westport Utility District 1
(12) A portion of the City of Madison

PS 14 pumps to the NEI and collects drainage from both the Waunakee and DeForest Legs of the NEI. Figure 2.16-2 shows the area of the District served first by PS 14 as well as the extra-jurisdictional pumping stations, interceptors, and force main. This figure also shows the summary of the capacity analysis for MMSD infrastructure.

Most of the PS 14 service area lies within the Yahara Lakes watershed although a portion of the Village of Dane lies in the Wisconsin River Watershed.

B. Description of Pumping Station

1. History of Station

PS 14, located at 5000 School Road in Madison, was constructed and placed into service in 1972. The comminutor was removed in 1984. A new telemetry system was installed in 1984 and upgraded with radio telemetry in 1990 (the radio was subsequently replaced in 2000). The roof was replaced in 1996. In 2002, the control system was revised and upgraded. The Pump A motor starter was replaced in 2005. A firm capacity upgrade was completed in 2008, which also required electrical and control system modifications.

2. Current Design Capacities and Limitations

The maximum pumping capacity of PS 14 is approximately 15.6 mgd, whereas the firm capacity is 15.0 mgd. Pumped flows from 1996 through 2007 averaged approximately 3.5 mgd. In 2008, this pumping station received a firm capacity increase from 8.9 to 15 mgd. With this increase, the firm capacity of this pumping station is projected to be reached between 2023 and 2038.

The 2008 Condition Assessment gave this pumping station an overall condition rating of 17.05, making its ordinal ranking the seventh highest priority for improvements among the District's 17 pumping stations. A project is included in the 2009 Capital Projects Budget (MMSD project 863-00-50) that will address the deficiencies. The project is scheduled to begin in 2015 and be completed in 2018. The rating for each pumping station element was as follows:

Adequacy	Rating
Maximum Flow Capacity	Good
Firm Flow Capacity	Good
Power System Redundancy	Poor
Mechanical Condition	Excellent
Building and Structural Condition	Adequate
Electrical Condition	Poor to Adequate
Criticality Factor	1.10
Overall Score	17.05

No interceptor segments will reach capacity prior to the year 2010 under either flow projection.

3. Additional Near-Term Planned Improvements (2010-2020)

As discussed above, PS 14 has been identified as in need of a major rehabilitation project to address capacity deficiencies, electrical service, and power redundancy. A major rehabilitation project to achieve these items is planned for PS 14 between 2015 and 2018 and was included in the MMSD 2009 Capital Projects Budget.

The following interceptor segment has been identified as needing additional capacity between 2010 and 2020:

- 14Mi (MH 14-356 to MH 14-345)

4. Long-Term Considerations (2020-2060)

   a. Pumping Station: Based on the current configuration of the MMSD conveyance system, firm capacity for this pumping station will be reached by about the year 2023 under high flow projections and about the year 2038 for low flow projections. The firm capacity deficit under high flow projections in the year 2060 is projected to be 5.16 mgd.

   b. Force Mains: The force main is not expected to reach its capacity prior to the year 2060.

c. Interceptors: The following interceptor segments upstream of PS 14 will reach their design capacity between 2020 and 2060:

(1) 14B (MH 14-196 to MH 14-193).
(2) 14D (MH 14-182 to MH 14-171) High Flow Projections Only
(3) 14E (MH 14-171 to MH 14-166) High Flow Projections Only
(4) 14Fi (MH 14-166 to MH 14-165) High Flow Projections Only
(5) 14Fii (MH 14-165 to MH 14-162) High Flow Projections Only
(6) 14G (MH 14-162 to MH 14-156) High Flow Projections Only
(7) 14Jii (MH 14-415 to MH 14-411) High Flow Projections Only
(8) 14Jv (MH 14-407 to MH 14-134) High Flow Projections Only
(9) 14K (MH 14-134 to MH 14-102) High Flow Projections Only
(10) 14Li (MH 14-362 to MH 14-358)
(11) 14Lii (MH 14-358 to MH 14-356) High Flow Projections Only

(12) 14Mii (MH 14-345 to MH 14-333)
(13) 14Miii (MH 14-333 to MH 14-323) High Flow Projections Only
(14) 14Miv (MH 14-333 to MH 14-323)
(15) 14N (MH 14-323 to MH 14-315)
(16) 14Oi (MH 14-315 to MH 14-301) High Flow Projections Only

5. Potential Options for Rerouting Flow from PS 14 to Mendota WWTP

Flow balancing between watersheds or within watersheds may prove to be an important consideration for the District in the future to meet watershed-related objectives. Therefore, it may become desirable to route flow from DeForest, Waunakee, and/or the north side of Madison to a treatment facility other than NSWTP, which would in turn provide relief to PS 13 and ultimately PS 10 and PS 7. One possible option for this type of diversion from PS 13 would be to construct a Mendota WWTP near the Yahara River where it enters Lake Mendota. Some or all of the flow from PS 14 could then be rerouted to the Mendota WWTP instead of NSWTP, thereby returning treated effluent closer to its point of origin and providing a source of additional base flow to the Yahara River from just above Lake Mendota and following downstream. Diversion to a Mendota WWTP would serve the purpose of providing relief to PS 13, PS 10, and PS 7. The projected year 2060 average daily flow from PS 14 would be 6.89 mgd. The resulting peak hourly flow using the MDC would be 20.3 mgd.

6. System and Electrical Redundancy Review

PS 14 is served from two feeds from separate MG&E substations (HKP 1307 and AMN 1311). The two services feed a MMSD owned automatic transfer switch (G&W switch) at the primary (utility) voltage. The switch always seeks an active power feed and in turn feeds a bank of utility transformers that convert the power to the pumping station’s utilization voltage of 480 volts. Although redundant feeds power the site, failure of any transformer in the bank of transformers or of the switch will result in a loss of power to the station.

This pumping station is not provided with a standby power generator nor does the station have a permanent connection for a portable generator. However, MMSD does have a portable generator large enough to power any of the pumps at this pumping station. As with any time a generator is used, extreme caution must be taken to prevent any possible back-feeding of power into the utility's system. The pumping station's power system redundancy and standby power options will be reviewed during the planning and design process for the station's rehabilitation.

According to the March 13, 2007, MMSD Emergency Response Manual, during a service outage there is adequate storage in the interceptors and local sewers upstream of this pump station to prevent any basement back-ups or sewer overflows for six hours under normal flow conditions and two hours under high flow conditions.

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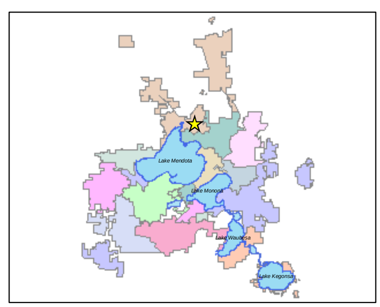
PS 14-School Road
5000 School Road, Madison

Originally Constructed	1971
Upgrades	19,962,008
Capacity (2008)	
Maximum (mgd)	15.6
Firm (mgd)	15.0

Pumps	Q (gpm)	H (ft)	Motor (HP)	Year On-Line
14A	7,200	24	60	2008
14B	7,200	24	60	1971
14C	10,800	29	100	1971

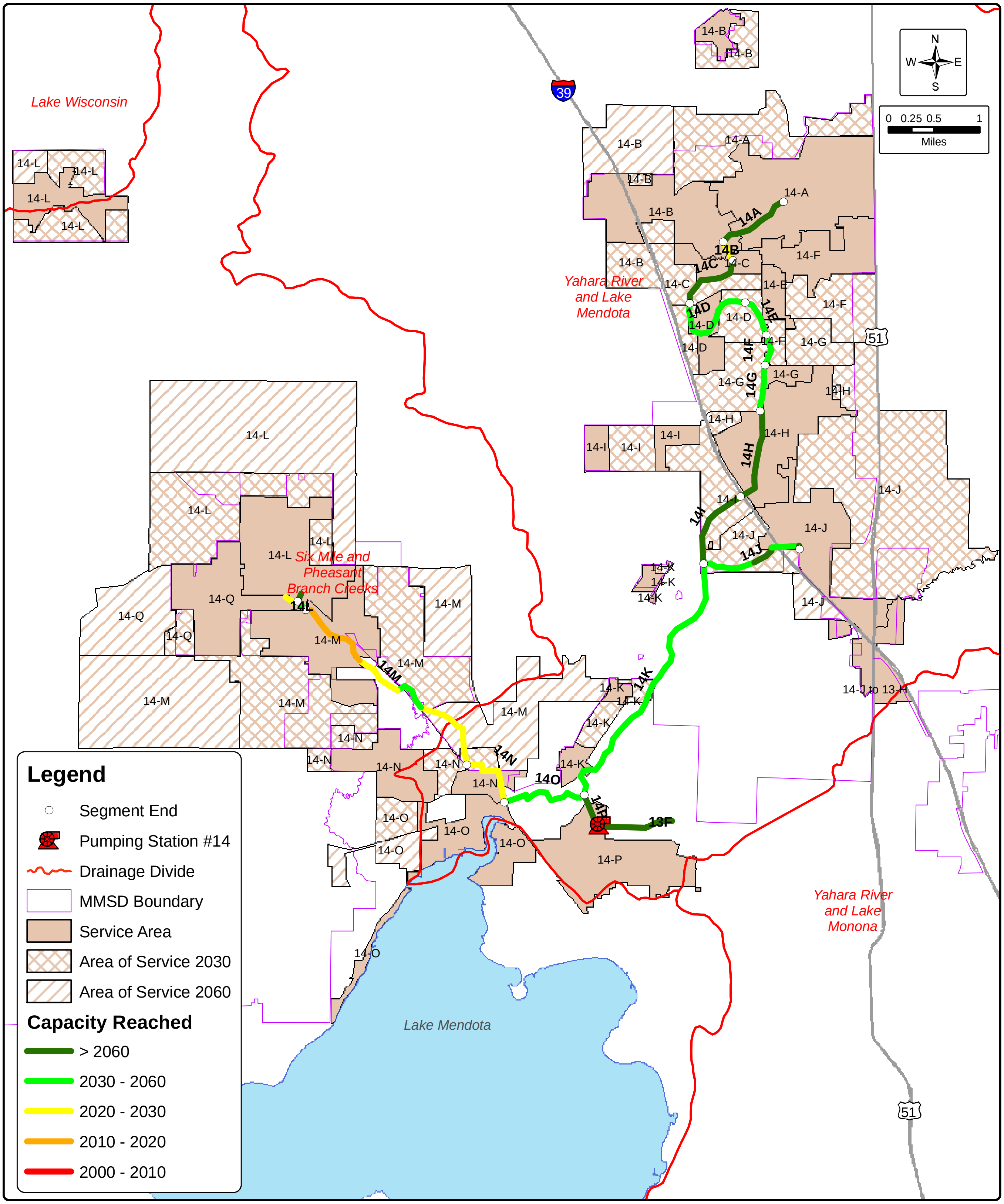


PS 14 - 5000 SCHOOL ROAD
CITY OF MADISON

CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT

STRAND ASSOCIATES, INC.® ENGINEERS

FIGURE 2.16-1
1-547.001



S:\@SAI\501--550\547\001\Data\GIS\Figures\Figure 2.16-2 PS 14 Drainage Area 8x11.mxd

PS 14 - 5000 SCHOOL ROAD
CITY OF MADISON

INTERCEPTOR AND FORCE MAIN ANALYSIS
CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT

STRAND ASSOCIATES, INC.® ENGINEERS

FIGURE 2.16-2
1-547.001

2.17 PS 15–2115 ALLEN BOULEVARD, CITY OF MADISON (MARSHALL PARK)

A. Areas Served by Pumping Station

PS 15 conveys gravity drainage from sewers within its service area and receives pumped discharge from several small pumping stations. Figure 2.17-1 shows the location of the station and highlights some of its features. Areas of the District that comprise the PS 15 service area include portions of the City of Madison, the City of Middleton and the Town of Westport. Routine operation of PS 15 is to pump to the WI upstream of PS 8, while the option exists to pump directly to PS 16 for the purpose of flow diversion. Flow to PS 15 may also undergo gravity diversion to PS 5 via flow through a gate in MH 05-102A near PS 15. PS 15 collects drainage primarily from the WI. Figure 2.17-2 shows the area of the District served by PS 15 as well as the force main and interceptor capacity analysis. The WI along Lake Mendota and above Pumping Station 5 is in need of rehabilitation. As an alternative to rehabbing a portion of this sewer, the District contemplated routing this flow from service area 5A to PS 15. However, it is now likely that the line will be rehabbed to maintain redundancy for PS 15; retaining this route keeps the possibility of sending some of the PS 15 flow to PS 5. This project was included in the 2009 MMSD Capital Projects Budget (832-00-70)

The PS 15 service area lies almost entirely in the Yahara Lakes watershed. A very small area lies in the Black Earth Creek (Wisconsin River) watershed.

B. Description of Pumping Station

1. History of Station

PS 15, located at 2115 Allen Boulevard in Madison, was constructed and placed into service in 1974. A major pumping equipment revision project was completed in 1983, which included one new pump and motor. A new telemetry system was installed in 1984 and upgraded with radio telemetry in 1990 (radio subsequently replaced in 2000). The barminutor was removed in 1989. A new station control center was installed in 2003.

2. Current Design Capacities and Limitations

The maximum pumping capacity of PS 15 is approximately 8.8 mgd, whereas the firm capacity is 5.8 mgd. Data from the last decade shows an average flow rate for PS 15 of approximately 1.2 mgd. The 2060 projected peak hourly flow for PS 15 is approximately 7.57 mgd or 1.776 mgd greater than the firm capacity of the station. The projected peak hourly flow is not projected to exceed the station's maximum capacity before the year 2060.

The 2008 Condition Assessment gave the pumping station an overall rating score of 18.00, making its ordinal ranking the fifth highest priority for improvements among the District's 17 pumping stations. Improvements at this pumping station are scheduled for construction in five to ten years. The rating for each pumping station element was as follows:

Adequacy	Rating
Maximum Flow Capacity	Excellent
Firm Flow Capacity	Excellent
Power System Redundancy	Poor
Mechanical Condition	Good
Building and Structural Condition	Poor
Electrical Condition	Adequate
Criticality Factor	1.2
Overall Score	18.00

3. Additional Near-Term Planned Improvements (2010-2020)

As shown in the Condition Assessment table, PS 15 has been identified as in need of a rehabilitation project to address power redundancy, electrical system condition, and structural condition. The 2009 MMSD Capital Budget identifies the project for rehabilitation to occur in 2015 and 2016. Capacity issues should also be addressed during the rehabilitation project.

4. Long-Term Considerations (2020-2060)

a. Pumping Station: Based on the current configuration of the MMSD conveyance system, firm capacity for this pumping station will be reached in the year 2035 under low flow projections, but may have been reached by the year 2009 for high flow projections. Maximum pumping capacity would not be reached until after the year 2060 under either flow projection. Any proposed capacity increase should be considered as a part of the near-term improvements for the pumping station upgrade. Improvements to this station are included in the 2009 MMSD Capital Projects Budget (865-00-50).

b. Force Mains: The force main for PS 15 is not expected to reach capacity prior to the year 2060 under either flow projection.

c. Interceptors: Interceptor segment 15A (siphon) is projected to reach capacity in 2041 under high flow projections and 2056 under low flow projections, and 15Ci is projected to reach capacity at about year 2028 under high flow projections and 2058 under low flow projections.

5. Alternative Operations of PS 15: PS 8 and PS 16

PS 15 is capable of pumping flow either to PS 8, which is on the WI (normal operation) or to PS 16, which leads to the NSVI (alternate operation)., From the time PS 15 was placed into service until the construction of PS 16, PS 15 discharged to the WI. When PS 16 was placed into service, a newly constructed force main between the stations was utilized and PS 15 flow was pumped to PS 16. In response to odor complaints in the vicinity of PS 16 and because of the significantly higher operating costs associated with pumping from PS 16 to PS 12, MMSD began again routing flow from PS 15 to PS 8 in 1996.

6. System and Electrical Redundancy Review

PS 15 is served from two separate feeds from an MG&E Substation (PHB 1305 and PHB 1306). The two services feed a MMSD owned automatic transfer switch (G&W switch) at the primary (utility) voltage. The switch always seeks an active power feed and in turn feeds a utility transformer that converts the power to the pumping station's utilization voltage of 480 volts. Although redundant feeds power the site, failure of the transformer or of the switch will result in a loss of power to the station.

This pumping station is not provided with a standby power generator nor does the station have a permanent connection for a portable generator. However, MMSD does have a portable generator large enough to power either of the two smaller pumps at this pumping station. The backup generator at PS 17 could be used to power the larger pump if it was available (PS 17 with utility power). As with any time a generator is used, extreme caution must be taken to prevent any possible back-feeding of power into the utility's system. The pumping station's power system redundancy and standby power options will be reviewed during the planning and design process for the station's rehabilitation.

According to the March 13, 2007, MMSD Emergency Response Manual, during a service outage there is adequate storage in the interceptors and local sewers upstream of this pump station to prevent any basement back-ups or sewer overflows for one hour under normal flow conditions and 30 minutes under high flow conditions.

S:\@SAI\501--550\547\001\Data\GIS\Figures\Figure 2.17-1 PS15.mxd

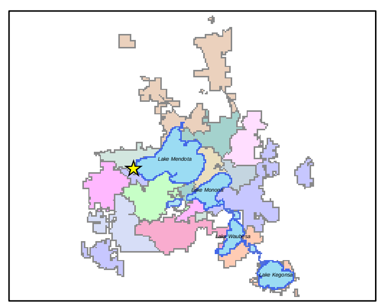
PS 15-Allen Boulevard
2115 Allen Boulevard, Madison

Originally Constructed	1975
Upgrades	1983

Capacity (2008)

Maximum (mgd)	5.8
Firm (mgd)	8.8

Pumps	Q (gpm)	H (ft)	Motor (HP)	Year On-Line
15B	3,000	68	100	1975
15A	4,000	76	100	1975
15C	6,100	100	200	1982

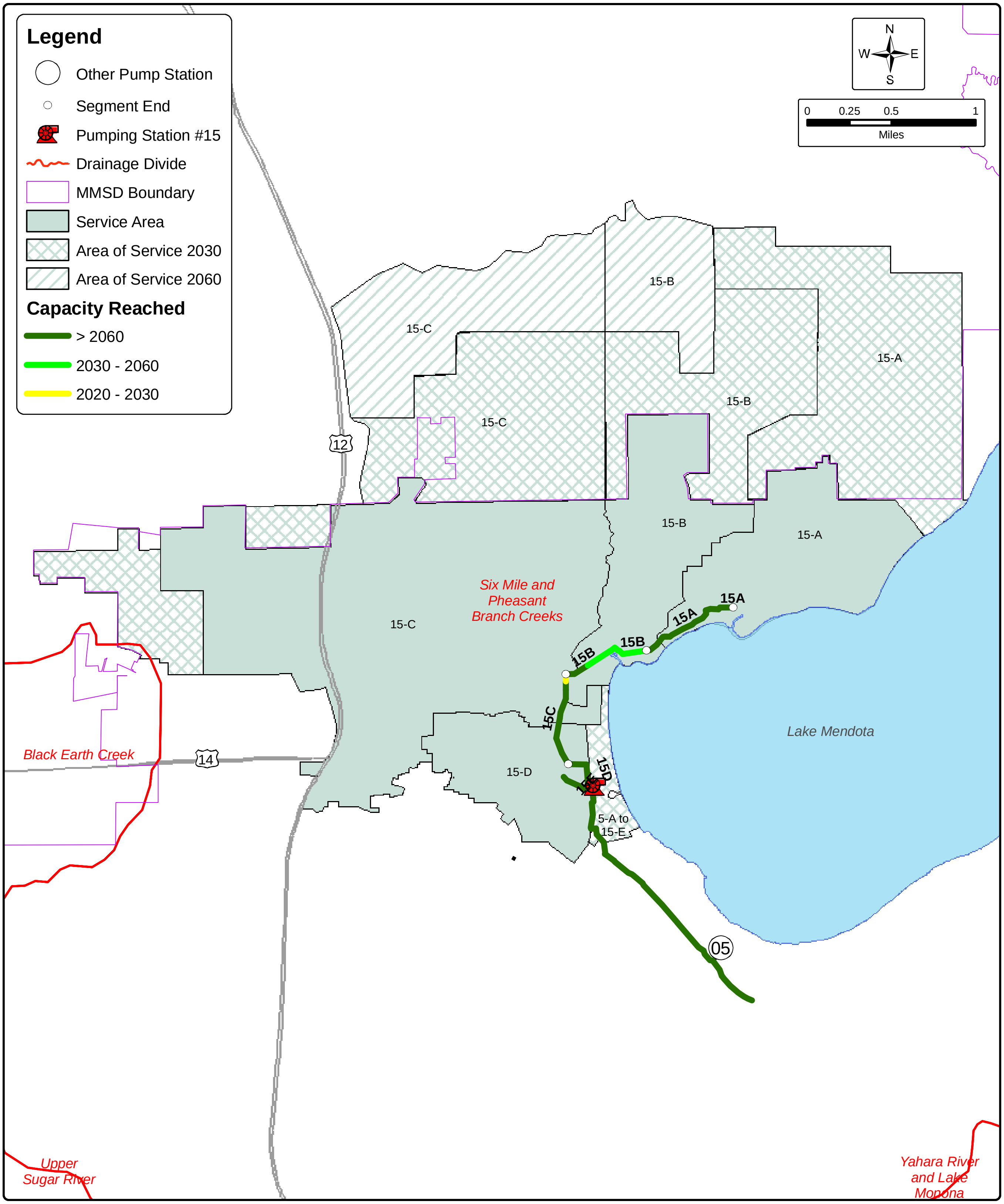


PS 15 - 2115 ALLEN BOULEVARD
CITY OF MADISON

CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT

STRAND ASSOCIATES, INC.® ENGINEERS

FIGURE 2.17-1
1-547.001



S:\@SAI\501--550\547\001\Data\GIS\Figures\Figure 2.17-2 PS 15 Drainage Area 8x11.mxd

PS 15 - 2115 ALLEN BOULEVARD
CITY OF MADISON

INTERCEPTOR AND FORCE MAIN ANALYSIS
CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT

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FIGURE 2.17-2
1-547.001

2.18 PS 16-1303 GAMMON ROAD, CITY OF MIDDLETON

A. Areas Served by Pumping Station

Under current routine operation, PS 16 is used exclusively to convey gravity drainage from sewers within its service area. Figure 2.18-1 shows the location of the station and highlights some of its features. Areas of the District that comprise the PS 16 service area include portions of the City of Middleton and the far west side of the City of Madison. Routine operation of PS 16 is to pump to the NSVI Mineral Point Extension upstream of PS 12. Flow to PS 16 may also undergo gravity diversion to PS 5 via overflow of a dam in MH 5-230 near PS 16. The amount of flow that may be transferred through this route is limited. Figure 2.18-2 shows the area of the District served by PS 16. An alternate conveyance option is for PS 16 to receive pumped flow from PS 15. Figure 2.18-2 also shows the force main and interceptor capacity analysis.

B. Description of Pumping Station

1. History of Station

PS 16, located at 1303 Gammon Road in Middleton, was constructed and placed into service in 1982. Because of the age of this pumping station, no significant modifications have been made to it. However, odor complaints have been received from the public in the vicinity of this pumping station since it was placed into service. As a result, odor control and ventilation projects have been ongoing at PS 16 since 1983. In addition, in 1996, flow from PS 15 was routed to PS 8 instead of PS 16 to reduce the age of the wastewater and associated odors at PS 16. This has reduced the concern regarding odors and allowed the District to reduce the operation of the chemical odor control unit. Other projects at this station include installation of a new telemetry system in 1984, radio telemetry units in 1990 (replaced in 2000), and replacement of the station control center and upgrades to the starter controls in 2009.

2. Current Design Capacities and Limitations

The maximum pumping capacity, as well as the firm capacity, of PS 16 is 18.7 mgd. Data from 1996-2007 shows an average flow rate for PS 16 of less than 2 mgd.

Anticipated growth on the west side of the District will serve to increase the flow to PS 16 over time, and the City of Middleton and the west side of Madison are expected to expand significantly in the future. Despite this, PS 16 exhibits no capacity-related concerns through 2060 based on either the low or high flow projections provided that PS 15 continues to pump to the WI and PS 8.

The 2008 Condition Assessment gave this pumping station an overall rating score of 9.45, making its ordinal ranking the fourteenth highest priority for improvements among the District's 17 pumping stations. No major improvements are scheduled during the next twenty years. The rating for each pumping station element was as follows:

Adequacy	Rating
Maximum Flow Capacity	Excellent
Firm Flow Capacity	Excellent
Power System Redundancy	Good
Mechanical Condition	Good
Building and Structural Condition	*Excellent-(*Good)* *See text below*
Electrical Condition	Good
Criticality Factor	1.05
Overall Score	9.45

A review of the station in 2007 noted there were signs of masonry cracking in a couple of locations in the structure. The District may wish to review its assessment of building condition based on this review. Changing the rating from excellent to good would change the overall score from 9.45 to 10.5 and would change its priority from 14 to 13. An ongoing project at PS 16 is replacing the older obsolete control equipment. Work will be completed prior to 2010.

The only interceptor project associated with PS 16 that is scheduled is a relief sewer project on the WI Gammon Extension along Middleton Street upstream of PS 16. A portion of this interceptor (Voss Parkway and Fortune Drive) was relieved in 2002. Based on current (2008) MMSD staff assessments, this addressed the most critical needs. In the near term, the remaining sewers in this segment may be adequate.

3. Additional Near-Term Planned Improvements (2010-2020)

The remaining portion of the WI Gammon Extension relief may be completed between 2010 and 2020 if the capacity concerns over the portion not replaced are still determined to exist. This decision will be influenced, in part, by the timing for the District’s Lower Badger Mill Creek Interceptor. This interceptor will ultimately divert flows from the City of Madison’s South Point Lift Station to PS 17, rather than to the WI Gammon Extension.

4. Long-Term Considerations (2020-2060)

   a. Pumping Station: No additional capacity improvements are required for this pumping station prior to the year 2060 under all flow projections and even with the potential for PS 15 to pump to PS 16.

   b. Force Mains: No additional capacity is required for the PS 16 force main through the year 2060.

   c. Interceptors: The following interceptor segments are projected to reach capacity between 2020 and 2060:

      (1) 16Aii (MH 05-315 to MH 05-310)
      (2) 16Aiv (MH 05-306 to MH 05-236)

5. PS 15 Operational Impacts on PS 16

PS 15 is capable of pumping flow either to PS 16 or PS 8, although current routine operation is for PS 15 to pump to PS 8.

6. System and Electrical Redundancy Review

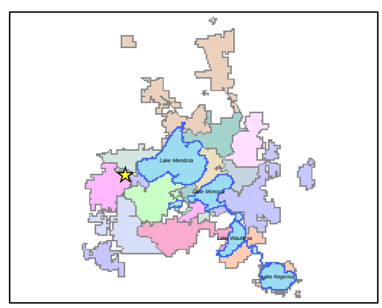
Pumping Station 16 is served from two separate feeds from an MG&E Substation (PHB 1313 and PHB 1314). The services from MG&E feed opposite ends of a double-ended switchgear lineup, feeding two separate 13.8 kV to 2400 volt transformers. The 2400 volt gear feeds the two opposite ends of the motor starter lineup across the room. Therefore, within the station, the power system is relatively robust and redundant. No provision has been made at this pumping station for standby power or for a portable generator connection. The pump motors are 500 hp at 2400 volts and the District does not have a portable generator set, including the one located at PS 17, capable of powering this pumping station. The redundancy of power to this pumping station should be reviewed at some point in the future, as the station is fed by two feeds from opposite busses of the same utility substation, making routing of power to the station and important consideration

According to the March 13, 2007, MMSD Emergency Response Manual, during a service outage there is adequate storage in the interceptors and local sewers upstream of this pump station to prevent any basement back-ups or sewer overflows for four hours under normal flow conditions and two hours under high flow conditions.

S:\@SAI\501--550\547\001\Data\GIS\Figures\Figure 2.18-1 PS16.mxd

PS 16-Gammon
1303 Gammon Road, Middleton

Originally Constructed				1982
Upgrades				-
Capacity (2008)				
Maximum (mgd)				18.7
Firm (mgd)				18.7
Pumps	Q (gpm)	H (ft)	Motor (HP)	Year On-Line
16A	7,000	182	500	1982
16B	7,000	182	500	1982
16C	7,000	182	500	1982



PS 16 - 1303 GAMMON ROAD
CITY MIDDLETON

CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT

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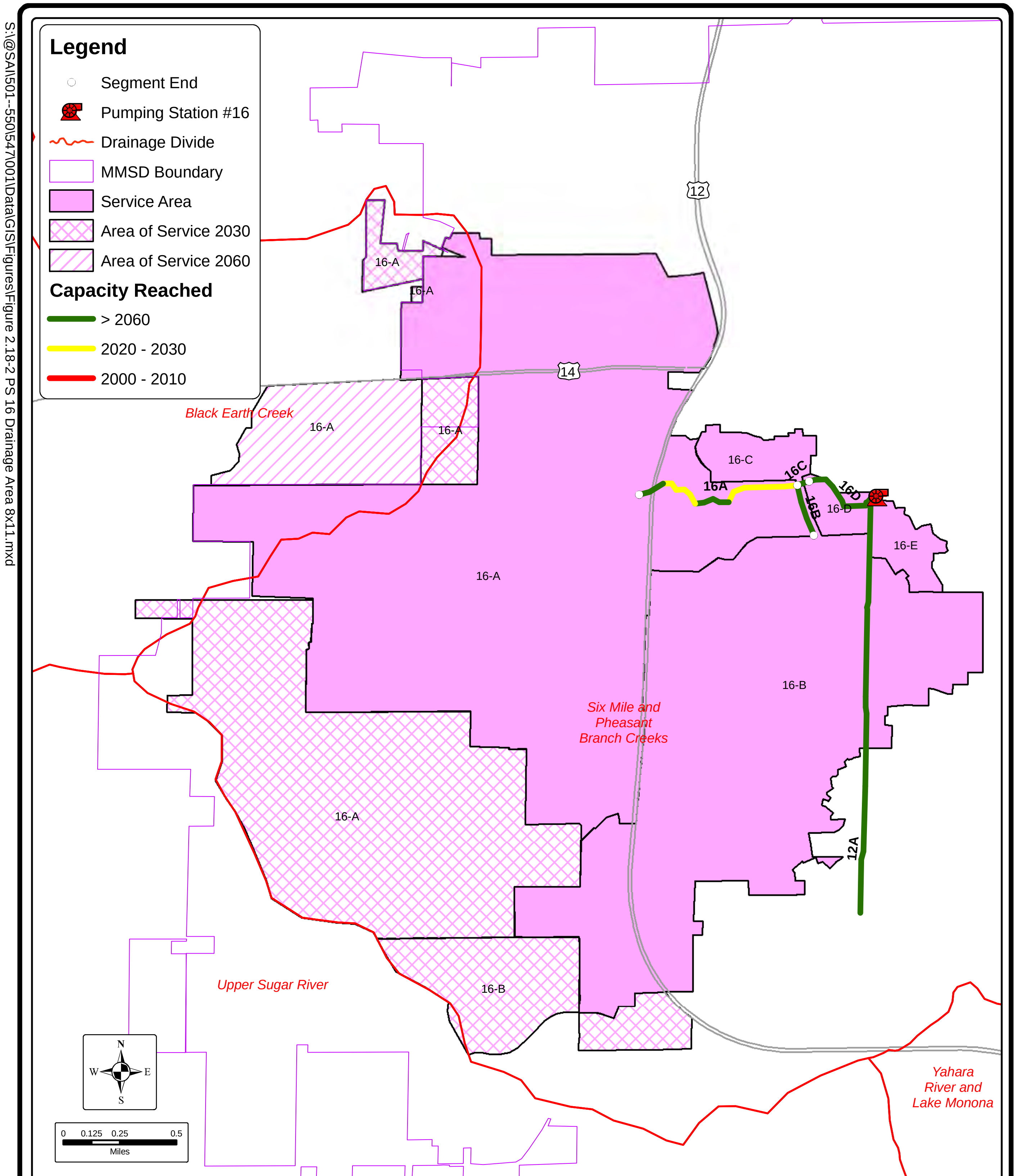
FIGURE 2.18-1
1-547.001



PS 16 - 1303 GAMMON ROAD
CITY OF MIDDLETON

INTERCEPTOR AND FORCE MAIN ANALYSIS
CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT

STRAND ASSOCIATES, INC.® ENGINEERS

FIGURE 2.18-2
1-547.001

2.19 PS 17–407 BRUCE STREET, CITY OF VERONA

A. Areas Served by Pumping Station

PS 17 exclusively conveys gravity drainage from sewers within its service area. Figure 2.19-1 shows the location of this station and highlights some of its features. The area of the District that comprises the PS 17 service area is located in the City of Verona. PS 17 collects drainage from its service area and pumps to the NSVI. Figure 2.19-2 shows the area of the District served first by PS 17 as well as the force main and interceptor capacity analysis.

B. Description of Pumping Station

1. History of Station

PS 17, located at 407 Bruce Street in Verona, was constructed and placed into service in 1996. This pumping station, which replaced Verona's WWTP, is one of MMSD's newest stations. Because of the age of this pumping station, no major modifications have been made to it as of yet. However, a recent change in how the station is controlled allows for dual pumping should flows become high enough. This change increased the pumping stations firm and maximum capacity and required the utility to install a larger transformer (300 kVA versus the original 150 kVA).

2. Current Design Capacities and Limitations

The maximum pumping capacity, as well as the firm capacity, of PS 17 is 4.6 mgd. Data from the last decade shows an average flow rate for PS 17 of approximately 0.7 mgd. Based on the flow projections, this pumping station was projected to reach its revised capacity of 4.6 mgd by the year 2007 under high flow projections and 2011 under low flow projections. Note that this projection assumes that all flow in the Lower Badger Mill Creek valley flows to PS 17. This interceptor has not yet been fully constructed. Capacity improvements will be required prior to completion of this interceptor. At present, flow is diverted from the upper portions of this valley to the NSVI above PS 12 via the Midtown Extension. Assuming that the entire LBMCI is constructed, the overall capacity deficit for the high flow projections is 8.98 mgd unless flow is diverted from this pumping station to other pumping stations or to a satellite treatment plant.

The 2008 Condition Assessment gave this pumping station an overall rating score of 16.80, making its ranking the eighth highest priority for improvements among the District's 17 pumping stations. Improvements are scheduled for construction in five to ten years. The rating for each pumping station element was as follows (note that these ratings assume the LBMCI is fully constructed – until then, the flow capacity is considered adequate):

Adequacy	Rating
Maximum Flow Capacity	Very Poor
Firm Flow Capacity	Very Poor
Power System Redundancy	Excellent
Mechanical Condition	Adequate

Adequacy	Rating
Building and Structural Condition	Excellent
Electrical Condition	Excellent
Criticality Factor	1.05
Overall Score	16.80

3. Additional Near-Term Planned Improvements (2010-2020)

There are no near-term planned improvements for PS 17 except a project to address potential capacity issues; however, the mechanical condition of the pumps is somewhat suspect and this problem will likely be addressed at the same time Based on current and future sewer segments tributary to PS 17, this pumping station is projected to reach its revised capacity of 4.6 mgd when the LBMCI is fully constructed and connects the flow from the City of Madison development above Midtown Road to PS 17.

The 2009 MMSD Capital Projects Budget includes a project to upgrade capacity at PS 17 (867-00-50), scheduled for completion in 2014.

The 2009 MMSD Capital Budget includes a line item to construct additional segments of the Lower Badger Mill Creek Interceptor beginning in 2014.

4. Long-Term Considerations (2020-2060)

a. Pumping Station: Based on the current configuration of the MMSD conveyance system, firm capacity for this pumping station will not be reached unless the upstream flow diversions occur, which is not expected before 2014.

b. Force Mains: The force main segments for PS 17 are expected to reach capacity prior to 2060:

(1) 17FMi (PS17 to 17-14450)
(2) 17 FMii (17-14450 to MH 12-110)

c. Interceptors: The existing interceptor segment 17A has adequate capacity through the year 2060. Interceptor segments 17B and 17C are planned to be constructed in 2014.

5. Potential Options for Rerouting Flow from PS 12 and PS 17 to Sugar River WWTP

Flow-balancing between watersheds may prove to be an important consideration for the District in the future to meet watershed-related objectives. Therefore, it may become desirable to route flow from the City of Verona and the west side of the City of Madison to a treatment facility other than NSWTP. One possible option to achieve this flow-balancing would be to construct a Sugar River WWTP on the southwest side of the Madison area that would discharge into the Sugar River Drainage Basin. Flow from PS 17 and potentially a portion of PS 12 flow would then be rerouted to the Sugar River WWTP instead of NSWTP, thereby returning treated effluent closer to its point of origin. The gravity drainage service areas for these pumping stations are in the Sugar River basin. A byproduct of this change would be that PS 17 would potentially pump directly to the new WWTP instead of to PS 12. In addition, future capacity issues may be resolved at PS 17 depending on how the new sewers associated with the Sugar River WWTP are routed.

The estimated average daily flow generated by the gravity drainage areas of PS 17, which are located within the Sugar River Basin, would be as follows:

PS 17	2030 Low	2030 High	2060 High
17A	0.52	0.8	1.05
17B	0.25	0.78	0.78
17C	0.76	1.06	1.06
17D	0.69	0.78	1.38
ADF	2.22	3.42	4.27
PHF	7.83	11.26	13.58

One alternative that would alleviate the size of the capacity increase for PS 17 would be to route interceptor segments 17B and 17C to a new Upper Sugar River Treatment Plant located near the wetlands area as CTH PD crosses the Sugar River. This would provide the following average daily flow relief potential to PS 17:

CTH PD	2030 Low	2030 High	2060 High
17B	0.25	0.78	0.78
17C	0.76	1.06	1.06
ADF	1.01	1.84	1.84
PHF	4.03	6.68	6.68

This would reduce the peak hourly flow for PS 17 from approximately 13.58 mgd to 8.45 mgd. This would reduce the additional capacity requirements from 8.98 mgd to 3.85 mgd. Diverting the flow from 17 B and 17 C from PS 17 will delay the need for capacity relief in the PS 17 force main beyond 2060.

6. System and Electrical Redundancy Review

PS 17 is served by a single utility feed from Alliant Energy (VER N88) and a “permanent” portable standby generator currently sized to handle the peak hourly flow of 4.6 mgd. The electrical capacity will need to be reviewed as flows to the station increase. Although the portable generator at PS 17 is intended to be the permanent backup power source for the pumping station, since it is portable, it could be used elsewhere within the District in the event of an emergency. However, doing so puts this pumping station at risk and without a backup power feed. This issue and the possibility of a second permanent utility service should be explored periodically in the future and when station capacity must increase.

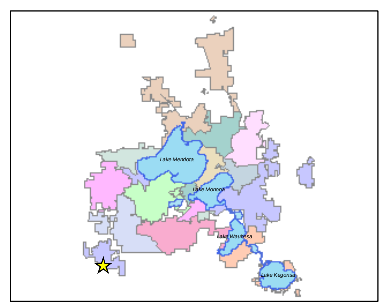
According to the March 13, 2007, MMSD Emergency Response Manual, during a service outage there is adequate storage in the interceptors and local sewers upstream of this pump station to prevent any basement back-ups or sewer overflows for two hours under normal flow conditions and one hour under high flow conditions.

S:\@SAI\501--550\547\001\Data\GIS\Figures\Figure 2.19-1 PS17.mxd

PS 17-Verona
704 Bruce Street, Verona

Originally Constructed	1996
Upgrades	-
Capacity (2008)	
Maximum (mgd)	4.6
Firm (mgd)	4.6

Pumps	Q (gpm)	H (ft)	Motor (HP)	Year On-Line
17A	3,200	115	100	1996
17B	3,200	115	100	1996
17C	3,200	115	100	1996

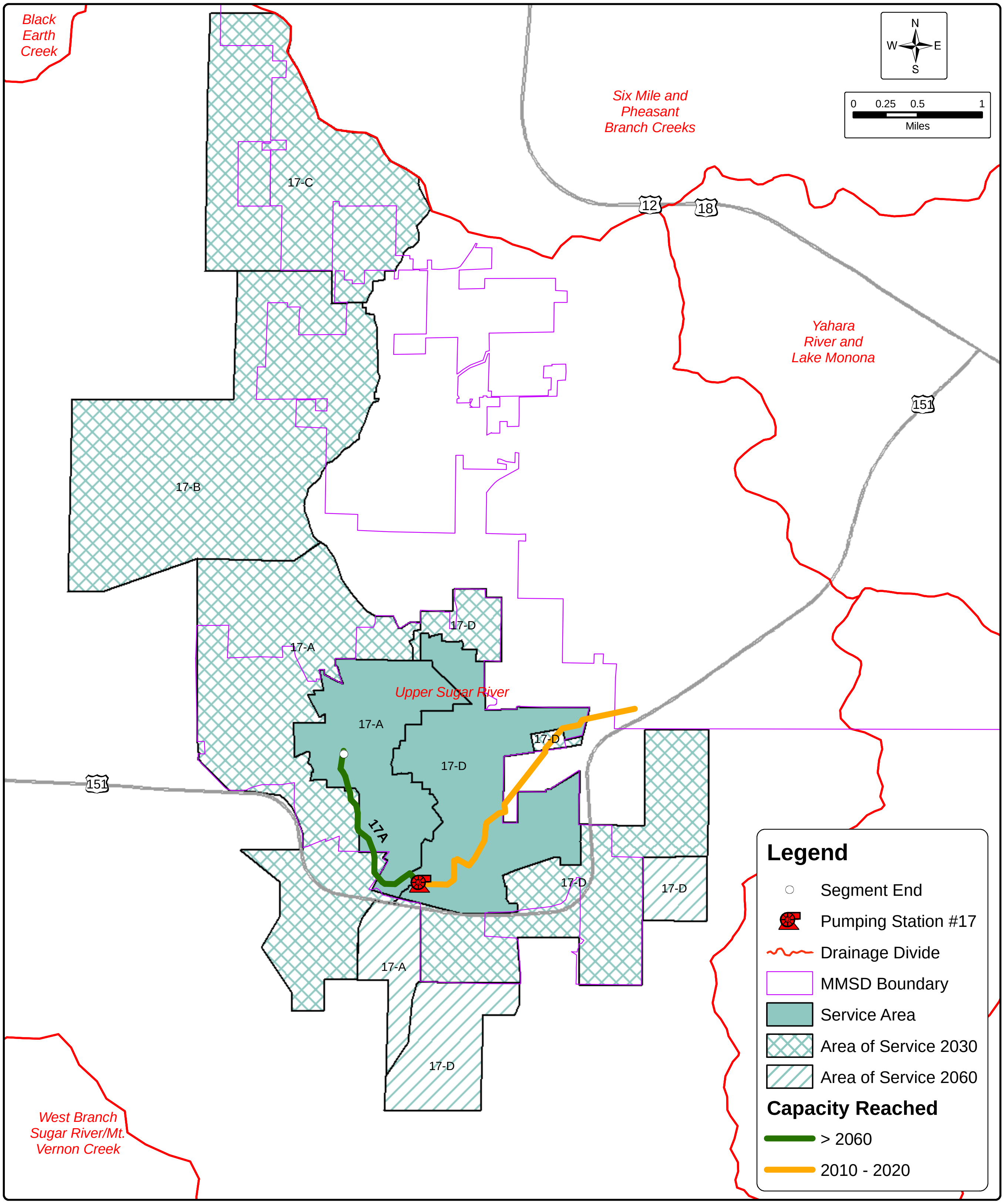


PS 17 - 704 BRUCE STREET
CITY OF VERONA

CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT

STRAND ASSOCIATES, INC.® ENGINEERS

FIGURE 2.19-1
1-547.001



PS 17 - 405 BRUCE STREET
CITY OF VERONA

INTERCEPTOR AND FORCE MAIN ANALYSIS
CONVEYANCE FACILITIES ANALYSIS
MADISON METROPOLITAN SEWERAGE DISTRICT

STRAND ASSOCIATES, INC.® ENGINEERS

FIGURE 2.19-2
1-547.001

SECTION 3
EXISTING CONVEYANCE FACILITY ANALYSES

3.01 EXISTING AND FUTURE SYSTEM NEEDS WITH NO SATELLITE OPTIONS

A. Pumping Stations Summary

Tables 3.01-1 through 3.01-9 present the review of pumping station capacities for the years 2000, 2010, 2020, 2030, and 2060. Appendix A summarizes the development of these flows based on CARPC analysis. A summary of the CARPC report is also included in Appendix A. These tables were prepared based on the currently available operating modes of the conveyance system and all flows being conveyed to the NSWTP.

1. Year 2000-2010 System Needs

The analysis of the pumping stations summarized in Tables 3.01-1, 3.01-2, and 3.01-3 shows the following stations to be of concern from a capacity consideration:

a. PS 7 Firm Capacity (High Flow Projections)
b. PS 11 Firm Capacity (High Flow Projections)
c. PS 12 Firm Capacity
d. PS 15 Firm Capacity (High Flow Projections)
e. PS 17 Firm and Maximum Capacity (High Flow Projections Only). Based on transfer of flows from 17 B and 17 C currently pumped to NSVI

These analyses were done by comparing the identified peak hourly flow generated using the Madison Design Curve and both the maximum and firm pumping station capacities.

The CTFM project that transferred PS 1 flow from PS 6 to PS 2 alleviated the concerns regarding PS 7 peak hourly flow capacity. With the 2008 mode of operation (*97* percent of PS 1 flow to PS 2 and *3* percent of PS 1 flow to PS 6), the need for potential capacity increases at or near PS 7 was delayed. The analyses presented in Tables 3.01-1 through 3.01-4 route all of PS 1 flows to PS 2.

A 50-million-gallon storage basin was constructed at the site of the former MMSD sludge lagoons to address the limited capacity of the force main to Badfish Creek. The pumps that pump to Badfish Creek are limited to a capacity of 75 mgd by force main pressure considerations. The maximum permitted flow to Badger Mill Creek is 3.6 mgd, resulting in a total maximum effluent pumping rate of 78.6 mgd.

2. Year 2010-2020 System Needs

Without the construction of satellite plants to potentially divert flow away from existing pumping stations, the analysis presented in Tables 3.01-4 and 3.01-5 shows the following stations will be of concern from a capacity consideration by the year 2020:

a. PS 7 Firm Capacity
b. PS 7 Maximum Capacity (High Flow Projections Only)
c. PS 11 Firm Capacity
d. PS 11 Maximum Capacity (High Flow Projections Only)

e. PS 12 Firm Capacity
f. PS 12 Maximum Capacity (High Flow Projections Only)
g. PS 13 Firm Capacity
h. PS 13 Maximum Capacity (High Flow Projections Only)
i. PS 15 Firm Capacity (High Flow Projections Only)
j. PS 17 Firm and Maximum Capacity

3. Year 2020-2030 System Needs

Without the construction of satellite plants to potentially divert flow away from existing pumping stations, the analysis presented in Tables 3.01-6 and 3.01-7 shows the following stations will be of concern from a capacity consideration by the year 2030:

a. PS 7 Firm Capacity and Maximum Capacity
b. PS 9 Firm and Maximum Capacity (High Flow Projections Only)
c. PS 11 Firm and Maximum Capacity
d. PS 12 Firm Capacity
e. PS 12 Maximum Capacity (High Flow Projections Only)
f. PS 13 Firm and Maximum Capacity
g. PS 14 Firm and Maximum Capacity (High Flow Projections Only)
h. PS 15 Firm Capacity (High Flow Projections Only)
i. PS 17 Firm and Maximum Capacity

4. Year 2030-2060 System Needs

Without the construction of satellite plants to potentially divert flow away from existing pumping stations, the analysis presented in Tables 3.01-8 and 3.01-9 shows the following stations will be of concern from a capacity consideration by the year 2060:

a. PS 4 Firm and Maximum Capacity (High Flow Projections Only)
b. PS 7 Firm and Maximum Capacity
c. PS 9 Firm and Maximum Capacity
d. PS 11 Firm and Maximum Capacity
e. PS 12 Firm and Maximum Capacity
f. PS 13 Firm and Maximum Capacity
g. PS 14 Firm and Maximum Capacity
h. PS 15 Firm Capacity
i. PS 17 Firm and Maximum Capacity

5. Specific Capacity Needs (PSs 4, 7, 11, 12, 13, 14, 15, and 17)

The above sections summarize the pumping stations' needs for MMSD through the year 2060. This section will address the projected timing for capacity exceedances and the overall firm capacity addition required to meet the pumping station needs under the highest projected flow scenario. Descriptions of potential projects and timing for each of the pumping stations with capacity deficits follow.

a. PS 4

PS 4 is projected to reach capacity under high flow projections about the year 2047. The projected firm capacity deficit is 0.09 mgd. This is a minor adjustment to the overall capacity that should be reviewed with the next project that would replace the current pumps in PS 4. MMSD in its 2009 Capital Projects Budget (853-00-50) identified a project for rehabilitation of PS 4 to begin in 2017 and be completed in 2019.

b. PS 7

PS 7 is projected to have reached firm capacity under high flow projections in 2005 and will reach capacity in 2011 under low flow projections. The maximum identified firm capacity deficit is 33.27 mgd occurring under the high flow projections for the year 2060. The maximum pumping capacity is projected to be exceeded between 2012 and 2027.

The District 2009 Capital Funds Project Budget has two projects: 868-00-51 (PS 18) and 868-00-52 (PS 18 FM) that will address the capacity needs of PS 7.

c. PS 9

The firm pumping capacity for PS 9 is projected to be exceeded by about the year 2022 under high flow projections and 2041 under low flow projections. The 2009 MMSD Capital Projects Budget included a project (853-00-50) for PS 9 revisions. This project is scheduled to begin in 2017 and be completed in 2019. A potential revision in firm pumping capacity to address the projected 1.89 mgd firm capacity deficit should be evaluated with the design of these revisions.

d. PS 11

PS 11 is projected to reach its firm capacity between 2006 and 2010. Maximum pumping capacity is projected to be exceeded between 2016 and 2026.

The 2009 Capital Projects Budget (861-00-52) identified a project to address capacity and condition needs. This project is scheduled to begin in 2013 and be completed in 2016. The projected firm capacity deficit without construction of a satellite plant would be 19.32 mgd under the high flow projections for 2060.

e. PS 12

PS 12 is projected to reach its firm capacity between 2005 and 2008 while the maximum pumping capacity is projected to be exceeded between 2019 and 2031.

The 2009 MMSD Capital Projects Budget (861-00-52) includes a project to address capacity and conditions needs for PS 12. This project is scheduled to begin in 2013 and be completed in 2016. The projected maximum deficit under 2060 high flow projections is 15.7 mgd.

f. PS 13

The maximum identified firm capacity deficit is 9.44 mgd under the high flow projections for 2060. Exceedance of the revised firm (2008) and maximum pumping capacities based on the CARPC flow projections would occur between 2010 and 2020. The 2009 MMSD Capital Projects Budget includes a project (863-00-50) that addresses the condition needs for the pumping station. A review of the need for revision to the firm pumping capacity should be included with the planning for this project. The project is scheduled to begin in 2015 and be completed in 2018.

g. PS 14

The maximum firm capacity deficit is 5.16 mgd. The firm capacity is projected to be exceeded between 2023 and 2038. The 2009 MMSD Capital Projects Budget includes a project (863-00-50) that addresses the condition needs for the pumping station. A review of the need for revision to the firm pumping capacity should be included with the planning of this project. The project is scheduled to begin in 2015 and be completed in 2018.

h. PS 15

PS 15 is projected to reach its firm pumping capacity between 2009 and 2035. The projected firm pumping capacity deficit is 1.77 mgd. The maximum pumping capacity for the station is not exceeded by 2060.

The 2009 MMSD Capital Projects Budget includes a project (865-00-50) that addresses the condition needs for the pumping station. A review of the need for revision to the firm pumping capacity should be included with the planning for this project. The project is scheduled to begin in 2014 and be completed in 2016.

i. PS 17

The firm pumping capacity for PS 17 is projected to be exceeded between 2007 and 2011 based on all the flow in the PS 17 service area reaching PS 17. Under the current operating mode, flows from PS 17 Service Area B and C are pumped to the NSVI-Midtown Extension without passing through PS 17. The projected firm capacity deficit under the 2060 high flow projections is 8.97 mgd. The timing for improvements for this project will be reviewed when MMSD does the planning for the remainder of the Lower Badger Mill Creek Interceptor extension. The 2009 Capital Projects Budget includes an upgrade to PS 17 (867-00-50) to be constructed at the same time the Lower Badger Mill Creek Interceptor extension is constructed. Presently these projects are anticipated to begin in 2013 and be completed in 2014; however, the actual timing is fairly uncertain. In addition, other changes to the pumping station would be required if a Sugar River Treatment Plant were constructed.

B. Force Mains

1. Capacity Review

A review of the capacity of the existing MMSD force mains is summarized in Table 3.01-10. The following force mains have future force main velocities at peak hourly flow in excess of the nominal maximum velocity target of 8 feet per second:

a. CTFM (20-inch PVC and 24-inch DIP Only)
b. PSs 2, 3, and 4 FM-2057 (High Flow Projections Only)
c. PS 7 FM (2024-2050)
d. PS 10 FM-2041 High Flow Projections Only
e. PS 11FM (2025-2048)
f. PS 17FM i-16 inch (2016-2026)
g. PS 17 FMii-20 inch (2031-2060)

The need for any improvements to the common force main serving PSs 2, 3, and 4 should be based on the actual operation of each of the stations rather than the nominal 8 fps criteria used in the force main capacity analysis. The maximum velocity in the PSs 2, 3, and 4 force main would be 8.1 fps.

Capacity issues for the PS 7 force main will be addressed by the PS 18/PS 18 Force Main project scheduled in the 2009 Capital Projects Budget to occur between 2010 and 2013.

The impact of the peak flows for PS 10 will depend on actual growth in the service areas tributary to PS 10. The maximum projected force main velocity for PS 10 would be 8.5 fps at 38.74 mgd. The current firm pumping capacity of this station is 42.5 mgd with a resulting maximum force main velocity of 9.3 fps at the firm pumping capacity of 42.5 mgd.

The PS 11 force main sizing should be reviewed when planning is done for the rehabilitation project scheduled to be completed between 2015 and 2018. This will also allow the District sufficient time to determine if a satellite treatment plant could be constructed that would potentially eliminate the need for any force main capacity improvements. The maximum identified velocity under the projected peak hourly flows for PS 11 would be 9.6 fps.

The need for a PS 17 force main to PS 12 will depend on growth in the PS 17 service area. The 16-inch section of PS 17 force main is projected to reach capacity between 2016 and 2026. The 20-inch section of PS 17 force main is projected to reach capacity between 2031 and 2060. Preliminary planning for a relief force main, if a Sugar River Plant is not constructed, would be included with the proposed upgrade to PS 17, anticipated in conjunction with the construction of the remaining sections of the LBMCI and tentatively scheduled for construction in 2014 (MMSD Project 867-00-50).

2. Age and Condition Review

Figure 3.01-1 shows the age distribution of the MMSD force mains. The general condition of the force mains is good since they tend to operate in a full pipe condition. In early 2009 the PS 6 force main was inspected as a part of an emergency repair and found to be in very good condition despite its being placed in service in 1950. This assessment of condition is based on MMSD inspections of force mains when they are not in service. The portions of the force main reaching age 75 years follow:

a. By 2015 - 11 percent
b. 2016 to 2025 - 3 percent
c. 2026 to 2035 - 30 percent
d. 2036 to 2045 - 13 percent
e. 2046 to 2055 - 8 percent

Of these above values, approximately 13 percent (PS 7 and PS 11) may need relief or replacement because of capacity considerations as a result of the limited pressure capabilities. These will not be required if satellite plants can reduce the flows to PS 7 and PS 11, thereby reducing the flow to the NSWTP, or if PS 18 is constructed that would reduce the capacity concerns for the PS 7 force main.

C. Interceptors

Table 3.01-11 summarizes the interceptor capacity segments potentially in need of relief before 2060. The table includes flow projections for the expected segments as well as the projected timing for relief under the low flow projections and the high flow projections.

1. Year 2000-2010 Needs

The following interceptor segments were identified as reaching capacity between 2000 and 2010:

Interceptor Segment	Upstream Manhole	Downstream Manhole	Length (ft)	Diameter (in.)	Capacity Deficit in Year 2060 (mgd)
2Bi	02-211	02-008	900	24	4.23
2Bii	02-008	02-005A	1,260	24	3.58
3i	03-111	03-102	3,492	12	0.32
3ii	02-102	PS 3	308	10	0.40
7Fi	07-932	07-313	14	42	37.89
7Fii	07-313	07-215	5,591	48	21.67
7Mii	07-211	PS 7	5,342	60	28.00
8Aiii	02-545	02-538	3,121	27	3.20
8Aiv	02-538	02-536	1,200	24	3.63
8Di	02-531A	02-519	4,363	36	3.08
8Diii	02-518	02-516	204	36	3.08
8Div	02-516	08-228	10	36	4.56
10Bi	10-121	10-118	874	36	11.59
10Bii	10-118	10-201	1,597	42	14.75
10G	10-102A	10-101	959	48	16.67
10H	10-101	PS 10	108	48	17.99
10Ei	10-201	10-115	140	42	18.16

Interceptor Segment	Upstream Manhole	Downstream Manhole	Length (ft)	Diameter (in.)	Capacity Deficit in Year 2060 (mgd)
10Eii	10-115	10-104A	4,412	48	16.20
10Eiii	10-104A	10-102A	1,110	48	16.63
		Total	**35,005**		

These interceptor segments represent 7 percent of the District's 2010 interceptor length. The MMSD in its 2009 Capital Projects budget identified the following project related to the above-listed sewers reaching capacity:

MMSD Project Number	Interceptor Segment	Capital Budget Project Cost	Project Start	Project Finish
839-00-78	10B, 10E, 10G, 10H	$10,200,000	2009	2010
839-00-79	7F	$5,300,000	2012	2014

2. Year 2010-2020 Needs

The following interceptor segments were identified as reaching capacity between the years 2010 and 2020.

Interceptor Segment	Upstream Manhole	Downstream Manhole	Length (feet)	Diameter (inches)	Capacity Deficit in Year 2060 (mgd)
7B	07-437	07-426	5,510	18	3.13
7Ci	07-734	07-728	2,917	21	7.87
7Dii	07-425	07-416	3,861	30	9.82
7Ji	07-249	07-242	2,794	18	4.11
7Mi	07-215	07-211	2,468	60	27.12
8Av	02-536	02-535	600	21	1.71
8Avi	02-535	02-532	841	21	1.71
8Iii	02-038	02-034	1,460	18	1.02
8Ji	02-034	02-032	816	20	1.44
10A	10-145	10-121	10,973	48	7.53
11Aiii	11-169	11-167	465	42	8.79
11Aiv	11-167	11-161E	1,436	42	7.94
11Fii	11-111A	11-106A	2,716	54	11.86
11Fiii	11-106A	11-104	1,689	54	12.48
11Fiv	10-104	PS 11	1,525	54	12.48
12Hi	12-110	12-101	3,484	48	9.57
12Hii	12-101	PS 12	38	48	9.12
13A-Ei	13-122A	13-116H	153	48	5.53
14Mi	14-356	14-345	4,659	24	3.15
		Total	**48,405**		

These interceptor segments represent 10 percent of the District's 2010 interceptor length. The MMSD in their 2009 Capital Projects budget identified the following projects related to the above-listed sewers reaching capacity:

MMSD Project Number	Interceptor Segment(s)	Capital Budget Project Cost	Project Start	Project Finish
832-00-72	8Iii, 8Ji	$600,000	2009	2011

3. Year 2020-2030 Needs

The following interceptor segments were identified as reaching capacity between the years 2020 and 2030.

Interceptor Segment	Upstream Manhole	Downstream Manhole	Length (feet)	Diameter (inches)	Capacity Deficit in Year 2060 (mgd)
7Cii	07-728	07-723	2,496	21	6.82
7Ciii	07-723	07-707	7,899	24	6.25
7Civ	07-707	07-426	3,474	24	5.11
7Jii	07-242	07-231	4,974	24	2.49
7Jiii	07-231	07-228	1,347	24	1.30
7Kii	07-224	07-222	650	30	2.44
8C	02-531I	02-531A	2,653	21	0.33
8Jii	02-032	02-513	1,704	21	1.04
8Xiii	08-121	08-120	16	2@30	2.10
11Aii	11-171	11-169	812	42	7.98
11Avi	11-161A	11-159	1,321	36	5.86
11Bi	11-159	11-158	340	36	6.62
11Biii	11-156	11-151A	2,220	42	4.80
11C	11-151A	11-145	3,784	42	5.50
11Diii	11-137	11-129	3,995	33	7.01
11Dv	11-127	11-116A	4,855	54	7.20
11Fi	11-116A	11-111A	2,788	54	7.91
13Hii	13-105	PS 13	1,758	48	4.89
14B	14-196	14-193	1,203	21	1.80
14Li	14-362	14-358	775	10	0.26
14Mii	14-345	14-338	2,859	21	2.69
14Miv	14-333	14-323	4,889	30	1.99
14N	14-323	14-315	4,055	30	2.74
15Ci	05-113	05-112A	227	24	1.02
16Aii	05-315	05-310	1,002	18	0.90
16Aiv	05-306	05-236	1,771	24	1.05
		Total	**63,867**		

These interceptors segments represent 13 percent of the District's 2010 interceptor length.

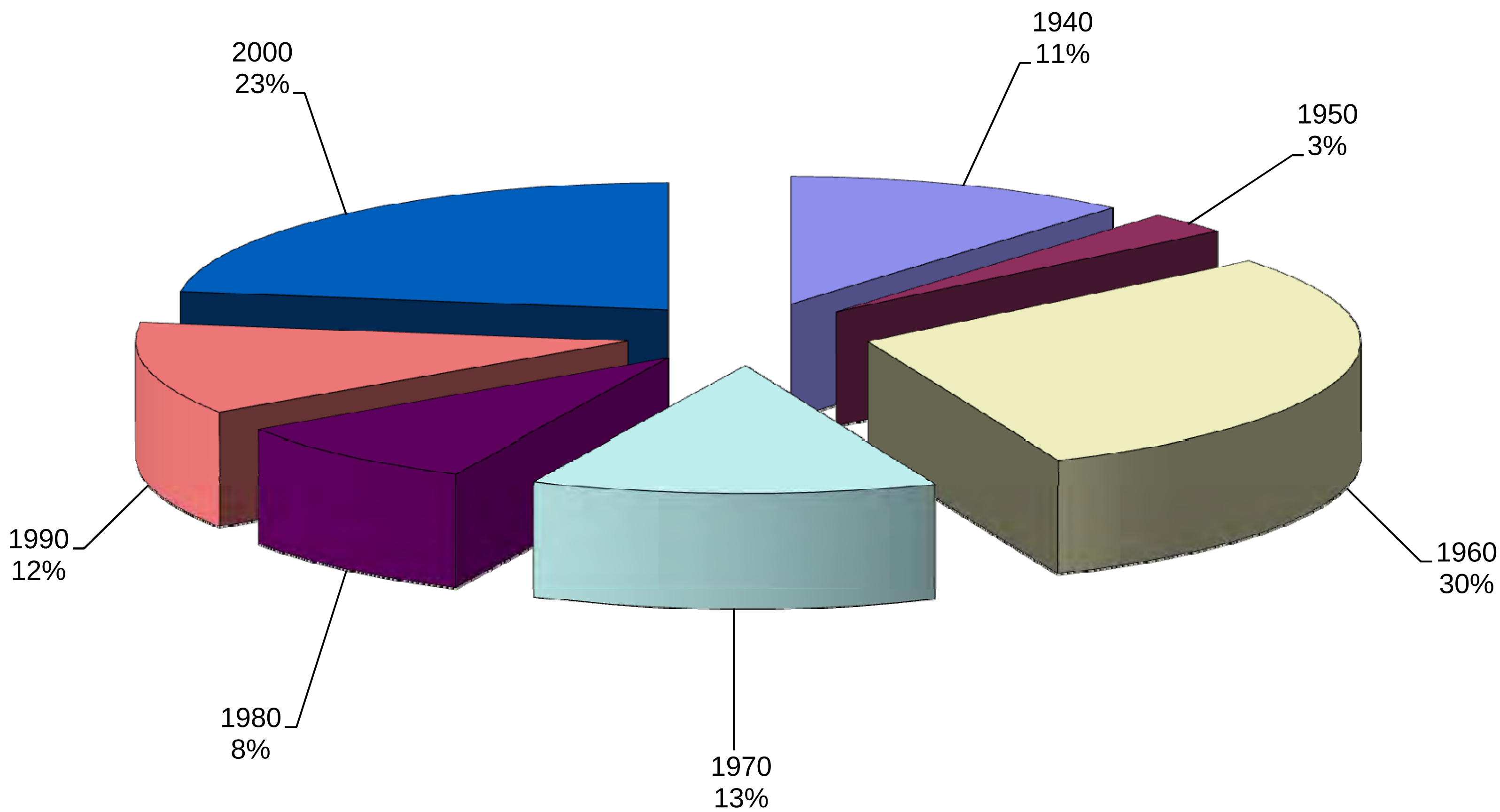
4. Year 2030-2060 Needs

The following interceptor segments were identified as reaching capacity between the years 2030 and 2060.

Interceptor Segment	Upstream Manhole	Downstream Manhole	Length (feet)	Diameter (inches)	Capacity Deficit in Year 2060 (mgd)
7Di	07-426	07-425	153	36	5.12
7Diii	07-416	07-415	355	42	1.39
7E	07-415	07-932	8,067	42	1.68
7Ki	07-228	07-224	2,001	30	2.02
7Kiii	07-222	07-218	1,647	36	2.15
7Kiv	07-218	07-215	1,606	36	1.59
8Ii	02-041	02-038	1,063	18	0.22
8R	02-150	02-145	1,215	24	0.05
8Sii	02-142	02-136	1,669	27	0.23
9A	09-108	09-104	1,678	24	1.12
9Bi	09-104	09-101	1,373	27	0.73
9Bii	09-101	PS 9	285	24	1.77
11Di	11-145	11-141	1,558	36	0.51
11Dii	11-141	11-137	1,648	30	2.57
11Div	11-129	11-127	733	36	3.32
13G	13-132	13-122A	4,397	48	0.49
13Hi	13-105A	13-105	125	46.5	2.74
14D	14-182	14-171	5,724	21	0.29
14E	14-171	14-166	2,351	21	0.41
14Fi	14-166	14-165	488	21	1.76
14Fii	14-165	14-162	1,401	24	0.26
14G	14-162	14-156	2,687	24	0.61
14Jii	14-415	14-411	1,619	15	0.29
14Jiv	14-407	14-134	3,059	18	0.15
14K	14-134	14-102	16,679	36	1.11
14Lii	14-358	14-356	674	24	0.24
14Miii	14-338	14-333	2,110	21	1.01
14Oi	14-315	14-301	5,251	30	1.43
15A	05-116	054-115	2,099	14	0.22
15Ci	05-113	05-106	227	24	1.02
16Aii	05-315	05-310	1,002	18	0.90
		Total	**74,944**		

These interceptors segments represent 15 percent of the District's 2010 interceptor length.

FIGURE 3.01-1
MADISON METROPOLITAN SEWERAGE DISTRICT-FORCE MAIN AGE



MADISON METROPOLITAN SEWERAGE DISTRICT
50-YEAR MASTER PLAN (2010-2060)

TECHNICAL MEMO 3
CONVEYANCE FACILITIES ANALYSIS

TABLE 3.01-1
PUMPING STATION ANALYSIS (YEAR 2000)

Pumping Station		Pumping Station Location	Capacity (mgd) Maximum	Firm	Flows-2000 Average Daily (mgd)	Peak Hourly (mgd)	Peak Hourly to Firm Capacity	Peak Hourly to Maximum Capacity
1		104 North First Street						
		Pumping Station 6	18.0	15.0	6.39	19.07	127%	106%
		Pumping Station 2	20.3	20.3	6.39	19.07	94%	94%
2		833 W. Washington Ave (Brittingham Park)	41.0	41.0	10.38	28.69	70%	70%
3		Nine Springs	1.5	1.5	0.31	1.24	82%	82%
4		522 John Nolen Drive	4.2	4.2	0.97	3.88	92%	92%
5		5221 Lake Mendota Drive (Spring Harbor)	3.6	3.6	0.65	2.60	72%	72%
6		402 Walter Street (Operational Potential)	24.2	24.2	1.55	5.79	24%	24%
7		6300 Metropolitan Lane(Current Operational Mode)	45.0	39.0	13.20	35.12	90%	78%
8		967 Plaenert Drive	34.1	34.0	8.87	25.13	74%	74%
9		4612 Larson Beach Road, McFarland	4.5	4.5	0.81	3.24	72%	72%
10		110 Regas Road	42.2	42.2	8.04	23.14	55%	55%
11		4760 East Clayton Road	31.2	25.5	7.56	21.97	86%	70%
12		2739 Fitchrona Road	23.5	16.6	4.47	14.11	85%	60%
13		3634 Amelia Earhart Dri ve	20.2	20.0	5.58	17.01	85%	84%
14		5000 School Road	15.6	15.0	3.32	10.99	73%	70%
15		2115 Allen Blvd	8.8	5.8	1.43	5.41	93%	61%
16		1301 Gammon Road (Operational Potential)	18.7	18.7	1.51	5.66	30%	30%
16		1301 Gammon Road (Maximum Potential)	18.7	18.7	2.94	9.92	53%	53%
17		407 Bruce Street, Verona	4.6	4.6	0.67	2.68	58%	58%
Notes	1	Peak Hourly to Firm or Maximum Capacity > 100 percent are highlighted, except as noted below.						
	2	The Upstream Pumping Analysis for PS 7 includes the maximum pump capacity for PSs 6, 9, and 10 and the average daily flow for the service area for PS 7 x MDC.						
	3	The Maximum Potential for both PS 6 and PS 7 is based on 100 percent of the flow from PS 1 routed to PS 6 and subsequently to PS 7.						
	4	The firm capacity to peak hourly flow and the maximum capacity to peak hourly flow for PS 1 to PS 6 are not highlighted since capacity is available to PS 2.						
	5	Flows to PS 6 are based on all flow being pumped from PS 1 to PS 6. This is not the normal operating mode.						

MADISON METROPOLITAN SEWERAGE DISTRICT
50-YEAR MASTER PLAN (2010-2060)

TECHNICAL MEMO 3
CONVEYANCE FACILITIES ANALYSIS

TABLE 3.01-2
PUMPING STATION 2010 TAZ

Pumping Station		Pumping Station Location	Capacity (mgd) Maximum	Firm	Flows-2010 TAZ Projection Average Daily (mgd)	Peak Hourly (mgd)	Peak Hourly to Firm Capacity	Peak Hourly to Maximum Capacity
1		104 North First Street						
		Pumping Station 6	18.0	15.0	6.00	18.09	121%	101%
		Pumping Station 2	20.3	20.3	6.00	18.09	89%	89%
2		833 W. Washington Ave (Brittingham Park)	41.0	41.0	10.15	28.16	69%	69%
3		Nine Springs	1.5	1.5	0.31	1.25	83%	83%
4		522 John Nolen Drive	4.2	4.2	0.97	3.90	93%	93%
5		5221 Lake Mendota Drive (Spring Harbor)	3.6	3.6	0.63	2.53	70%	70%
6		402 Walter Street (Operational Potential)	24.2	24.2	1.61	5.97	25%	25%
7		6300 Metropolitan Lane (Current Operational Mode)	45.0	39.0	14.83	38.74	99%	86%
8		967 Plaenert Drive	34.1	34.0	8.89	25.18	74%	74%
9		4612 Larson Beach Road, McFarland	4.5	4.5	0.90	3.58	80%	80%
10		110 Regas Road	42.2	42.2	8.89	25.18	60%	60%
11		4760 East Clayton Road	31.2	25.5	9.04	25.53	100%	82%
12		2739 Fitchrona Road	23.5	16.6	5.66	17.22	104%	73%
13		3634 Amelia Earhart Dri ve	20.2	20.0	6.18	18.54	93%	92%
14		5000 School Road	15.6	15.0	3.76	12.20	81%	78%
15		2115 Allen Blvd	8.8	5.8	1.45	5.48	94%	62%
16		1301 Gammon Road (Operational Potential)	18.7	18.7	1.82	6.63	35%	35%
17		407 Bruce Street, Verona	4.6	4.6	1.18	4.60	100%	100%
Notes	1	Peak Hourly to Firm or Maximum Capacity > 100 percent are highlighted, except as noted below.						
	2	The Upstream Pumping Analysis for PS 7 includes the maximum pump capacity for PSs 6, 9, and 10 and the average daily flow for the service area for PS 7 x MDC.						
	3	The Maximum Potential for both PS 6 and PS 7 is based on 100 percent of the flow from PS 1 routed to PS 6 and subsequently to PS 7.						
	4	The firm capacity to peak hourly flow and the maximum capacity to peak hourly flow for PS 1 to PS 6 are not highlighted since capacity is available to PS 2.						
	5	Flows to PS 6 are based on all flow being pumped from PS 1 to PS 6. This is not the normal operating mode.						

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MADISON METROPOLITAN SEWERAGE DISTRICT
50-YEAR MASTER PLAN (2010-2060)

TECHNICAL MEMO 3
CONVEYANCE FACILITIES ANALYSIS

TABLE 3.01-3
PUMPING STATION 2010 UF

					Flows-2010 UF Projection			
Pumping			**Capacity (mgd)**		**Average Daily**	**Peak Hourly**	**Peak Hourly**	**Peak Hourly**
Station		**Pumping Station Location**	**Maximum**	**Firm**	**(mgd)**	**(mgd)**	**to Firm Capacity**	**to Maximum Capacity**
1		104 North First Street						
		Pumping Station 6	18	15	5.17	15.95	106%	89%
		Pumping Station 2	20.3	20.3	5.17	15.95	79%	79%
2		833 W. Washington Ave (Brittingham Park)	41	41	9.77	27.25	66%	66%
3		Nine Springs	1.51	1.51	0.32	1.29	86%	86%
4		522 John Nolen Drive	4.2	4.2	0.99	3.96	94%	94%
5		5221 Lake Mendota Drive (Spring Harbor)	3.6	3.6	0.62	2.48	69%	69%
6		402 Walter Street	24.2	24.2	1.61	5.97	25%	25%
7		6300 Metropolitan Lane	45	39	16.78	42.99	110%	96%
8		967 Plaenert Drive	34.1	34	8.80	24.97	73%	73%
9		4612 Larson Beach Road, McFarland	4.5	4.5	0.96	3.86	86%	86%
10		110 Regas Road	42.2	42.2	9.78	27.28	65%	65%
11		4760 East Clayton Road	31.2	25.5	10.05	27.92	109%	89%
12		2739 Fitchrona Road	23.5	16.6	6.48	19.29	116%	82%
13		3634 Amelia Earhart Dri ve	20.2	20	6.75	19.98	100%	99%
14		5000 School Road	15.6	15	3.96	12.74	85%	82%
15		2115 Allen Blvd	8.8	5.8	1.56	5.82	100%	66%
16		1301 Gammon Road (Operational Potential)	18.7	18.7	2.02	7.23	39%	39%
16		1301 Gammon Road (Maximum Potential)	18.7	18.7	2.38	8.30	44%	44%
17		407 Bruce Street, Verona	4.6	4.6	0.97	3.90	85%	85%
Notes	1	Peak Hourly to Firm or Maximum Capacity > 100 percent are highlighted, except as noted below.						
	2	The Upstream Pumping Analysis for PS 7 includes the maximum pump capacity for PSs 6, 9, and 10 and the average daily flow for the service area for PS 7 x MDC.						
	3	The Maximum Potential for both PS 6 and PS 7 is based on 100 percent of the flow from PS 1 routed to PS 6 and subsequently to PS 7.						
	4	The firm capacity to peak hourly flow and the maximum capacity to peak hourly flow for PS 1 to PS 6 are not highlighted since capacity is available to PS 2.						
	5	Flows to PS 6 are based on all flow being pumped from PS 1 to PS 6. This is not the normal operating mode.						

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MADISON METROPOLITAN SEWERAGE DISTRICT
50-YEAR MASTER PLAN (2010-2060)

TECHNICAL MEMO 3
CONVEYANCE FACILITIES ANALYSIS

TABLE 3.01-4
PUMPING STATION 2020 TAZ

			Flows-2020 TAZ Projection					
Pumping Station		**Pumping Station Location**	**Capacity (mgd) Maximum**	**Capacity (mgd) Firm**	**Average Daily (mgd)**	**Peak Hourly (mgd)**	**Peak Hourly to Firm Capacity**	**Peak Hourly to Maximum Capacity**
1		104 North First Street						
		Pumping Station 6	18	15	5.61	17.08	114%	95%
		Pumping Station 2	20.3	20.3	5.61	17.08	84%	84%
2		833 W. Washington Ave (Brittingham Park)	41	41	9.92	27.61	67%	67%
3		Nine Springs	1.51	1.51	0.32	1.27	84%	84%
4		522 John Nolen Drive	4.2	4.2	0.98	3.91	93%	93%
5		5221 Lake Mendota Drive (Spring Harbor)	3.6	3.6	0.62	2.47	69%	69%
6		402 Walter Street	24.2	24.2	1.67	6.16	25%	25%
7		6300 Metropolitan Lane	45	39	16.51	42.40	109%	94%
8		967 Plaenert Drive	34.1	34	8.63	24.55	72%	72%
9		4612 Larson Beach Road, McFarland	4.5	4.5	0.98	3.94	87%	87%
10		110 Regas Road	42.2	42.2	9.77	27.26	65%	65%
11		4760 East Clayton Road	31.2	25.5	10.56	29.11	114%	93%
12		2739 Fitchrona Road	23.5	16.6	6.89	20.31	122%	86%
13		3634 Amelia Earhart Drive	20.2	20	6.80	20.09	100%	99%
14		5000 School Road	15.6	15	4.21	13.42	89%	86%
15		2115 Allen Blvd	8.8	5.8	1.48	5.55	96%	63%
16		1301 Gammon Road (Operational Potential)	18.7	18.7	2.15	7.61	41%	41%
17		407 Bruce Street, Verona	4.6	4.6	1.71	6.83	149%	149%
Notes	1	Peak Hourly to Firm or Maximum Capacity > 100 percent are highlighted, except as noted below.						
	2	The Upstream Pumping Analysis for PS 7 includes the maximum pump capacity for PSs 6, 9, and 10 and the average daily flow for the service area for PS 7 x						
	3	The Maximum Potential for both PS 6 and PS 7 is based on 100 percent of the flow from PS 1 routed to PS 6 and subsequently to PS 7.						
	4	The firm capacity to peak hourly flow and the maximum capacity to peak hourly flow for PS 1 to PS 6 are not highlighted since capacity is available to PS 2.						
	5	Flows to PS 6 are based on all flow being pumped from PS 1 to PS 6. This is not the normal operating mode.						

MADISON METROPOLITAN SEWERAGE DISTRICT
50-YEAR MASTER PLAN (2010-2060)

TECHNICAL MEMO 3
CONVEYANCE FACILITIES ANALYSIS

TABLE 3.01-5
PUMPING STATION 2020 UF

			Flows-2020 UF Projection					
Pumping Station		**Pumping Station Location**	**Capacity (mgd) Maximum**	**Firm**	**Average Daily (mgd)**	**Peak Hourly (mgd)**	**Peak Hourly to Firm Capacity**	**Peak Hourly to Maximum Capacity**
1		104 North First Street						
		Pumping Station 6	18	15	5.35	16.43	110%	91%
		Pumping Station 2	20.3	20.3	5.35	16.43	81%	81%
2		833 W. Washington Ave (Brittingham Park)	41	41	10.25	28.40	69%	69%
3		Nine Springs	1.51	1.51	0.34	1.34	89%	89%
4		522 John Nolen Drive	4.2	4.2	1.01	4.03	96%	96%
5		5221 Lake Mendota Drive (Spring Harbor)	3.6	3.6	0.62	2.48	69%	69%
6		402 Walter Street	24.2	24.2	1.67	6.17	25%	25%
7		6300 Metropolitan Lane	45	39	20.36	50.59	130%	112%
8		967 Plaenert Drive	34.1	34	9.05	25.57	75%	75%
9		4612 Larson Beach Road, McFarland	4.5	4.5	1.12	4.41	98%	98%
10		110 Regas Road	42.2	42.2	11.52	31.32	74%	74%
11		4760 East Clayton Road	31.2	25.5	12.54	33.63	132%	108%
12		2739 Fitchrona Road	23.5	16.6	8.48	24.20	146%	103%
13		3634 Amelia Earhart Dri ve	20.2	20	7.95	22.92	115%	113%
14		5000 School Road	15.6	15	4.61	14.48	97%	93%
15		2115 Allen Blvd	8.8	5.8	1.71	6.27	108%	71%
16		1301 Gammon Road (Operational Potential)	18.7	18.7	2.54	8.77	47%	47%
16		1301 Gammon Road (Maximum Potential)	18.7	18.7	2.54	8.77	47%	47%
17		407 Bruce Street, Verona	4.6	4.6	2.50	8.65	188%	188%
Notes	1	Peak Hourly to Firm or Maximum Capacity > 100 percent are highlighted, except as noted below.						
	2	The Upstream Pumping Analysis for PS 7 includes the maximum pump capacity for PSs 6, 9, and 10 and the average daily flow for the service area for PS 7 x						
	3	The Maximum Potential for both PS 6 and PS 7 is based on 100 percent of the flow from PS 1 routed to PS 6 and subsequently to PS 7.						
	4	The firm capacity to peak hourly flow and the maximum capacity to peak hourly flow for PS 1 to PS 6 are not highlighted since capacity is available to PS 2.						
	5	Flows to PS 6 are based on all flow being pumped from PS 1 to PS 6. This is not the normal operating mode.						

MADISON METROPOLITAN SEWERAGE DISTRICT 50-YEAR MASTER PLAN (2010-2060)

TECHNICAL MEMO 3
CONVEYANCE FACILTIES ANALYSIS

TABLE 3.01-6
PUMPING STATION ANALYSIS (2030-TRAFFIC AREA ZONE PROJECTIONS)

					Flows-2030 TAZ Analysis			
Pumping Station		**Pumping Station Location**	**Capacity (mgd) Maximum**	**Firm**	**Average Daily (mgd)**	**Peak Hourly (mgd)**	**Peak Hourly to Firm Capacity**	**Peak Hourly to Maximum Capacity**
1		First Street						
		Pumping Station 6	18	15	5.22	16.08	107%	89%
		Pumping Station 2	20.3	20.3	5.22	16.08	79%	79%
2		Brittingham Park-Operational Potential	41	41	9.69	27.06	66%	66%
3		Nine Spings	1.51	1.51	0.32	1.28	85%	85%
4		John Nolen	4.2	4.2	0.98	3.93	94%	94%
5		Spring Harbor	3.6	3.6	0.60	2.40	67%	67%
6		Walter Street-Operational Potential	27	24.1	1.73	6.36	26%	24%
7		Metropolitan Lane-Operational Potential	45	39	18.14	45.90	118%	102%
8		Plaenert	39	34.1	8.51	24.27	71%	62%
9		McFarland	4.5	4.5	1.07	4.24	94%	94%
10		Regas Road	42.2	42.2	10.62	29.25	69%	69%
11		Clayton Road	31.2	25.5	12.04	32.51	128%	104%
12		Fitchrona Road	23.5	16.6	8.08	23.24	140%	99%
13		Amelia Earhart	20.2	20	7.40	21.56	108%	107%
14		School Road	15.6	15	4.65	14.58	97%	93%
15		Allen Blvd	8.8	5.8	1.50	5.63	97%	64%
16		Gammon-Operational Potential	18.7	18.7	2.46	8.53	46%	46%
17		Verona	4.6	4.6	2.22	7.82	170%	170%
		Peak Pumped Flow to NSWTP	161.91	145.31	49.68	124.20		
Notes	1	Peak Hourly to Firm or Maximum Capacity > 100 percent are highlighted, except as noted below.						
	2	The Upstream Pumping Analysis for PS 7 includes the maximum pump capacity for PSs 6, 9, and 10 and the average daily flow for the service area for PS 7 x MDC.						
	3	The Maximum Potential for both PS 6 and PS 7 is based on 100 percent of the flow from PS 1 routed to PS 6 and subsequently to PS 7.						
	4	The firm capacity to peak hourly flow and the maximum capacity to peak hourly flow for PS 1 to PS 6 are not highlighted since capacity is available to PS 2.						

MADISON METROPOLITAN SEWERAGE DISTRICT
50-YEAR MASTER PLAN (2010-2060)

TECHNICAL MEMO 3
CONVEYANCE FACILITIES ANALYSIS

TABLE 3.01-7
YEAR 2030 UNCERTAINTY FACTOR ANALYSIS

			Flows-2030 Uncertainty Factor Analysis					
Pumping Station		**Pumping Station Location**	**Capacity (mgd) Maximum**	**Firm**	**Average Daily (mgd)**	**Peak Hourly (mgd)**	**Peak Hourly to Firm Capacity**	**Peak Hourly to Maximum Capacity**
1		First Street						
		Pumping Station 6	18	15	5.54	16.91	113%	94%
		Pumping Station 2	20.3	20.3	5.54	16.91	83%	83%
2		Brittingham Park-Operational Potential	41	41	10.74	29.52	72%	72%
2		Brittingham Park-Maximum Potential	41	41	10.74	29.52	72%	72%
3		Nine Spings	1.51	1.51	0.35	1.40	92%	92%
4		John Nolen	4.2	4.2	1.03	4.10	98%	98%
5		Spring Harbor	3.6	3.6	0.63	2.53	70%	70%
6		Walter Street-Operational Potential	27	24.1	1.74	6.38	26%	24%
6		Walter Street-Maximum Potential	27	24.1	7.28	21.28	88%	79%
7		Metropolitan Lane-Maximum Potential	45	39	29.48	69.09	177%	154%
7		Metropolitan Lane-Operational Potential	45	39	23.94	59.85	153%	133%
7		Metroplitan Lane-Upstream Pumping	45	39		95.91	246%	213%
8		Plaenert	39	34.1	9.31	26.18	77%	67%
9		McFarland	4.5	4.5	1.28	4.92	109%	109%
10		Regas Road	42.2	42.2	13.26	35.26	84%	84%
11		Clayton Road	31.2	25.5	15.03	39.18	154%	126%
12		Fitchrona Road	23.5	16.6	10.48	28.92	174%	123%
13		Amelia Earhart	20.2	20	9.14	25.77	129%	128%
14		School Road	15.6	15	5.26	16.19	108%	104%
15		Allen Blvd	8.8	5.8	1.83	6.65	115%	76%
16		Gammon-Operational Potential	18.7	18.7	3.05	10.23	55%	55%
16		Gammon-Maximum Potential	18.7	18.7	4.68	14.67	78%	78%
17		Verona	4.6	4.6	3.41	11.24	244%	244%
		Maximum Pumped Flow to NSWTP	161.91	145.31	60.40	151.00		
Notes	1	Peak Hourly to Firm or Maximum Capacity > 100 percent are highlighted, except as noted below.						
	2	The Upstream Pumping Analysis for PS 7 includes the maximum pump capacity for PSs 6, 9, and 10 and the average daily flow for the service area for PS 7 x						
	3	The Maximum Potential for both PS 6 and PS 7 is based on 100 percent of the flow from PS 1 routed to PS 6 and subsequently to PS 7.						
	4	The firm capacity to peak hourly flow and the maximum capacity to peak hourly flow for PS 1 to PS 6 are not highlighted since capacity is available to PS 2.						

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MADISON METROPOLITAN SEWERAGE DISTRICT
50-YEAR MASTER PLAN (2010-2060)

TECHNICAL MEMO 3
CONVEYANCE FACILITIES ANALYSIS

TABLE 3.01-8
2060 LOW FLOW

					Flows-2060 Low			
Pumping			**Capacity (mgd)**		**Average Daily**	**Peak Hourly**	**Peak Hourly**	**Peak Hourly**
Station		**Pumping Station Location**	**Maximum**	**Firm**	**(mgd)**	**(mgd)**	**to Firm Capacity**	**to Maximum Capacity**
1		First Street						
		Pumping Station 6	18	15	5.54	16.91	113%	94%
		Pumping Station 2	20.3	20.3	5.54	16.91	83%	83%
2		Brittingham Park-Operational Potential	41	41	10.74	29.52	72%	72%
3		Nine Spings	1.51	1.51	0.35	1.40	92%	92%
4		John Nolen	4.2	4.2	1.03	4.10	98%	98%
5		Spring Harbor	3.6	3.6	0.63	2.53	70%	70%
6		Walter Street-Operational Potential	27	24.1	1.74	6.38	26%	24%
7		Metropolitan Lane-Maximum Potential	45	39	29.48	73.70	189%	164%
8		Plaenert	34	34	9.31	26.18	77%	77%
9		McFarland	4.5	4.5	1.28	4.92	109%	109%
10		Regas Road	42.2	42.2	13.26	35.26	84%	84%
11		Clayton Road	31.2	25.5	15.03	39.18	154%	126%
12		Fitchrona Road	23.5	16.6	10.48	28.92	174%	123%
13		Amelia Earhart	20.2	20	9.14	25.77	129%	128%
14		School Road	15.6	15	5.26	16.19	108%	104%
15		Allen Blvd	8.8	5.8	1.83	6.65	115%	76%
16		Gammon-Operational Potential	18.7	18.7	3.05	10.23	55%	55%
17		Verona	4.6	4.6	3.41	11.24	244%	244%
		Peak Pumped Flow to NSWTP	156.91	145.21	60.40	151.00		
Notes	1	Peak Hourly to Firm or Maximum Capacity > 100 percent are highlighted, except as noted below.						
	2	The Upstream Pumping Analysis for PS 7 includes the maximum pump capacity for PSs 6, 9, and 10 and the average daily flow for the service area for PS 7 x MDC.						
	3	The Maximum Potential for both PS 6 and PS 7 is based on 100 percent of the flow from PS 1 routed to PS 6 and subsequently to PS 7.						
	4	The firm capacity to peak hourly flow and the maximum capacity to peak hourly flow for PS 1 to PS 6 are not highlighted since capacity is available to PS 2.						

MADISON METROPOLITAN SEWERAGE DISTRICT
50-YEAR MASTER PLAN (2010-2060)

TECHNICAL MEMO 3
CONVEYANCE FACILITIES ANALYSIS

TABLE 3.01-9
2060 HIGH POPULATION

					Flows-2060 High			
Pumping			**Capacity (mgd)**		**Average Daily**	**Peak Hourly**	**Firm Capacity**	**Maximum Capacity**
Station		**Pumping Station Location**	**Maximum**	**Firm**	**(mgd)**	**(mgd)**	**to Peak Hourly**	**to Peak Hourly**
1		First Street						
		Pumping Station 6	18.0	15.0	6.14	18.45	123%	102%
		Pumping Station 2	20.3	20.3	6.14	18.45	91%	91%
2		Brittingham Park-Operational Potential	41.0	41.0	12.56	33.68	82%	82%
3		Nine Spings	1.5	1.5	0.35	1.40	93%	93%
4		John Nolen	4.2	4.2	1.09	4.30	102%	102%
5		Spring Harbor	3.6	3.6	0.67	2.68	74%	74%
6		Walter Street-Operational Potential	24.2	24.2	1.99	7.14	30%	30%
7		Metropolitan Lane-Operational Potential	45.0	39.0	28.92	72.30	185%	161%
8		Plaenert	34.1	34.0	10.09	28.01	82%	82%
9		McFarland	4.5	4.5	1.75	6.41	142%	142%
10		Regas Road	42.2	42.2	14.83	38.74	92%	92%
11		Clayton Road	31.2	25.5	17.63	44.81	176%	144%
12		Fitchrona Road	23.5	16.6	11.95	32.30	195%	137%
13		Amelia Earhart	20.2	20.0	10.71	29.45	147%	146%
14		School Road	15.6	15.0	6.83	20.17	134%	129%
15		Allen Blvd	8.8	5.8	2.13	7.56	130%	86%
16		Gammon-Operational Potential	18.7	18.7	3.17	10.57	57%	57%
16		Gammon-Maximum Potential	18.7	18.7	5.30	16.29	87%	87%
17		Verona	4.6	4.6	4.27	13.58	295%	295%
Notes	1	Peak Hourly to Firm or Maximum Capacity > 100 percent are highlighted, except as noted below.						
	2	The Upstream Pumping Analysis for PS 7 includes the maximum pump capacity for PSs 6, 9, and 10 and the average daily flow for the service area for PS 7 x MDC.						
	3	The Maximum Potential for both PS 6 and PS 7 is based on 100 percent of the flow from PS 1 routed to PS 6 and subsequently to PS 7.						
	4	The firm capacity to peak hourly flow and the maximum capacity to peak hourly flow for PS 1 to PS 6 are not highlighted since capacity is available to PS 2.						

MADISON METROPOLITAN SEWERAGE DISTRICT
50-YEAR MASTER PLAN (2010-2060)

TECHNICAL MEMO 3
CONVEYANCE FACILITIES ANALYSIS

TABLE 3.01-10
FORCE MAIN ANALYSIS EXISTING

Pumping Station Force Main	Length (feet)	Diameter	Material	Year Installed	Nominal Force Main Capacity 8 fps Velocity (mgd)	Pressure (mgd)	Required Capacity 2030 Low (mgd)	Required Capacity 2030 High (mgd)	Required Capacity 2060 Low (mgd)	Required Capacity 2060 High (mgd)	Excess Available Capacity 2060 (mgd)	Additional Required Capacity 2060 Based on 8 fps	Addition Required High	Addition Required Low
1	2,638	30	RCCP	1948	25.4		16.08	16.90	16.90	18.45	6.92			
CTFM	14,213	30	DIP	2002	25.4		16.08	16.90	16.90	18.45	6.92			
	998	20	PVC	1995	11.3		16.08	16.90	16.90	18.45		7.17		
	1,346	24	DIP	2000	16.2		16.08	16.90	16.90	18.45		2.21		
2	9,890	36	DIP	2001	36.5		27.06	29.53	29.53	33.69	2.85			
2 and 4	6,395	36	DIP	2001	36.5		29.36	31.88	31.88	36.12	0.42			
	364	36	DIP	2005	36.5		29.36	31.88	31.88	36.12	0.42			
2, 3, and 4	1,123	36	DIP	2001	36.5		30.10	32.68	32.68	36.90		0.36	2057	
3	5	8	CIP	1959	1.8		1.29	1.40	1.40	1.40	0.40			
	21	8	DIP	2000	1.8		1.29	1.40	1.40	1.40	0.40			
4	100	16	CIP	1959	7.2		3.93	4.09	4.09	4.29	2.93			
	53	16	DIP	2000	7.2		3.93	4.09	4.09	4.29	2.93			
5	28	16	DIP	1996	7.2		2.40	2.52	2.52	2.68	4.54			
	457	16	RCCP	1959	7.2		2.40	2.52	2.52	2.68	4.54			
5 and 15	1,742	24	RCCP	1959	16.2		7.47	8.54	8.54	9.52	6.72			
6	7,214	36	RCCP	1948	36.5		6.36	6.37	6.37	7.14	29.40			
7	6,996	36	RCCP	1948	36.5	55	45.90	59.86	59.86	72.30		17.30	2024	2050
	6,996	36	RCCP	1963	36.5	55	45.90	59.86	59.86	72.30		17.30	2024	2050
	1,332	48	RCCP	1963	65.0		45.90	59.86	59.86	72.30		7.35	2042	
	323	48	DIP	2005	65.0		45.90	59.86	59.86	72.30		7.35	2042	
8	13,174	42	RCCP	1964	49.7		24.27	26.17	26.17	28.02	21.71			
	194	36	RCCP	1964	36.5		24.27	26.17	26.17	28.02	8.52			
	334	42	DIP	2005	49.7		24.27	26.17	26.17	28.02	21.71			
9	4,329	20	DIP	1987	11.3		4.24	4.93	4.93	6.39	4.89			
	40	14	DIP	1987	5.5		4.24	4.93	4.93	6.39		0.86	2048	
	2,197	10			2.8		4.24	4.93	4.93	6.39		3.57		
10	11,109	36	RCCP	1964	36.5		29.25	35.26	35.26	38.74		2.20	2040	
11	4,173	36	RCCP	1965	36.5		32.51	39.17	39.17	44.82		8.28	2025	2048
	91	36	DIP	2005	36.5		32.51	39.17	39.17	44.82		8.28	2025	2050
12	4,786	36	RCCP	1968	36.5		23.24	28.93	28.93	32.3	4.24			
13	1,927	36	RCCP	1969	36.5		21.56	25.77	25.77	29.44	7.10			
14	3,108	30	RCCP	1971	25.4		14.58	16.18	16.18	20.16	5.21			
	1,358	30	RCCP	1971	25.4		15.30	16.90	16.90	20.84	4.53			
15-8	1,360	24	DIP	1974	16.2		5.63	6.65	6.65	7.57	8.67			
	1,071	24	DIP	1974	16.2		5.63	6.65	6.65	7.57	8.67			
	4,837	20	RCCP	1959	11.3		5.63	6.65	6.65	7.57	3.71			
	18	24	RCCP	1959	16.2		5.63	6.65	6.65	7.57				
16	7,214	36	DIP	1979	36.5		8.53	10.24	10.24	10.55	25.99			
	2,965	30	DIP	1980	25.4		8.53	10.24	10.24	10.55	14.82			
17	13,357	16	DIP	1995	7.2		7.82	11.25	11.25	13.57		6.35	2016	2026
	3,071	20	DIP	1995	11.3		7.82	11.25	11.25	13.57		2.29	2031	2060
Total (feet)	142,947													
Total (Miles)	27.1													

Notes: MMSD has 50-million gallons of treated storage at the NSWTP that is used if the effluent flows exceed 78.6 mgd. If flows extend for a period of time, discharge to Nine Springs Creek will occur via an overflow structure.
Estimates for velocities in Force Mains 2, 3, and 4 are based on all pumping stations pumping at firm capacity at the same time.
Material abbreviations are based on the Collection System Facilities Plan Update-2008.

TABLE 3.01-11
INTERCEPTOR CAPACITY ANALYSIS

Pumping Station/Pump Basin	From MH	to MH	Length	Existing Sewer	Segment Description	Capacity	Flows					Deficit	High	Low	Master Plan
			(feet)	(in)		(mgd)	2000	2030 TAZ	2030 UF	2060 Low	2060 High				Project #
2Xii	02-300	02-101	3		WI-Spring Street Relief	6.54	7.2	7.92	8.86	8.86	11.01	4.47	2000	2000	2-1
2Bi	02-011	02-008	900	24	OWI from MH 02-012	4.62	5.65	6.59	7.69	7.69	8.85	4.23	2000	2000	2-21
2Bii	02-008	02-005A	1260	24	OWI from MH 02-012	5.27	5.65	6.59	7.69	7.69	8.85	3.58	2000	2000	2-22
3i	03-111	03-102	3,492	12	Rimrock Interceptor	1.08	1.24	1.29	1.4	1.4	1.4	0.32	2000	2000	3-11
3ii	03-102	PS 3	308	10	Rimrock Interceptor	1	1.24	1.29	1.4	1.4	1.4	0.4	2000	2000	3-12
7Ai	07-955	07-954	95	48	NEI-PS 10 to SEI	40.45	25.09	31.3	37.44	37.44	40.86	0.41	2056		7-1
7Ci	07-734	07-728	2,917	21	FEI-Door Creek Extension	4.36	0.18	2.62	7.14	7.14	12.23	7.87	2018	2041	7-31
7Cii	07-728	07-723	2,496	21	FEI-Door Creek Extension	5.41	0.18	2.62	7.14	7.14	12.23	6.82	2022	2048	7-32
7Ciii	07-723	07-707	7,899	24	FEI-Door Creek Extension	5.98	0.18	2.62	7.14	7.14	12.23	6.25	2025	2052	7-33
7Civ	07-707	07-426	3,474	24	FEI-Door Creek Extension	7.12	0.18	2.62	7.14	7.14	12.23	5.11	2030	2060	7-34
7Di	07-426	07-425	153	36	FEI-Upstream of Interstate	12.19	1.68	6.41	11.11	11.11	17.31	5.12	2035		7-41
7Dii	07-425	07-416	3,861	30	FEI-Upstream of Interstate	7.49	1.68	6.41	11.11	11.11	17.31	9.82	2018	2037	7-42
7Diii	07-416	07-415	355	42	FEI-Upstream of Interstate	15.92	1.68	6.41	11.11	11.11	17.31	1.39	2053		7-43
7E	07-415	07-932	8,067	42	FEI-Downstream of Interstate	15.92	1.96	6.71	11.44	11.44	17.6	1.68	2052		7.5
7Fi	07-932	07-313	14	42	NEI-Downstream of FEI	15.92	26.75	35.94	45.5	45.5	53.68	37.76	2000	2000	7-61
7Fii	07-313	07-215	5,591	48	NEI-Downstream of FEI	32.14	26.75	35.94	45.5	45.5	53.68	21.54	2009	2018	7-62
7Ji	07-249	07-242	2,794	18	SEI-Blooming Grove Extension	2.25	0.37	1.21	5.21	5.21	6.36	4.11	2012	2038	7-71
7Jii	07-242	07-231	4,974	24	SEI-Blooming Grove Extension	3.87	0.37	1.21	5.21	5.21	6.36	2.49	2022	2050	7-72
7Jiii	07-231	07-228	1,347	24	SEI-Blooming Grove Extension	5.06	0.37	1.21	5.21	5.21	6.36	1.3	2029	2059	7-73
7Ki	07-228	07-224	2,001	30	SEI-McFarland Relief-Downstream of Blooming Grove Ext.	10.26	3.84	6.18	9.98	9.98	12.28	2.02	2034		7-81
7Kii	07-224	07-222	650	30	SEI-McFarland Relief-Downstream of Blooming Grove Ext.	10.26	4.21	6.54	10.42	10.42	12.7	2.44	2058		7-82
7Kiii	07-222	07-218	1,647	36	SEI-McFarland Relief-Downstream of Blooming Grove Ext.	10.55	4.21	6.54	10.42	10.42	12.7	2.15	2032		7-83
7Kiv	07-218	07-215	1,606	36	SEI-McFarland Relief	11.4	4.51	6.82	10.71	10.71	12.99	1.59	2039		7-84
7Mi	07-215	07-211	2,468	60	SEI-Downstream of NEI	37.62	29.44	40.1	52.28	52.28	64.74	27.12	2011	2023	7-91
7Mii	07-211	PS 7	5,342	60	SEI-Downstream of NEI	37.62	30.09	40.74	53.01	53.01	65.62	28	2010	2021	7-92
8Aiii	02-545	02-538	3,121	27	WI Relief to MH 02-519	8.95	9.79	10.20	11.21	11.21	12.15	3.20	2000	2000	8-11
8Aiv	02-538	02-536	1,200	24	WI Relief to MH 02-519	8.52	9.79	10.20	11.21	11.21	12.15	3.63	2000	2000	8-12
8Av	02-536	02-535	600	21	WI Relief to MH 02-519	10.44	9.79	10.20	11.21	11.21	12.15	1.71	2014	2037	8-13
8Avi	02-535	02-532	841	21	WI Relief to MH 02-519	10.44	9.79	10.20	11.21	11.21	12.15	1.71	2014	2037	8-14
8C	02-531I	02-531A	2,653	21	WI-Midvale Relief	3.55	3.19	3.16	3.57	3.57	3.88	0.33	2028	2059	8-2
8Di	02-531A	02-519	4,363	36	WI Relief to 02-519 after Midvale Relief	12.19	12.58	12.93	14.17	14.17	15.27	3.08	2000	2000	8-31
8Diii	02-518	02-516	204	10	WI Relief from MH 02-519	12.19	12.58	12.93	14.17	14.17	15.27	3.08	2000	2000	8-32
8Div	02-516	08-228	10	36	WI Relief from MH 02-519	12.19	14.21	14.45	15.67	15.67	16.75	4.56	2000	2000	8-33
8Xiii	08-121	08-120	16	2@30	Wi Randall Relief to SWI	21.13	19.93	19.93	21.58	21.58	23.23	2.10	2022	2052	8-10
8Ii	02-041	02-038	1,063	18	OWI to MH 02-308	2.71	1.40	1.49	2.20	2.20	2.93	0.22	2051		8-61
8Iii	02-038	02-034	1,460	18	OWI from MH 02-308	1.92	1.40	1.49	2.20	2.20	2.93	1.01	2020	2048	8-62
8Ji	02-034	02-032	816	20	OWI from MH 02-038	2.84	2.41	2.49	3.47	3.47	4.28	1.44	2012	2041	8-71
8Jii	02-032	02-513	1,704	21	OWI from MH 02-038	3.24	2.41	2.49	3.47	3.47	4.28	1.04	2023	2053	8-72
8R	02-150	02-145	1,215	24	SWI	5.84	5.32	5.26	5.55	5.55	5.89	0.05	2056		8-8
8Sii	02-142	02-136	1,669	27	SWI	5.66	5.32	5.26	5.55	5.55	5.89	0.23	2040		8-9
9A	09-108	09-104	1,678	24	SEI Upstream of MH 09-104	4.13	2.05	2.92	3.67	3.67	5.25	1.12	2039		9-1
9Bi	09-104	09-101	1,373	27	SEI Upstream of MH 09-101	5.66	3.22	4.24	4.93	4.93	6.39	0.73	2045		9-21
9Bii	09-101	PS 9	285	24	SEI Upstream of MH 09-101	4.62	3.22	4.24	4.93	4.93	6.39	1.77	2025	2047	9-22

TABLE 3.01-11
INTERCEPTOR CAPACITY ANALYSIS

Pumping Station/Pump Basin	From MH	to MH	Length	Existing Sewer	Segment Description	Capacity	Flows					Deficit	High	Low	Master Plan
			(feet)	(in)		(mgd)	2000	2030 TAZ	2030 UF	2060 Low	2060 High				Project #
Pumping Station/Pump Basin	From MH	to MH	Length	Existing Sewer	Segment Description	Capacity	Flows					Deficit	High	Low	Master Plan
			(feet)	(in)		(mgd)	2000	2030 TAZ	2030 UF	2060 Low	2060 High				Project #
10A	10-145	10-121	10,973	48	NEI PS 13 to PS 10	24.55	19.09	24.11	28.47	28.47	32.08	7.53	2017	2033	10-1
10Bi	10-121	10-118	874	36	NEI PS 13 to PS 10	21.54	20.06	25.10	29.54	29.54	33.13	11.59	2005	2009	10-21
10Bii	10-118	10-201	1,597	42	NEI PS 13 to PS 10	18.38	20.06	25.10	29.54	29.54	33.13	14.75	2000	2000	10-22
10Ei	10-201	10-115	140	42	NEI PS 13 to PS 10-Downstream of Lien Extension	18.38	20.85	27.04	33.02	33.02	36.54	18.16	2000	2000	10-31
10Eii	10-115	10-104A	4412	48	NEI PS 13 to PS 10-Downstream of Lien Extension	20.75	21.26	27.44	33.44	33.44	36.95	16.2	2000	2000	10-32
10Eiii	10-104A	10-102A	1110	48	NEI PS 13 to PS 10-Downstream of Lien Extension	20.75	21.72	27.87	33.87	33.87	37.38	16.63	2000	2000	10-33
10G	10-102A	10-101	959	48	NEI-Downstream of NEI and Hwy 30 Extension	20.75	21.74	27.90	33.91	33.91	37.42	16.67	2000	2000	10-4
10H	10-101	PS10	108	48	NEI-Downstream of NEI and Hwy 30 Extension	20.75	23.13	29.25	35.26	35.26	38.74	17.99	2000	2000	10-5
11Aii	11-171	11-169	812	42	NSVI PS 12 to MH 11-127	24.32	14.12	23.24	28.93	28.93	32.30	7.98	2022	2036	11-11
11Aiii	11-169	11-167	465	42	NSVI PS 12 to MH 11-127	24.32	14.99	24.04	29.76	29.76	33.11	8.79	2020	2031	11-12
11Aiv	11-167	11-161E	1,436	42	NSVI PS 12 to MH 11-127	25.17	14.99	24.04	29.76	29.76	33.11	7.94	2020	2036	11-13
11Avi	11-161A	11-159	1,321	36	NSVI PS 12 to MH 11-127	27.25	14.99	24.04	29.76	29.76	33.11	5.86	2025	2047	11-14
11Bi	11-159	11-158	340	36	NSVI PS 12 to MH 11-127	27.25	15.91	24.84	30.53	30.53	33.87	6.62	2023	2043	11-21
11Biii	11-156	11-151A	2,220	42	NSVI PS 12 to MH 11-127	29.07	15.91	24.84	30.53	30.53	33.87	4.80	2028	2052	11-22
11C	11-151A	11-145	3,784	42	NSVI PS 12 to MH 11-127	29.07	16.23	25.39	31.09	31.09	34.57	5.5	2026	2049	11-3
11Di	11-145	11-141	1,558	36	NSVI PS 12 to MH 11-127	37.81	19.82	29.18	34.91	34.91	38.32	0.51	2056		11-41
11Dii	11-141	11-137	1,648	30	NSVI PS 12 to MH 11-127	35.75	19.82	29.18	34.91	34.91	38.32	2.57	2037		11-42
11Diii	11-137	11-129	3,995	33	NSVI PS 12 to MH 11-127	31.31	19.82	29.18	34.91	34.91	38.32	7.01	2023	2041	11-43
11Div	11-129	11-127	733	36	NSVI PS 12 to MH 11-127	35	19.82	29.18	34.91	34.91	38.32	3.32	2031	2060	11-44
11Dv	11-127	11-116A	4,855	54	NSVI to PS 11	31.12	19.82	29.18	34.91	34.91	38.32	7.2	2022	2040	11-45
11Fi	11-116A	11-111A	2,788	54	NSVI to PS 11 after Syene Extension	31.12	20.53	29.85	35.63	35.63	39.03	7.91	2021	2037	11-51
11Fii	11-111A	11-106A	2,716	54	NSVI to PS 11 after Syene Extension	31.12	20.58	31.19	37.39	37.5	42.98	11.86	2019	2030	11-52
11Fiii	11-106A	11-104	1689	54	NSVI to PS 11 after HWY 14 Extension	31.12	21.29	31.84	38.03	38.03	43.6	12.48	2018	2028	11-53
11Fiv	11-104	PS11	1525	54	NSVI to PS 11 after HWY 14 Extension	31.12	21.70	32.22	38.9	38.9	44.56	13.44	2016	2027	11-54
12Hi	12-110	12-101	3,484	48	NSVI to PS 12	22.73	13.97	23.24	28.93	28.93	32.3	9.57	2018	2029	12-11
12Hii	12-101	PS 12	38	48	NSVI to PS 12	22.73	14.2	23.34	29.04	29.04	31.85	9.12	2017	2028	12-12
13G	13-132	13-122A	4,397	48	NEI PS 14 to PS 13	20.75	12.01	15.71	17.31	17.31	21.24	0.49	2056		13-1
13A-Ei	13-122A	13-116H	153	48	NEI PS 14 to PS 13	20.75	16.94	20.57	22.52	22.52	26.28	5.53	2020	2033	13-2
13Hi	13-105A	13-105	125	46.5	NEI PS 14 to PS 13	26.7	17	21.56	25.77	25.77	29.44	2.74	2038		13-3
13Hii	13-105	PS 13	1,758	48	NEI PS 14 to PS 13	24.55	17	21.56	25.77	25.77	29.44	4.89	2026	2051	13-4
14B	14-196	14-193	1,203	21	NEI-DeForest Extension	3.39	2.69	3.16	3.61	3.61	5.19	1.8	2023	2045	14-1
14D	14-182	14-171	5,724	21	NEI-DeForest Extension	5.51	2.97	3.87	4.32	4.32	5.80	0.29	2054		14-2
14E	14-171	14-166	2,351	21	NEI-DeForest Extension	5.51	3.13	4.02	4.45	4.45	5.92	0.41	2052		14-3
14Fi	14-166	14-165	488	21	NEI-DeForest Extension	5.51	3.76	4.9	5.35	5.35	7.27	1.76	2033		14-41
14Fii	14-165	14-162	1,401	24	NEI-DeForest Extension	7.01	3.76	4.9	5.35	5.35	7.27	0.26	2056		14-42
14G	14-162	14-156	2,687	24	NEI-DeForest Extension	7.01	3.81	5.23	5.72	5.72	7.62	0.61	2050		14-5
14Jii	14-415	14-411	1,619	15	NEI-Hwy 19 Extension	2.21	0.81	1.42	2.08	2.08	2.5	0.29	2039		14-61
14Jv	14-407	14-134	3,059	18	NEI-Hwy 19 Extension	2.35	0.81	1.42	2.08	2.08	2.5	0.15	2049		14-62
14K	14-134	14-102	16,679	36	NEI:DeForest Extension after HWY 10 Extension	9.63	5.57	7.45	8.58	8.58	10.74	1.11	2045		14-7
14Li	14-362	14-358	775	10	NEI-Waunakee Extension	1.54	1.34	1.52	1.58	1.58	1.8	0.26	2025	2040	14-81
14Lii	14-358	14-356	674	24	NEI-Waunakee Extension	5.47	3.45	4.42	4.69	4.69	5.71	0.24	2053		14-82
14Mi	14-356	14-345	4,659	24	NEI-Waunakee Extension	5.85	4.45	6.42	7.03	7.03	9	3.15	2016	2021	14-91
14Mii	14-345	14-338	2,859	21	NEI-Waunakee Extension	6.31	4.45	6.42	7.03	7.03	9	2.69	2022	2028	14-92
14Miii	14-338	14-333	2,110	21	NEI-Waunakee Extension	7.99	4.45	6.44	7.03	7.03	9	1.01	2045		14-93
14Miv	14-333	14-323	4,889	30	NEI-Waunakee Extension	7.01	4.45	6.42	7.03	7.03	9	1.99	2030	2059	14-94
14N	14-323	14-315	4,055	30	NEI-Waunakee Extension	7.01	4.86	7.02	7.65	7.65	9.75	2.74	2023	2030	14-10
14Oi	14-315	14-301	5,251	30	NEI-Waunakee Extension	9.18	5.46	7.63	8.28	8.28	10.61	1.43	2042		14-11
15A	05-116	05-115	2,099	14	WI-West Extension (Siphon)	3.43	1.5	1.97	3.3	3.3	3.65	3.65	2041	2056	15-3
15Ci	05-113	05-112A	227	24	WI Extension from MH 05-109	5.85	4.74	4.88	5.93	5.93	6.87	1.02	2028	2058	15-1
16Aii	05-315	05-310	1,002	18	WI	6.18	2.85	5.18	6.74	6.74	7.08	0.9	2026	2049	16-11
16Aiv	05-306	05-236	1,771	24	WI	6.03	2.85	5.18	6.74	6.74	7.08	1.05	2025	2046	16-12

3.02 IMPACTS OF SATELLITE PLANTS ON EXISTING SYSTEM NEEDS

A. Mendota Plant

A Mendota WWTP would treat wastewater near the Yahara River where it enters Lake Mendota. The effluent from this facility would serve to provide base flow for the Yahara River and Lakes system and possibly for Token Creek or the Yahara River upstream of Lake Mendota. Figure 3.02-1 shows the average daily flows for the pumping stations if all the flows to PS 14 are diverted to the Mendota Plant. Refer to Figure 2.02-1 for a comparison of flows if a Mendota plant were not constructed. Figure 3.02-2 shows the peak hourly flows for the pumping stations if all the flows to PS 14 are diverted to the Mendota Plant. Refer to Figure 2.02-2 for a comparison if a Mendota plant were not constructed.

Pumping stations potentially impacted by the construction of the Mendota plant include PSs 13, 10, and 7. PS 10 had no identified capacity needs. However, the ability to site and operate a WWTP on the north side of Lake Mendota may take longer than the time available to MMSD, based on current system needs downstream of PS 14. The Mendota plant may provide capacity relief for the NSWTP, however, or provide a more local source of water for streamflow augmentation, infiltration for groundwater recharge, or industrial reuse. See TM 7–Development of Alternatives for more discussion related to the Mendota plant which is noted in that technical memo as Project E1.

B. Sugar River Plant

A Sugar River WWTP would treat wastewater generated in the Sugar River basin and would provide capacity beyond the 3.6 mgd currently being discharged to the Badger Mill Creek portion of the Sugar River watershed. This would allow water pumped from the Sugar River watershed to remain in the watershed. A significant portion of the planned growth in the Central Urban Service Area (CUSA) and all the growth in the Verona Urban Service Area will occur in this watershed. Figure 3.02-3 shows the average daily flows for all pumping stations if all the flows to PS 17 are diverted to the Sugar River plant. In this figure the average daily flows assumed for the Badger Mill Creek discharge are 3.6 mgd. Refer to Figure 2.02-1 for a comparison of flows if a Sugar River plant were not constructed. Figure 3.02-4 shows the peak hourly flows if all the flows to PS 17 are diverted to the Sugar River plant. Peak hourly flows for the Badger Mill Creek effluent return are also assumed to be 3.6 mgd. Refer to Figure 2.02-2 for a comparison of flows if a Sugar River plant were not constructed.

Pumping stations potentially impacted by the construction of the Sugar River plant include PSs 11, 12, and 17.

Construction of a Sugar River plant with a 4.27 mgd average daily design flow (ADF), which is the flow contributed by the PS 17 service area in 2060, would result in a peak hourly flow using the Madison Design Curve (MDC) of 13.57 mgd. This would potentially eliminate the need for relief for all interceptor segments of the NSVI associated with PS 11. Construction of a Sugar River plant would also delay the need for firm capacity additions for PS 11 and PS 12.

C. Sun Prairie Plant

This option would route flows to the Sun Prairie WWTP which is located in the Koshkonong Creek watershed. A portion of the development in the CUSA will be occurring in this watershed. However, a significant portion of the Sun Prairie USA is located in the Token Creek watershed of the Yahara River watershed. Figures 3.02-5 and 3.02-6 show the average daily flows and peak hourly flows, respectively, should a portion of the flows be routed to the Sun Prairie WWTP. These figures are based on transferring the hydrologic balance of flows from the east side of the CUSA to the Sun Prairie WWTP. The hydrologic balance is the difference between the amount of wastewater generated in the Koshkonong watershed and the amount of the Sun Prairie USA wastewater generated in the Token Creek watershed transferred to the Koshkonong watershed.

A portion of the growth area for PS 7 between 2030 and 2060 is located in the Koshkonong Creek watershed. This growth area contains part of subbasin 7C.. If an ADF of 2.25 mgd can be rerouted from this area to the Sun Prairie WWTP, the need for relief for sewer segments 7Ci, 7Cii, 7Ciii, and 7Civ would be eliminated. This option is discussed in more detail in TM 7–Development of Alternatives as a part of Project E6 and the Alternative MP3.

D. Stoughton Plant

This option would route wastewater generated in the Door Creek watershed to the Stoughton WWTP or a new facility constructed to allow discharges to the Yahara River south of the dam in Stoughton. Figure 3.02-7 shows the average daily flows for the MMSD system if the flows generated in the Door Creek watershed tributary to PS 7 and tributary to PS 9 were diverted to the Stoughton plant. Figure 3.02-8 shows the peak hourly flows assuming that the peak hourly flows routed to Stoughton would be based on the average daily flows multiplied by the Madison Design Curve peaking factor.

A significant portion of the growth area for PS 7 as well as the existing and future growth areas for PS 9 lie in the Door Creek portion of the Yahara River and Lake Kegonsa watershed as well as the Lake Waubesa portion of the Yahara River and Lake Monona watershed. If the flows from this area were routed through a Door Creek Interceptor, Pumping Station, and Force Main to the Stoughton WWTP the potential need for relief of interceptor segments associated with 7J, 7K, 7B, 7E, and all segments of 7D except 7Dii would be eliminated. This is discussed in more detail in TM 7-Development of Alternatives.

E. Oregon Plant

The Village of Oregon's WWTP discharges to Badfish Creek. A potential option for this facility would be to construct a pumping station at the treatment plant and pump wastewater to the NSWTP through the NSVI. Figures 3.02-9 and 3.02-10 show the average daily flows and peak hourly flows if Oregon's wastewater were treated at the NSWTP. Treated effluent would be returned to Badfish Creek. At the time of the preparation of this plan, the Village of Oregon desired to retain its existing facilities and not discharge to or become a part of the MMSD. A portion of the MMSD service area (Oak Hill Correctional Facility) is currently served by the Village of Oregon.

Based on the development plans for Fitchburg, a significant expansion of the Oregon WWTP to serve current MMSD service areas is unlikely. However, a potential exists for the Village of Oregon to join the District. Flow from the Village of Oregon would be directed ultimately to the PS 11 service area to be pumped to the NSWTP.

A potential reserve in PS 11 for an ADF contribution of approximately 1.7 to 2 mgd should be considered when the firm capacity upgrades for PS 11 are reviewed. This would preserve a future option to connect the Village of Oregon with the MMSD if that option is desired by the village and meets appropriate regulatory criteria.

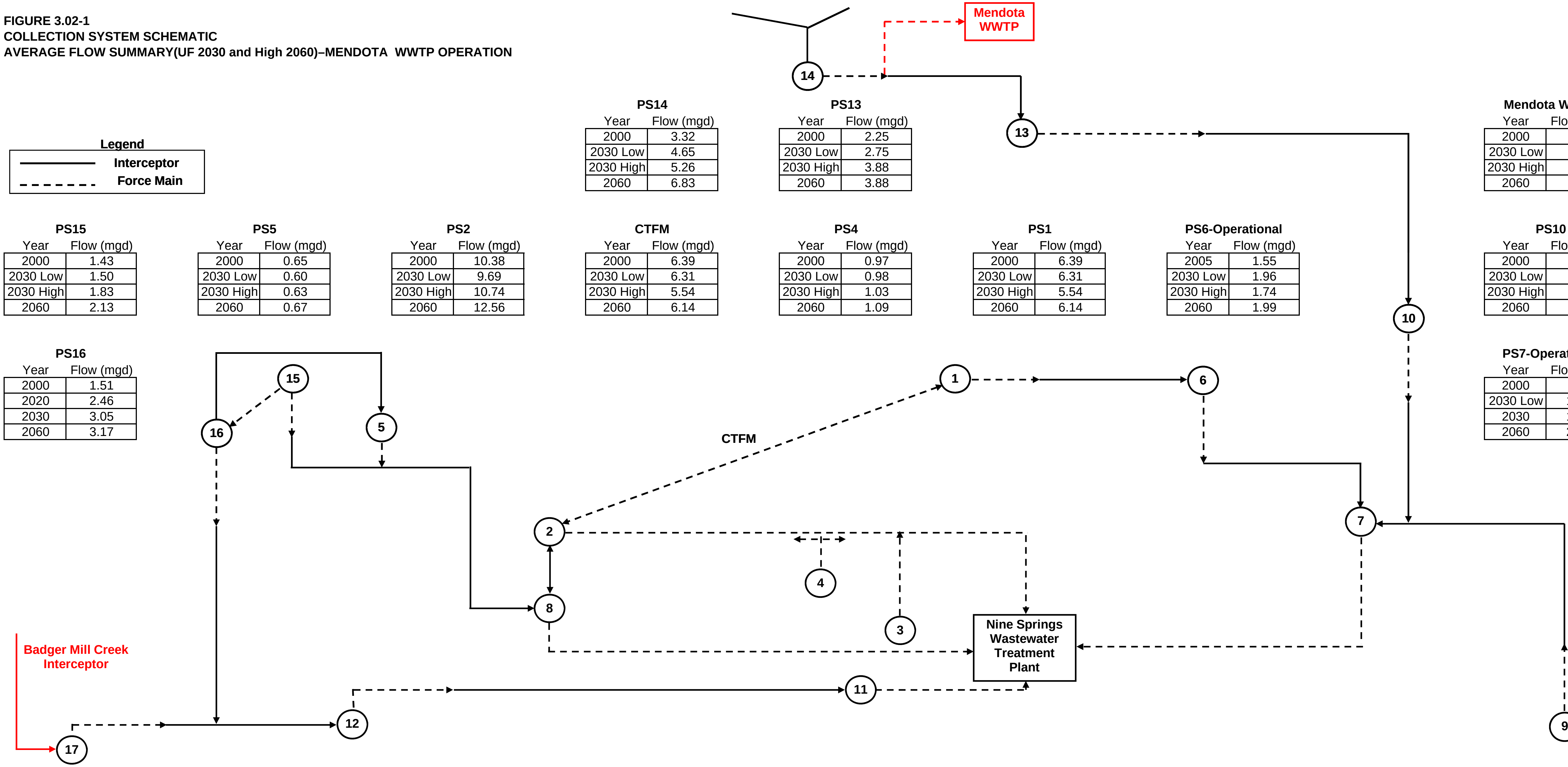
F. Effluent Reuse

Potential options to be considered in this 50-Year Master Plan include reuse options that could require construction of additional pumping facilities at either the NSWTP or other facilities to distribute the treated effluent to the use point.

MADISON METROPOLITAN SEWERAGE DISTRICT
50-YEAR MASTER PLAN (2010-2060)

TECHNICAL MEMO 3
CONVEYANCE FACILITIES ANALYSIS

FIGURE 3.02-1
COLLECTION SYSTEM SCHEMATIC
AVERAGE FLOW SUMMARY(UF 2030 and High 2060)–MENDOTA WWTP OPERATION



PS14

Year	Flow (mgd)
2000	3.32
2030 Low	4.65
2030 High	5.26
2060	6.83

PS13

Year	Flow (mgd)
2000	2.25
2030 Low	2.75
2030 High	3.88
2060	3.88

Mendota WWTP

Year	Flow (mgd)
2000	0.00
2030 Low	4.65
2030 High	5.26
2060	6.83

PS15

Year	Flow (mgd)
2000	1.43
2030 Low	1.50
2030 High	1.83
2060	2.13

PS5

Year	Flow (mgd)
2000	0.65
2030 Low	0.60
2030 High	0.63
2060	0.67

PS2

Year	Flow (mgd)
2000	10.38
2030 Low	9.69
2030 High	10.74
2060	12.56

CTFM

Year	Flow (mgd)
2000	6.39
2030 Low	6.31
2030 High	5.54
2060	6.14

PS4

Year	Flow (mgd)
2000	0.97
2030 Low	0.98
2030 High	1.03
2060	1.09

PS1

Year	Flow (mgd)
2000	6.39
2030 Low	6.31
2030 High	5.54
2060	6.14

PS6-Operational

Year	Flow (mgd)
2005	1.55
2030 Low	1.96
2030 High	1.74
2060	1.99

PS10

Year	Flow (mgd)
2000	8.04
2030 Low	5.97
2030 High	8.00
2060	8.00

PS16

Year	Flow (mgd)
2000	1.51
2020	2.46
2030	3.05
2060	3.17

PS7-Operational

Year	Flow (mgd)
2000	13.2
2030 Low	13.49
2030	18.68
2060	22.09

PS17

Year	Flow (mgd)
2000	0.67
2030 Low	2.22
2030 High	3.41
2060	4.27

PS12

Year	Flow (mgd)
2000	4.47
2030 Low	8.08
2030 High	10.48
2060	11.95

PS8

Year	Flow (mgd)
2000	8.87
2030 Low	8.51
2030 High	9.31
2060	10.09

PS11

Year	Flow (mgd)
2000	7.56
2030 Low	12.04
2030 High	15.03
2060	17.63

PS3

Year	Flow (mgd)
2000	0.31
2030 Low	0.32
2030 High	0.35
2060	0.35

NSWTP

Year	Flow (mgd)
2000	41.29
2030 Low	45.03
2030 High	55.14
2060	53.57

PS9

Year	Flow (mgd)
2000	0.81
2030 Low	1.07
2030 High	1.28
2060	1.75

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S:\MAD\1500--1599\1547\001\Spr\Technical Memos\CFA Spreadsheets\\CFA Tables Sections 1, 2 and 3 With District Comments (042009).xlsx\Figure 3.02-1 Average Flows-Men\8/26/2009

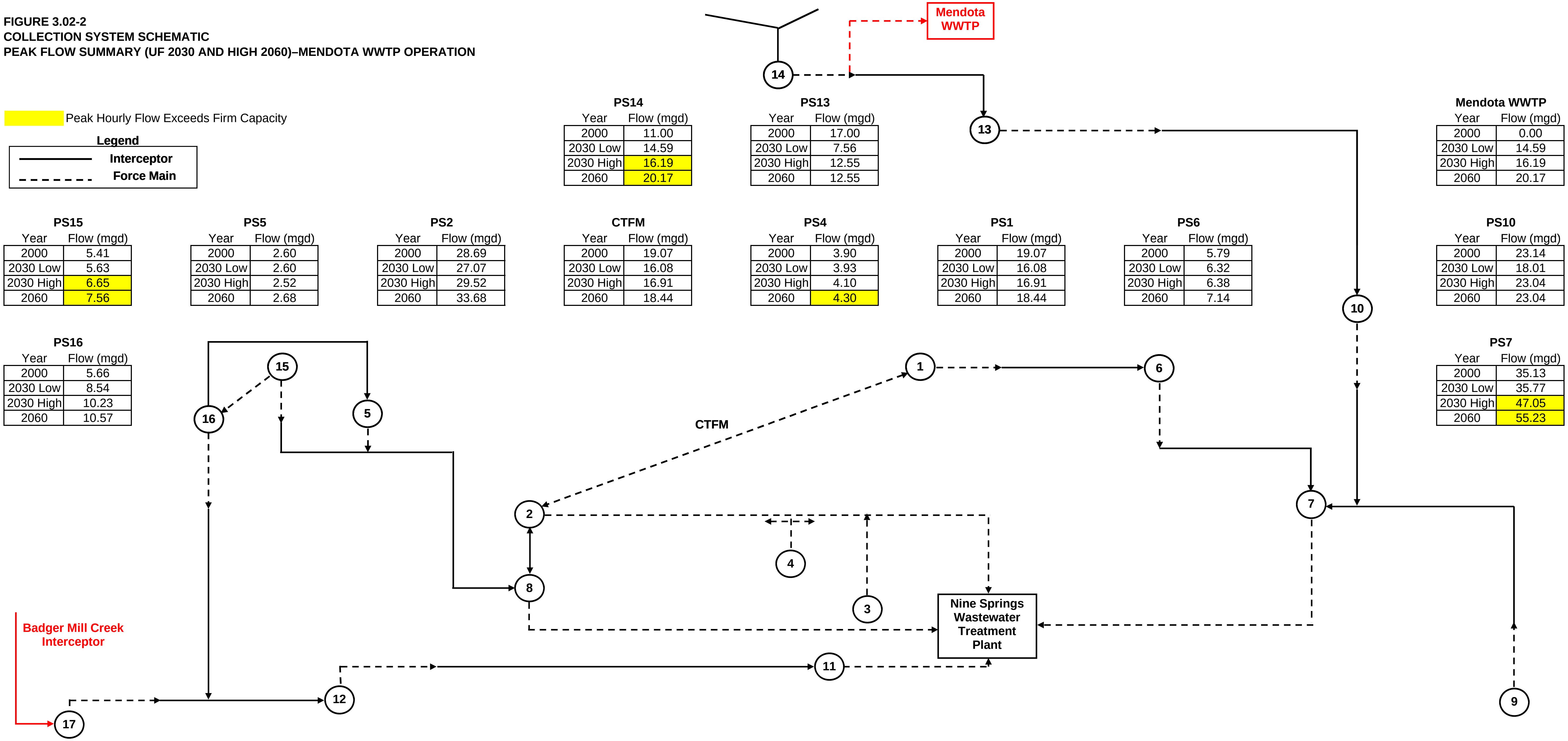
MADISON METROPOLITAN SEWERAGE DISTRICT
50-YEAR MASTER PLAN (2010-2060)

TECHNICAL MEMO 3
CONVEYANCE FACILITIES ANALYSIS

FIGURE 3.02-2
COLLECTION SYSTEM SCHEMATIC
PEAK FLOW SUMMARY (UF 2030 AND HIGH 2060)–MENDOTA WWTP OPERATION

Peak Hourly Flow Exceeds Firm Capacity

Legend
Interceptor
Force Main



PS14

Year	Flow (mgd)
2000	11.00
2030 Low	14.59
2030 High	16.19
2060	20.17

PS13

Year	Flow (mgd)
2000	17.00
2030 Low	7.56
2030 High	12.55
2060	12.55

Mendota WWTP

Year	Flow (mgd)
2000	0.00
2030 Low	14.59
2030 High	16.19
2060	20.17

PS15

Year	Flow (mgd)
2000	5.41
2030 Low	5.63
2030 High	6.65
2060	7.56

PS5

Year	Flow (mgd)
2000	2.60
2030 Low	2.60
2030 High	2.52
2060	2.68

PS2

Year	Flow (mgd)
2000	28.69
2030 Low	27.07
2030 High	29.52
2060	33.68

CTFM

Year	Flow (mgd)
2000	19.07
2030 Low	16.08
2030 High	16.91
2060	18.44

PS4

Year	Flow (mgd)
2000	3.90
2030 Low	3.93
2030 High	4.10
2060	4.30

PS1

Year	Flow (mgd)
2000	19.07
2030 Low	16.08
2030 High	16.91
2060	18.44

PS6

Year	Flow (mgd)
2000	5.79
2030 Low	6.32
2030 High	6.38
2060	7.14

PS10

Year	Flow (mgd)
2000	23.14
2030 Low	18.01
2030 High	23.04
2060	23.04

PS16

Year	Flow (mgd)
2000	5.66
2030 Low	8.54
2030 High	10.23
2060	10.57

PS7

Year	Flow (mgd)
2000	35.13
2030 Low	35.77
2030 High	47.05
2060	55.23

PS17

Year	Flow (mgd)
2000	2.68
2030 Low	7.83
2030 High	11.24
2060	13.58

PS12

Year	Flow (mgd)
2000	14.11
2030 Low	23.23
2030 High	28.92
2060	32.30

PS8

Year	Flow (mgd)
2000	25.13
2030 Low	24.27
2030 High	26.18
2060	28.01

PS11

Year	Flow (mgd)
2000	21.97
2030 Low	32.50
2030 High	39.18
2060	44.81

PS3

Year	Flow (mgd)
2000	1.24
2030 Low	1.28
2030 High	1.40
2060	1.40

NSWTP

Year	Flow (mgd)
2000	103.23
2030 Low	112.58
2030 High	137.85
2060	133.93

PS9

Year	Flow (mgd)
2000	3.24
2030 Low	4.23
2030 High	4.92
2060	6.41

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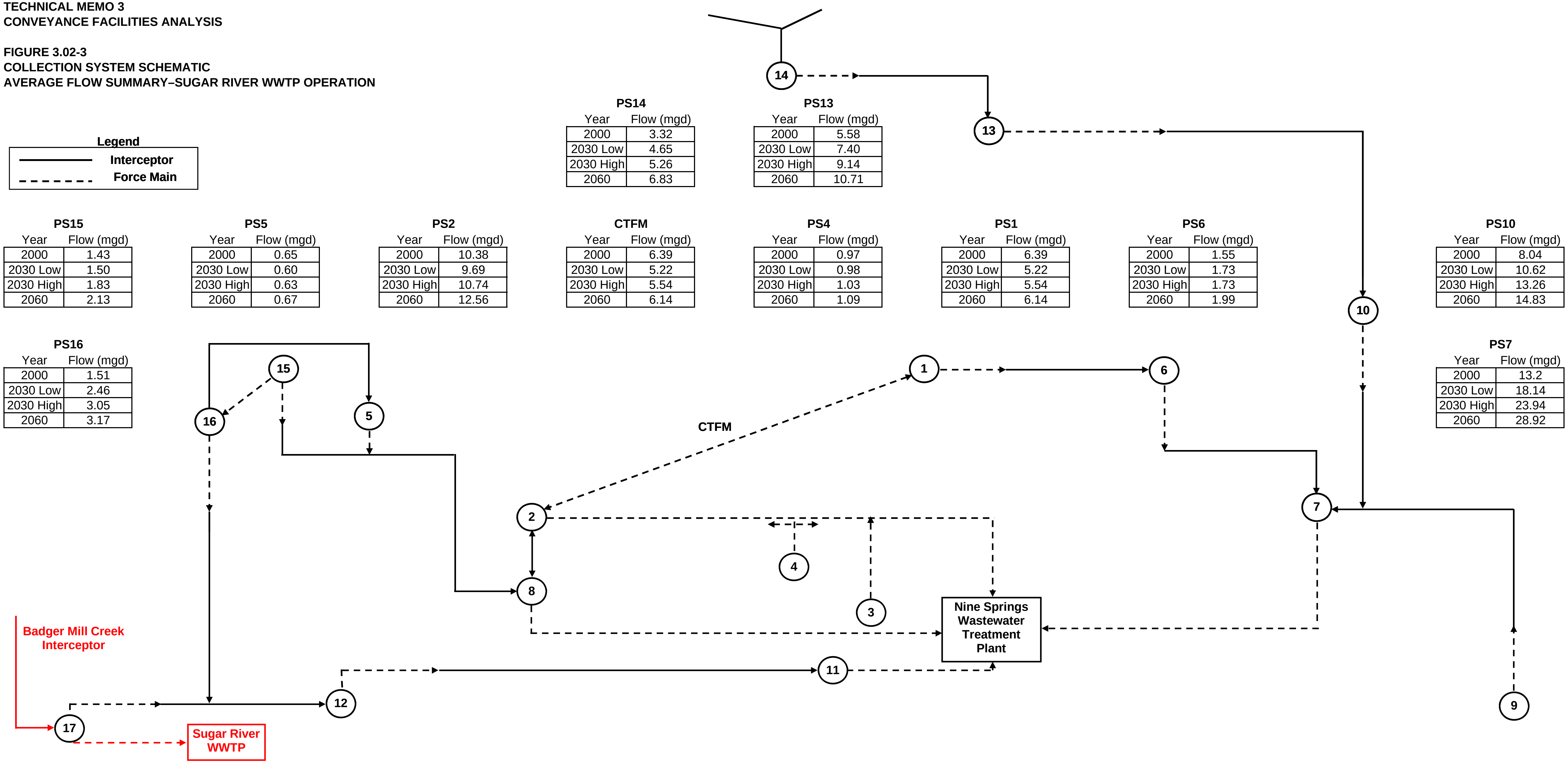
S:\MAD\1500--1599\1547\001\Spr\Technical Memos\CFA Spreadsheets\\CFA Tables Sections 1, 2 and 3 With District Comments (042009).xlsx\Figure 3.02-2 Peak Flows-Men8/26/2009

MADISON METROPOLITAN SEWERAGE DISTRICT
50-YEAR MASTER PLAN (2010-2060)

TECHNICAL MEMO 3
CONVEYANCE FACILITIES ANALYSIS

FIGURE 3.02-3
COLLECTION SYSTEM SCHEMATIC
AVERAGE FLOW SUMMARY–SUGAR RIVER WWTP OPERATION



PS14

Year	Flow (mgd)
2000	3.32
2030 Low	4.65
2030 High	5.26
2060	6.83

PS13

Year	Flow (mgd)
2000	5.58
2030 Low	7.40
2030 High	9.14
2060	10.71

PS15

Year	Flow (mgd)
2000	1.43
2030 Low	1.50
2030 High	1.83
2060	2.13

PS5

Year	Flow (mgd)
2000	0.65
2030 Low	0.60
2030 High	0.63
2060	0.67

PS2

Year	Flow (mgd)
2000	10.38
2030 Low	9.69
2030 High	10.74
2060	12.56

CTFM

Year	Flow (mgd)
2000	6.39
2030 Low	5.22
2030 High	5.54
2060	6.14

PS4

Year	Flow (mgd)
2000	0.97
2030 Low	0.98
2030 High	1.03
2060	1.09

PS1

Year	Flow (mgd)
2000	6.39
2030 Low	5.22
2030 High	5.54
2060	6.14

PS6

Year	Flow (mgd)
2000	1.55
2030 Low	1.73
2030 High	1.73
2060	1.99

PS10

Year	Flow (mgd)
2000	8.04
2030 Low	10.62
2030 High	13.26
2060	14.83

PS16

Year	Flow (mgd)
2000	1.51
2030 Low	2.46
2030 High	3.05
2060	3.17

PS7

Year	Flow (mgd)
2000	13.2
2030 Low	18.14
2030 High	23.94
2060	28.92

PS17

Year	Flow (mgd)
2000	0.67
2030 Low	2.22
2030 High	3.41
2060	4.27

Sugar River WWTP

Year	Flow (mgd)
2000	0.00
2030 Low	2.22
2030 High	3.41
2060	4.27

PS12

Year	Flow (mgd)
2000	4.47
2030 Low	5.86
2030 High	7.07
2060	7.68

PS8

Year	Flow (mgd)
2000	8.87
2030 Low	8.51
2030 High	9.31
2060	10.09

PS11

Year	Flow (mgd)
2000	7.56
2030 Low	9.82
2030 High	11.62
2060	13.36

PS3

Year	Flow (mgd)
2000	0.31
2030 Low	0.32
2030 High	0.35
2060	0.35

NSWTP

Year	Flow (mgd)
2000	41.29
2030 Low	47.46
2030 High	56.99
2060	56.13

PS9

Year	Flow (mgd)
2000	0.81
2030 Low	1.07
2030 High	1.28
2060	1.75

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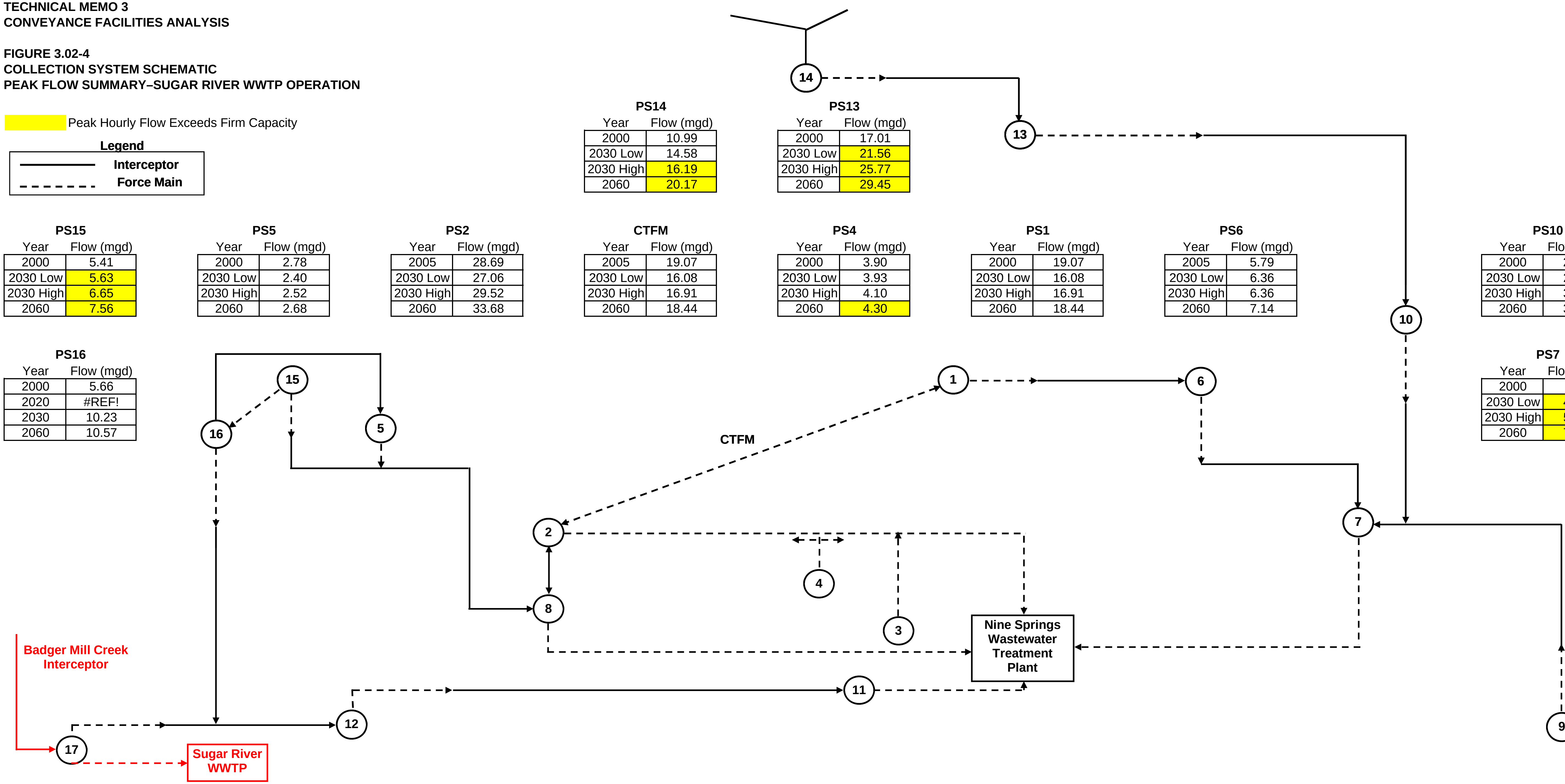
S:\MAD\1500--1599\1547\001\Spr\Technical Memos\CFA Spreadsheets\\CFA Tables Sections 1, 2 and 3 With District Comments (042009).xlsx\Figure 3.02-3 Average Flows SR\8/26/2009

MADISON METROPOLITAN SEWERAGE DISTRICT
50-YEAR MASTER PLAN (2010-2060)

TECHNICAL MEMO 3
CONVEYANCE FACILITIES ANALYSIS

FIGURE 3.02-4
COLLECTION SYSTEM SCHEMATIC
PEAK FLOW SUMMARY–SUGAR RIVER WWTP OPERATION

Peak Hourly Flow Exceeds Firm Capacity



PS14

Year	Flow (mgd)
2000	10.99
2030 Low	14.58
2030 High	16.19
2060	20.17

PS13

Year	Flow (mgd)
2000	17.01
2030 Low	21.56
2030 High	25.77
2060	29.45

PS15

Year	Flow (mgd)
2000	5.41
2030 Low	5.63
2030 High	6.65
2060	7.56

PS5

Year	Flow (mgd)
2000	2.78
2030 Low	2.40
2030 High	2.52
2060	2.68

PS2

Year	Flow (mgd)
2005	28.69
2030 Low	27.06
2030 High	29.52
2060	33.68

CTFM

Year	Flow (mgd)
2005	19.07
2030 Low	16.08
2030 High	16.91
2060	18.44

PS4

Year	Flow (mgd)
2000	3.90
2030 Low	3.93
2030 High	4.10
2060	4.30

PS1

Year	Flow (mgd)
2000	19.07
2030 Low	16.08
2030 High	16.91
2060	18.44

PS6

Year	Flow (mgd)
2005	5.79
2030 Low	6.36
2030 High	6.36
2060	7.14

PS10

Year	Flow (mgd)
2000	23.14
2030 Low	29.25
2030 High	35.26
2060	38.74

PS16

Year	Flow (mgd)
2000	5.66
2020	#REF!
2030	10.23
2060	10.57

PS7

Year	Flow (mgd)
2000	35.1
2030 Low	45.90
2030 High	59.85
2060	72.30

PS17

Year	Flow (mgd)
2000	2.68
2030 Low	7.82
2030 High	11.24
2060	13.58

Sugar River WWTP

Year	Flow (mgd)
2000	0.00
2030 Low	7.82
2030 High	11.24
2060	13.58

PS12

Year	Flow (mgd)
2000	14.11
2030 Low	17.72
2030 High	20.76
2060	22.26

PS8

Year	Flow (mgd)
2000	25.13
2030 Low	24.27
2030 High	26.18
2060	28.01

PS11

Year	Flow (mgd)
2000	21.97
2030 Low	27.38
2030 High	31.55
2060	35.48

PS3

Year	Flow (mgd)
2000	1.24
2030 Low	1.29
2030 High	1.40
2060	1.40

NSWTP

Year	Flow (mgd)
2000	103.23
2030 Low	118.65
2030 High	142.48
2060	140.33

PS9

Year	Flow (mgd)
2000	3.35
2030 Low	4.24
2030 High	4.92
2060	6.41

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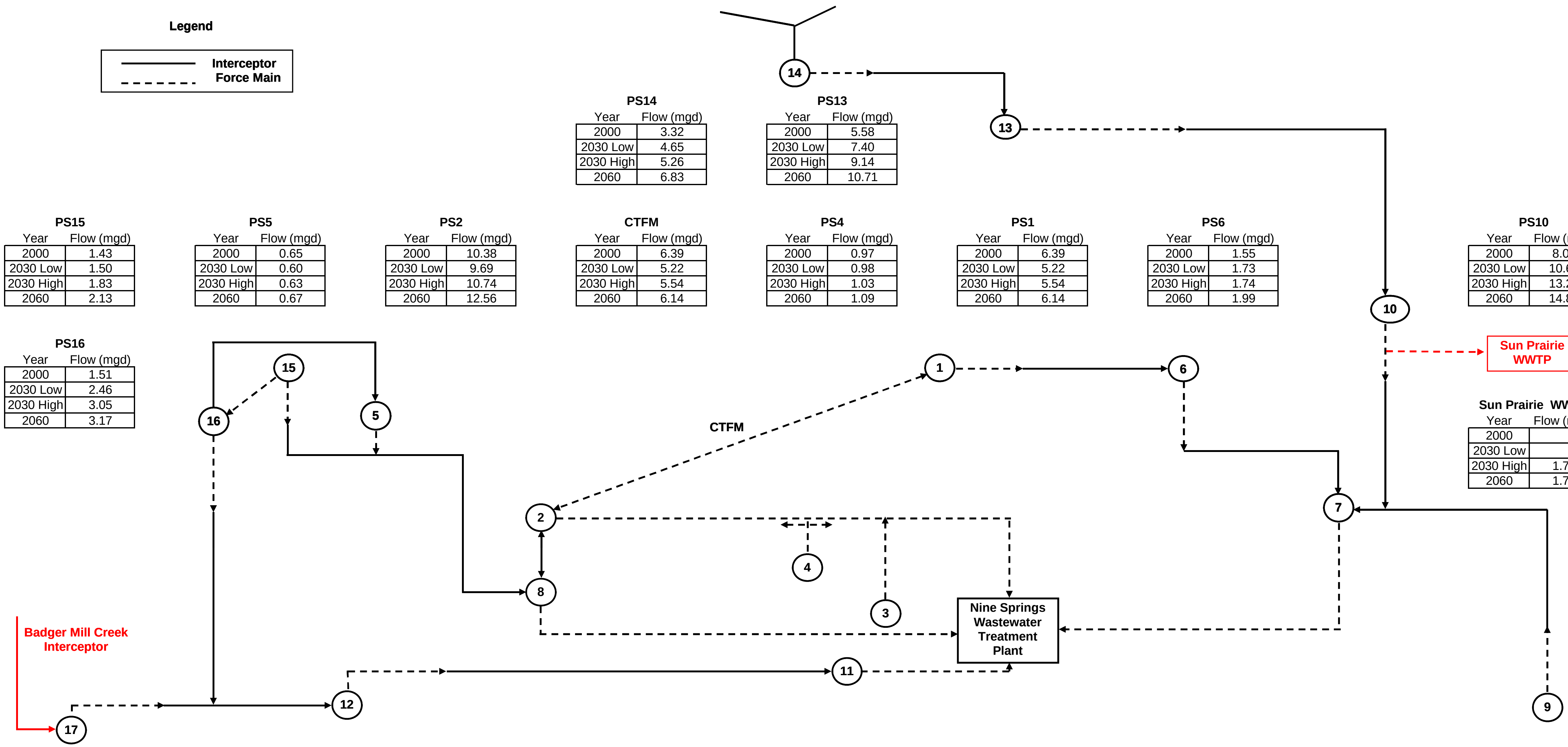
S:\MAD\1500--1599\1547\001\Spr\Technical Memos\CFA Spreadsheets\\CFA Tables Sections 1, 2 and 3 With District Comments (042009).xlsx\Figure 3.02-4 Peak Flows SR\8/26/2009

MADISON METROPOLITAN SEWERAGE DISTRICT
50-YEAR MASTER PLAN (2010-2060)

TECHNICAL MEMO 3
CONVEYANCE FACILITIES ANALYSIS

FIGURE 3.02-5
COLLECTION SYSTEM SCHEMATIC
AVERAGE FLOW SUMMARY–SPWTP ADF



PS14

Year	Flow (mgd)
2000	3.32
2030 Low	4.65
2030 High	5.26
2060	6.83

PS13

Year	Flow (mgd)
2000	5.58
2030 Low	7.40
2030 High	9.14
2060	10.71

PS15

Year	Flow (mgd)
2000	1.43
2030 Low	1.50
2030 High	1.83
2060	2.13

PS5

Year	Flow (mgd)
2000	0.65
2030 Low	0.60
2030 High	0.63
2060	0.67

PS2

Year	Flow (mgd)
2000	10.38
2030 Low	9.69
2030 High	10.74
2060	12.56

CTFM

Year	Flow (mgd)
2000	6.39
2030 Low	5.22
2030 High	5.54
2060	6.14

PS4

Year	Flow (mgd)
2000	0.97
2030 Low	0.98
2030 High	1.03
2060	1.09

PS1

Year	Flow (mgd)
2000	6.39
2030 Low	5.22
2030 High	5.54
2060	6.14

PS6

Year	Flow (mgd)
2000	1.55
2030 Low	1.73
2030 High	1.74
2060	1.99

PS10

Year	Flow (mgd)
2000	8.04
2030 Low	10.62
2030 High	13.26
2060	14.83

PS16

Year	Flow (mgd)
2000	1.51
2030 Low	2.46
2030 High	3.05
2060	3.17

Sun Prairie WWTP

Year	Flow (mgd)
2000	
2030 Low	
2030 High	1.78
2060	1.78

PS17

Year	Flow (mgd)
2000	0.67
2030 Low	2.22
2030 High	3.41
2060	4.27

PS12

Year	Flow (mgd)
2000	4.47
2030 Low	8.08
2030 High	10.48
2060	11.95

PS8

Year	Flow (mgd)
2000	8.87
2030 Low	8.51
2030 High	9.31
2060	10.09

PS11

Year	Flow (mgd)
2000	7.56
2030 Low	12.04
2030 High	15.03
2060	17.63

PS3

Year	Flow (mgd)
2000	0.31
2030 Low	0.32
2030 High	0.35
2060	0.35

NSWTP

Year	Flow (mgd)
2000	41.29
2030 Low	49.68
2030 High	58.62
2060	58.62

PS7

Year	Flow (mgd)
2000	13.2
2030 Low	18.14
2030 High	22.16
2060	27.14

PS9

Year	Flow (mgd)
2000	0.81
2030 Low	1.07
2030 High	1.28
2060	1.75

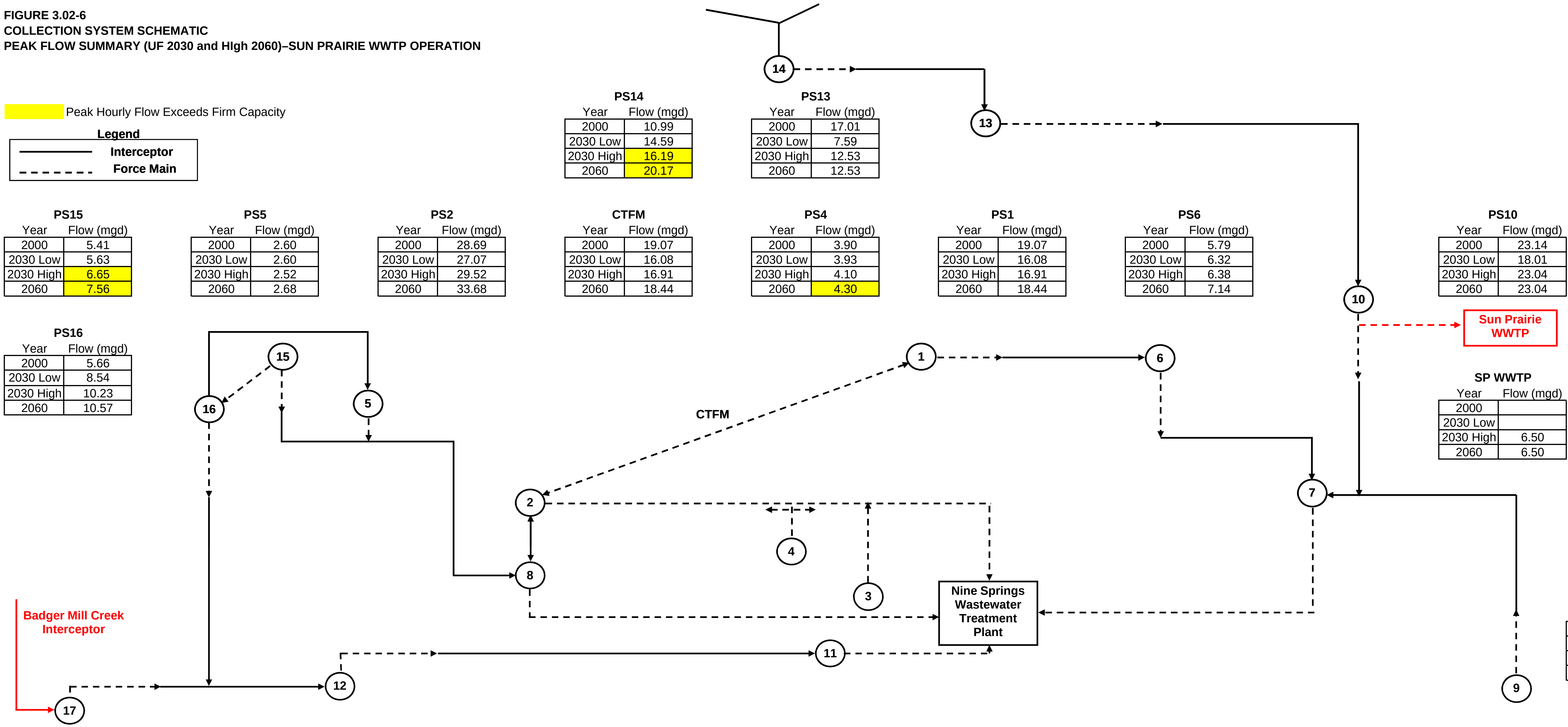
MADISON METROPOLITAN SEWERAGE DISTRICT
50-YEAR MASTER PLAN (2010-2060)

TECHNICAL MEMO 3
CONVEYANCE FACILITIES ANALYSIS

FIGURE 3.02-6
COLLECTION SYSTEM SCHEMATIC
PEAK FLOW SUMMARY (UF 2030 and HIgh 2060)–SUN PRAIRIE WWTP OPERATION

Peak Hourly Flow Exceeds Firm Capacity

Legend
- **Interceptor** (solid line)
- **Force Main** (dashed line)



PS14

Year	Flow (mgd)
2000	10.99
2030 Low	14.59
2030 High	16.19
2060	20.17

PS13

Year	Flow (mgd)
2000	17.01
2030 Low	7.59
2030 High	12.53
2060	12.53

PS15

Year	Flow (mgd)
2000	5.41
2030 Low	5.63
2030 High	6.65
2060	7.56

PS5

Year	Flow (mgd)
2000	2.60
2030 Low	2.60
2030 High	2.52
2060	2.68

PS2

Year	Flow (mgd)
2000	28.69
2030 Low	27.07
2030 High	29.52
2060	33.68

CTFM

Year	Flow (mgd)
2000	19.07
2030 Low	16.08
2030 High	16.91
2060	18.44

PS4

Year	Flow (mgd)
2000	3.90
2030 Low	3.93
2030 High	4.10
2060	4.30

PS1

Year	Flow (mgd)
2000	19.07
2030 Low	16.08
2030 High	16.91
2060	18.44

PS6

Year	Flow (mgd)
2000	5.79
2030 Low	6.32
2030 High	6.38
2060	7.14

PS10

Year	Flow (mgd)
2000	23.14
2030 Low	18.01
2030 High	23.04
2060	23.04

PS16

Year	Flow (mgd)
2000	5.66
2030 Low	8.54
2030 High	10.23
2060	10.57

SP WWTP

Year	Flow (mgd)
2000	
2030 Low	
2030 High	6.50
2060	6.50

PS7

Year	Flow (mgd)
2000	35.1
2030 Low	35.77
2030 High	55.40
2060	67.85

PS17

Year	Flow (mgd)
2000	2.68
2030 Low	7.83
2030 High	11.24
2060	13.58

PS12

Year	Flow (mgd)
2000	14.11
2030 Low	23.23
2030 High	28.92
2060	32.30

PS8

Year	Flow (mgd)
2000	25.13
2030 Low	24.27
2030 High	26.18
2060	28.01

PS11

Year	Flow (mgd)
2000	21.97
2030 Low	32.50
2030 High	39.18
2060	44.81

PS3

Year	Flow (mgd)
2000	1.24
2030 Low	1.28
2030 High	1.40
2060	1.40

NSWTP

Year	Flow (mgd)
2000	103.23
2030 Low	131.48
2030 High	124.20
2060	146.55

PS9

Year	Flow (mgd)
2000	3.24
2030 Low	4.23
2030 High	4.92
2060	6.41

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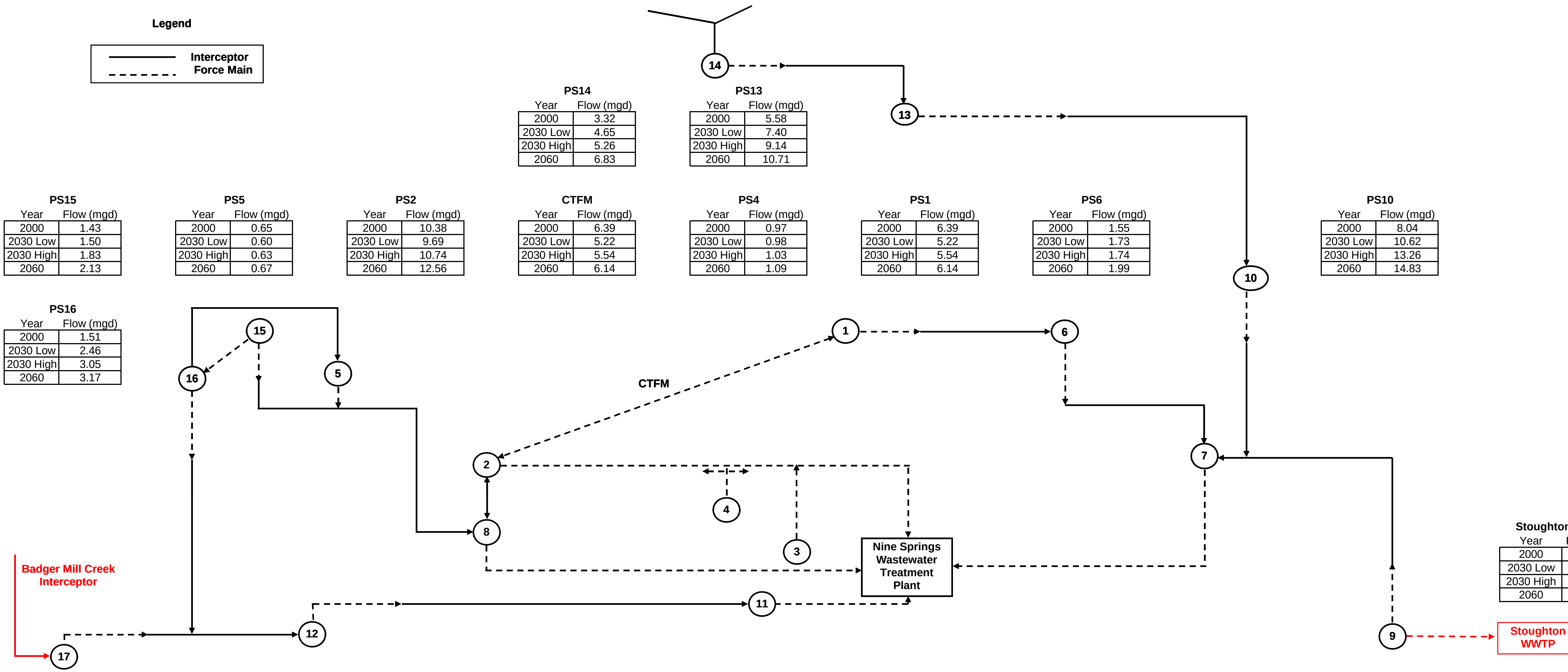
RML:pll\S:\MAD\1500--1599\1547\001\Spr\Technical Memos\CFA Spreadsheets\\CFA Tables Sections 1, 2 and 3 With District Comments (042009).xlsx\Figure 3.02-6 SPWTP Peak Flows
8/26/2009

MADISON METROPOLITAN SEWERAGE DISTRICT
50-YEAR MASTER PLAN (2010-2060)

TECHNICAL MEMO 3
CONVEYANCE FACILITIES ANALYSIS

FIGURE 3.02-7
COLLECTION SYSTEM SCHEMATIC
AVERAGE FLOW SUMMARY–STOUGHTON WWTP OPERATION



PS14

Year	Flow (mgd)
2000	3.32
2030 Low	4.65
2030 High	5.26
2060	6.83

PS13

Year	Flow (mgd)
2000	5.58
2030 Low	7.40
2030 High	9.14
2060	10.71

PS15

Year	Flow (mgd)
2000	1.43
2030 Low	1.50
2030 High	1.83
2060	2.13

PS5

Year	Flow (mgd)
2000	0.65
2030 Low	0.60
2030 High	0.63
2060	0.67

PS2

Year	Flow (mgd)
2000	10.38
2030 Low	9.69
2030 High	10.74
2060	12.56

CTFM

Year	Flow (mgd)
2000	6.39
2030 Low	5.22
2030 High	5.54
2060	6.14

PS4

Year	Flow (mgd)
2000	0.97
2030 Low	0.98
2030 High	1.03
2060	1.09

PS1

Year	Flow (mgd)
2000	6.39
2030 Low	5.22
2030 High	5.54
2060	6.14

PS6

Year	Flow (mgd)
2000	1.55
2030 Low	1.73
2030 High	1.74
2060	1.99

PS10

Year	Flow (mgd)
2000	8.04
2030 Low	10.62
2030 High	13.26
2060	14.83

PS16

Year	Flow (mgd)
2000	1.51
2030 Low	2.46
2030 High	3.05
2060	3.17

Stoughton WWTP

Year	Flow (mgd)
2000	
2030 Low	2.02
2030 High	3.32
2060	4.69

PS17

Year	Flow (mgd)
2000	0.67
2030 Low	2.22
2030 High	3.41
2060	4.27

PS12

Year	Flow (mgd)
2000	4.47
2030 Low	8.08
2030 High	10.48
2060	11.95

PS8

Year	Flow (mgd)
2000	8.87
2030 Low	8.51
2030 High	9.31
2060	10.09

PS11

Year	Flow (mgd)
2000	7.56
2030 Low	12.04
2030 High	15.03
2060	17.63

PS3

Year	Flow (mgd)
2000	0.31
2030 Low	7.40
2030 High	0.35
2060	0.35

NSWTP

Year	Flow (mgd)
2000	41.29
2030 Low	49.68
2030 High	57.08
2060	55.71

PS7

Year	Flow (mgd)
2000	13.20
2030 Low	16.12
2030 High	20.62
2060	24.23

PS9

Year	Flow (mgd)
2000	0.81
2030 Low	0.34
2030 High	0.36
2060	0.36

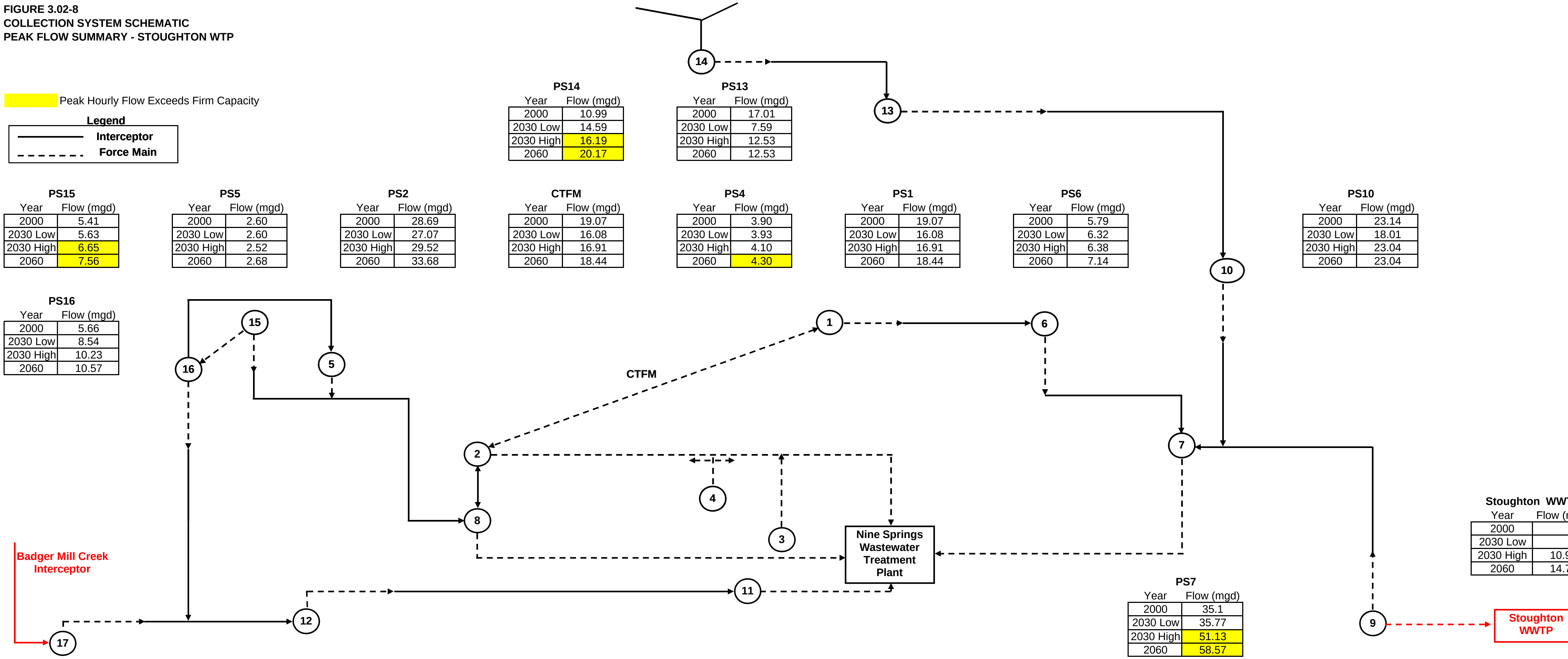
MADISON METROPOLITAN SEWERAGE DISTRICT
50-YEAR MASTER PLAN (2010-2060)

TECHNICAL MEMO 3
CONVEYANCE FACILITIES ANALYSIS

FIGURE 3.02-8
COLLECTION SYSTEM SCHEMATIC
PEAK FLOW SUMMARY - STOUGHTON WTP

Peak Hourly Flow Exceeds Firm Capacity

Legend
Interceptor
Force Main



PS14

Year	Flow (mgd)
2000	10.99
2030 Low	14.59
2030 High	16.19
2060	20.17

PS13

Year	Flow (mgd)
2000	17.01
2030 Low	7.59
2030 High	12.53
2060	12.53

PS15

Year	Flow (mgd)
2000	5.41
2030 Low	5.63
2030 High	6.65
2060	7.56

PS5

Year	Flow (mgd)
2000	2.60
2030 Low	2.60
2030 High	2.52
2060	2.68

PS2

Year	Flow (mgd)
2000	28.69
2030 Low	27.07
2030 High	29.52
2060	33.68

CTFM

Year	Flow (mgd)
2000	19.07
2030 Low	16.08
2030 High	16.91
2060	18.44

PS4

Year	Flow (mgd)
2000	3.90
2030 Low	3.93
2030 High	4.10
2060	4.30

PS1

Year	Flow (mgd)
2000	19.07
2030 Low	16.08
2030 High	16.91
2060	18.44

PS6

Year	Flow (mgd)
2000	5.79
2030 Low	6.32
2030 High	6.38
2060	7.14

PS10

Year	Flow (mgd)
2000	23.14
2030 Low	18.01
2030 High	23.04
2060	23.04

PS16

Year	Flow (mgd)
2000	5.66
2030 Low	8.54
2030 High	10.23
2060	10.57

Stoughton WWTP

Year	Flow (mgd)
2000	
2030 Low	
2030 High	10.99
2060	14.70

PS7

Year	Flow (mgd)
2000	35.1
2030 Low	35.77
2030 High	51.13
2060	58.57

PS17

Year	Flow (mgd)
2000	2.68
2030 Low	7.83
2030 High	11.24
2060	13.58

PS12

Year	Flow (mgd)
2000	14.11
2030 Low	23.23
2030 High	28.92
2060	32.30

PS8

Year	Flow (mgd)
2000	25.13
2030 Low	24.27
2030 High	26.18
2060	28.01

PS11

Year	Flow (mgd)
2000	21.97
2030 Low	32.50
2030 High	39.18
2060	44.81

PS3

Year	Flow (mgd)
2000	1.24
2030 Low	1.28
2030 High	1.40
2060	1.40

NSWTP

Year	Flow (mgd)
2000	103.23
2030 Low	124.20
2030 High	142.70
2060	139.28

PS9

Year	Flow (mgd)
2000	3.24
2030 Low	1.36
2030 High	1.44
2060	1.44

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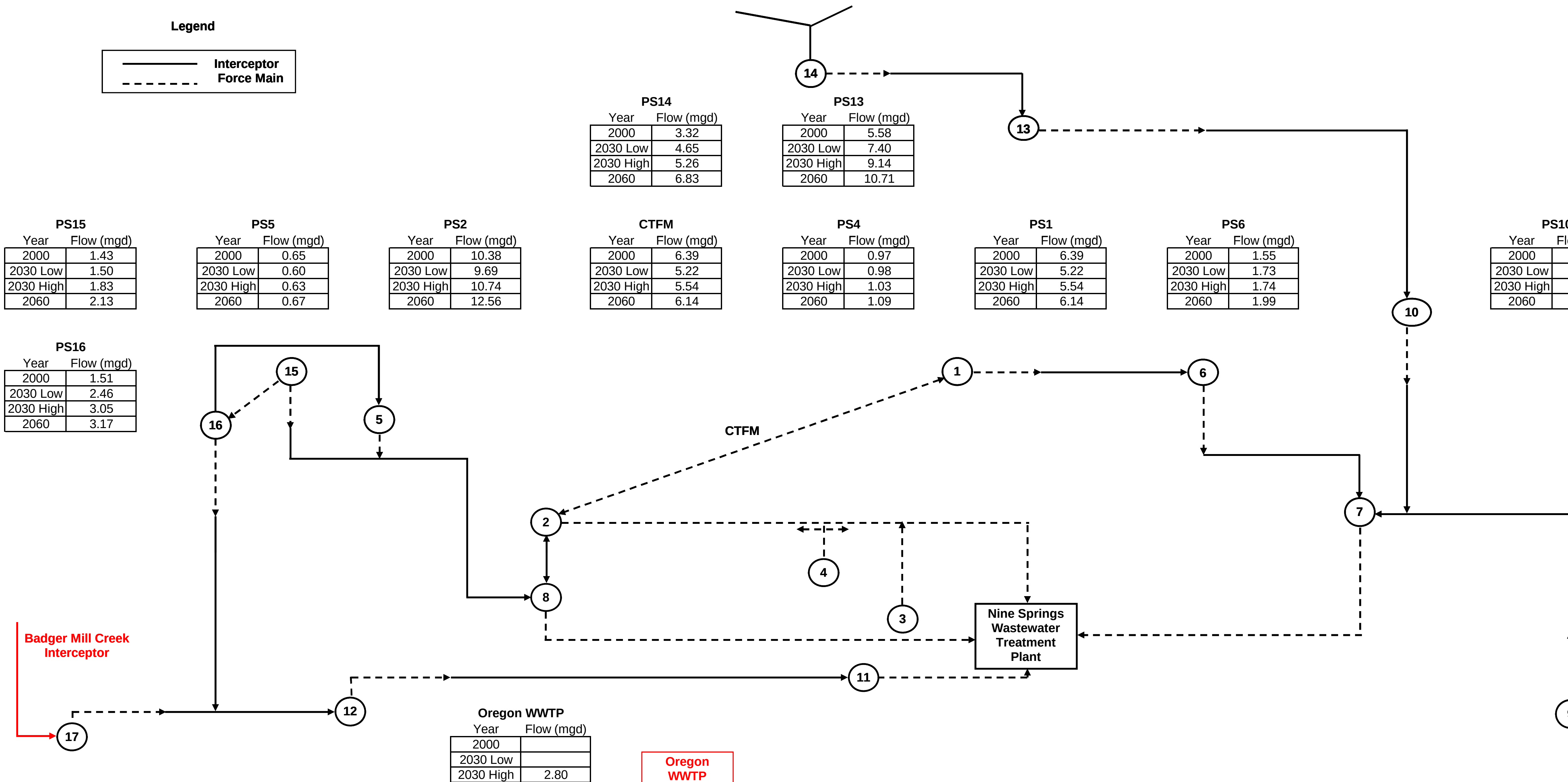
S:\MAD\1500--1599\1547\001\Spr\Technical Memos\CFA Spreadsheets\\CFA Tables Sections 1, 2 and 3 With District Comments (042009).xlsx\Figure 3.02-8 Stoughton PF\8/26/2009

MADISON METROPOLITAN SEWERAGE DISTRICT
50-YEAR MASTER PLAN (2010-2060)

TECHNICAL MEMO 3
CONVEYANCE FACILITIES ANALYSIS

FIGURE 3.02-9
COLLECTION SYSTEM SCHEMATIC
AVERAGE FLOW SUMMARY–OREGON



PS14

Year	Flow (mgd)
2000	3.32
2030 Low	4.65
2030 High	5.26
2060	6.83

PS13

Year	Flow (mgd)
2000	5.58
2030 Low	7.40
2030 High	9.14
2060	10.71

PS15

Year	Flow (mgd)
2000	1.43
2030 Low	1.50
2030 High	1.83
2060	2.13

PS5

Year	Flow (mgd)
2000	0.65
2030 Low	0.60
2030 High	0.63
2060	0.67

PS2

Year	Flow (mgd)
2000	10.38
2030 Low	9.69
2030 High	10.74
2060	12.56

CTFM

Year	Flow (mgd)
2000	6.39
2030 Low	5.22
2030 High	5.54
2060	6.14

PS4

Year	Flow (mgd)
2000	0.97
2030 Low	0.98
2030 High	1.03
2060	1.09

PS1

Year	Flow (mgd)
2000	6.39
2030 Low	5.22
2030 High	5.54
2060	6.14

PS6

Year	Flow (mgd)
2000	1.55
2030 Low	1.73
2030 High	1.74
2060	1.99

PS10

Year	Flow (mgd)
2000	8.04
2030 Low	10.62
2030 High	13.26
2060	14.83

PS16

Year	Flow (mgd)
2000	1.51
2030 Low	2.46
2030 High	3.05
2060	3.17

Oregon WWTP

Year	Flow (mgd)
2000	
2030 Low	
2030 High	2.80
2060	2.80

PS17

Year	Flow (mgd)
2000	0.67
2030 Low	2.22
2030 High	3.41
2060	4.27

PS12

Year	Flow (mgd)
2000	4.47
2030 Low	8.08
2030 High	10.48
2060	11.95

PS8

Year	Flow (mgd)
2000	8.87
2030 Low	8.51
2030 High	9.31
2060	10.09

PS11

Year	Flow (mgd)
2000	7.56
2030 Low	12.04
2030 High	17.83
2060	20.43

PS3

Year	Flow (mgd)
2000	0.31
2030 Low	7.40
2030 High	0.35
2060	0.35

NSWTP

Year	Flow (mgd)
2000	41.29
2030 Low	49.68
2030 High	63.20
2060	63.20

PS7

Year	Flow (mgd)
2000	13.2
2030 Low	18.14
2030 High	23.94
2060	28.92

PS9

Year	Flow (mgd)
2000	0.81
2030 Low	1.07
2030 High	1.28
2060	1.75

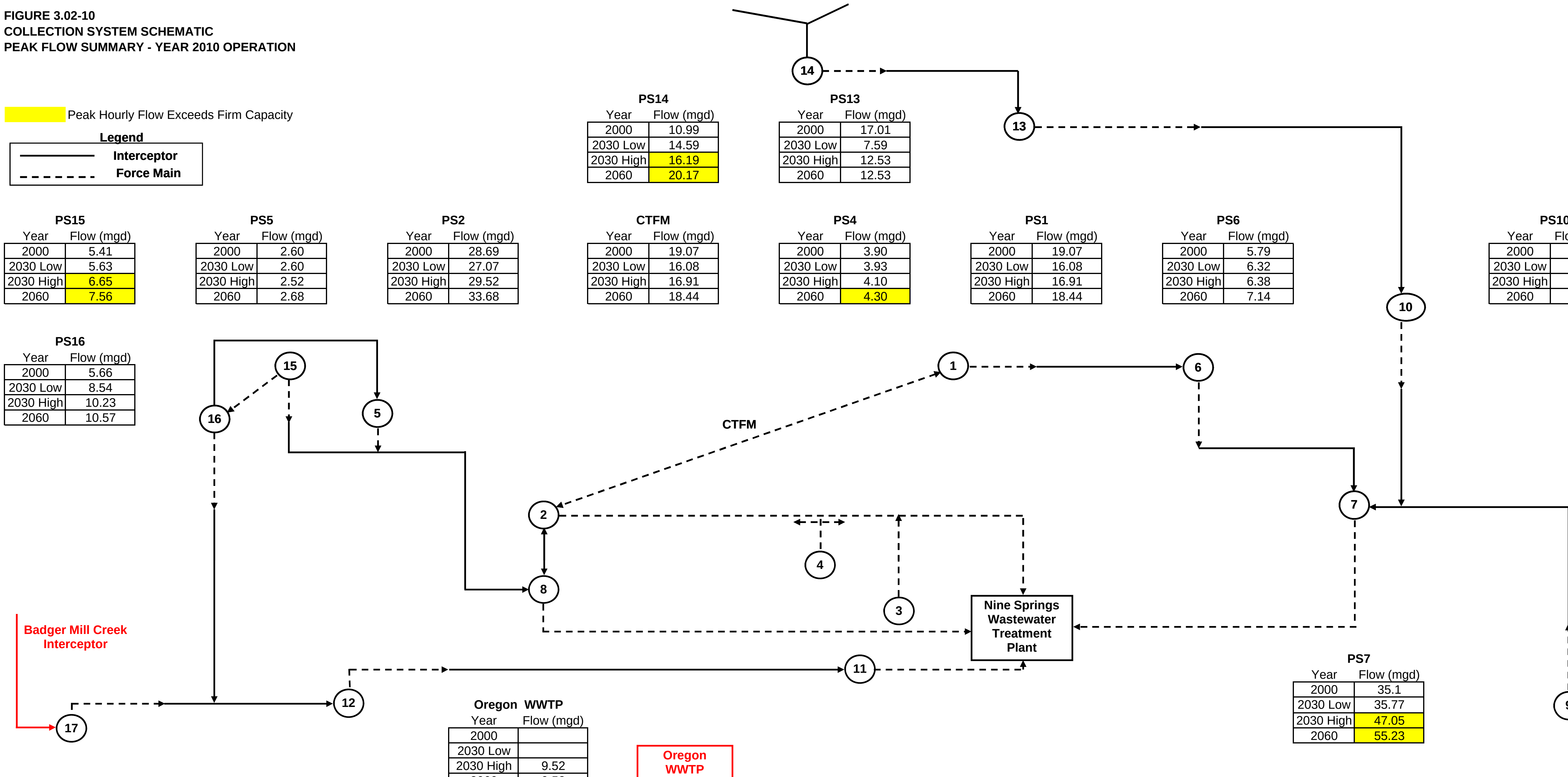
MADISON METROPOLITAN SEWERAGE DISTRICT
50-YEAR MASTER PLAN (2010-2060)

TECHNICAL MEMO 3
CONVEYANCE FACILITIES ANALYSIS

FIGURE 3.02-10
COLLECTION SYSTEM SCHEMATIC
PEAK FLOW SUMMARY - YEAR 2010 OPERATION

Peak Hourly Flow Exceeds Firm Capacity

Legend
Interceptor
Force Main



PS14

Year	Flow (mgd)
2000	10.99
2030 Low	14.59
2030 High	16.19
2060	20.17

PS13

Year	Flow (mgd)
2000	17.01
2030 Low	7.59
2030 High	12.53
2060	12.53

PS15

Year	Flow (mgd)
2000	5.41
2030 Low	5.63
2030 High	6.65
2060	7.56

PS5

Year	Flow (mgd)
2000	2.60
2030 Low	2.60
2030 High	2.52
2060	2.68

PS2

Year	Flow (mgd)
2000	28.69
2030 Low	27.07
2030 High	29.52
2060	33.68

CTFM

Year	Flow (mgd)
2000	19.07
2030 Low	16.08
2030 High	16.91
2060	18.44

PS4

Year	Flow (mgd)
2000	3.90
2030 Low	3.93
2030 High	4.10
2060	4.30

PS1

Year	Flow (mgd)
2000	19.07
2030 Low	16.08
2030 High	16.91
2060	18.44

PS6

Year	Flow (mgd)
2000	5.79
2030 Low	6.32
2030 High	6.38
2060	7.14

PS10

Year	Flow (mgd)
2000	23.14
2030 Low	18.01
2030 High	23.04
2060	23.04

PS16

Year	Flow (mgd)
2000	5.66
2030 Low	8.54
2030 High	10.23
2060	10.57

PS7

Year	Flow (mgd)
2000	35.1
2030 Low	35.77
2030 High	47.05
2060	55.23

Oregon WWTP

Year	Flow (mgd)
2000	
2030 Low	
2030 High	9.52
2060	9.52

PS17

Year	Flow (mgd)
2000	2.68
2030 Low	7.83
2030 High	11.24
2060	13.58

PS12

Year	Flow (mgd)
2000	14.11
2030 Low	23.23
2030 High	28.92
2060	32.30

PS8

Year	Flow (mgd)
2000	25.13
2030 Low	24.27
2030 High	26.18
2060	28.01

PS11

Year	Flow (mgd)
2000	21.97
2030 Low	32.50
2030 High	45.24
2060	51.08

PS3

Year	Flow (mgd)
2000	1.24
2030 Low	1.28
2030 High	1.40
2060	1.40

NSWTP

Year	Flow (mgd)
2000	103.23
2030 Low	124.20
2030 High	158.00
2060	158.00

PS9

Year	Flow (mgd)
2000	3.24
2030 Low	4.23
2030 High	4.92
2060	6.41

SECTION 4
EXISTING CONVEYANCE FACILITIES
PROJECTED COSTS WITH NO SATELLITE OPTIONS

4.01 INCORPORATION INTO ALTERNATIVE ANALYSES AND MMSD 50-YEAR FINANCIAL MODEL

The projected capital and operating costs presented in this section will serve as a baseline for alternative analysis for conveyance system projects that would be impacted by watershed diversions and identify conveyance system projects required regardless of any potential watershed diversions. This will essentially be the no-action alternative (or baseline) for comparison with other potential alternatives. These will be divided into projects related to pumping stations resulting from either insufficient capacity or condition, force mains resulting from insufficient capacity or condition, and interceptor projects resulting from insufficient capacity or condition.

4.02 PUMPING STATIONS PROJECTED CAPITAL COSTS

Section 3.01.A presented potential projects for pumping stations related to capacity needs without any watershed diversions. In addition, Section 2 presented projects required for pumping station maintenance independent of capacity needs as well as projects required to provide adequate standby power. Table 4.02-1 summarizes the project costs for both capacity issues and rehabilitation. All costs presented in this table are based on MMSD estimates included in the 2009 Capital Projects Budget.

A. Projects to Address Capacity Issues

The following pumping station projects are focused primarily on capacity related issues although rehabilitation is a major component of some of these pumping station projects and should be assessed at the time of the project:

1. PS 9
2. PS 11
3. PS 12
4. PS 13
5. PS 14
6. PS 17
7. PS 18

B. Projects to Address Station Condition

The following pumping station projects are focused primarily on condition-related issues but capacity should be addressed during the rehabilitation project, if appropriate:

1. PS 3
2. PS 4
3. PS 7
4. PS 15

C. Projects to Address Standby Power Needs

In the MMSD 2009 Capital Projects Budget there is one indicated project related to standby power needs: Backup Power for PS 7 (MMSD Project 440-00-20) scheduled for completion in 2009. The need for standby power should be reviewed with each future pumping station project.

MADISON METROPOLITAN SEWERAGE DISTRICT
50-YEAR MASTER PLAN (2010-2060)

TECHNICAL MEMO 3
CONVEYANCE FACILITIES ANALYSIS

TABLE 4.02-1
PUMPING STATION PROJECTS AND PROJECTED COSTS (DECEMBER 2008 DOLLARS)

	Projected Date		Scheduled	MMSD Project		Master Plan Project		Primary
Pumping Station	High Flow	Low Flow	Date	Number	Budget	Number	Budget	Driving
								Force
PS 3			2019	853-00-50	$1,090,000			Condition
PS 4	2045		2019	853-00-50	$1,090,000			Condition
PS 7	2006	2010	2015	857-00-70	$1,110,000	`		Capacity
PS 9	2022	2040	2019	853-00-50	$1,090,000			Capacity
PS 11	2005	2010	2016	861-00-52	$4,260,000			Capacity
PS 12	2005	2008	2016	861-00-52	$4,260,000			Capacity
PS 13	2010	2020	2018	863-00-50	$4,260,000			Capacity
PS 14	2023	2038	2018	863-00-50	$4,260,000			Capacity
PS 15	2028	2050	2017	864-00-50	$4,270,000			Condition
PS 17	2007	2011	2014	867-00-50	$2,200,000	17-1	$4,250,000	Capacity
PS 18	2006	2010	2015	868-00-51	$8,000,000			Capacity of PS 7
Totals					$35,890,000		$4,250,000	

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S:\MAD\1500--1599\1547\001\Spr\Technical Memos\CFA Spreadsheets\Section 4 of CFA Tables.xlsx\Table 4-02-1 Pumping Stations\8/26/2009

4.03 FORCE MAIN PROJECTED CAPITAL COSTS

Section 3.01 B presented potential projects for force mains related to capacity needs without any watershed diversions. In addition, Section 2 presented projects required for force main condition (age) independent of capacity needs. Table 4.03-1 summarizes the project costs for both capacity issues and force main age. The following costs were used for projects that were not included in the 2009 MMSD Capital Projects Budget:

Pipe Diameter	Project Cost Per LF
18	$175
21	$200
24	$250
30	$325
36	$400
42	$500

All costs presented in this table are based on December 2008 costs. The three projects that do not have costs in Table 4.03-1 will likely not be required. The CTFM PSs 2, 3, and 4 force main exceeds the nominal 8 fps by only 0.1 fps. This will likely not have a significant impact on the operation of any of the pumps in PSs 2, 3, or 4. The PS 7 force mains will likely not be required to be relieved since PS 18 and the PS 18 force main will provide additional capacity in the MMSD system parallel to the existing PS 7 force mains.

MADISON METROPOLITAN SEWERAGE DISTRICT
50-YEAR MASTER PLAN (2010-2060)

TECHNICAL MEMO 3
CONVEYANCE FACILITIES ANALYSIS

TABLE 4.03-1
PROJECTED FORCE MAIN CAPACITY COSTS

	Project Elements		Projected Date		Scheduled	MMSD Project		Master Plan Project	
Force Main	Length (Ft)	Diameter(in)	High Flow	Low Flow	Date	Number	Budget	Number	Budget
PS 2,3 and 4 FM			2050					3-2	$0
PS 7 FMi			2025	2052				7-111	$0
PS 7 FMii			2042					7-112	$0
PS 10 FM	11,109	36	2040					10-6	$4,400,000
PS 11 FM	4,164	36	2025	2050				11-6	$1,700,000
PS 17 Fmi	13,357	16	2015	2026				17-21	$2,300,000
PS 17 Fmii	3,071	20	2031	2060				17-22	$600,000
PS 18 FM	15,000	42	2025	2052	2013	868-00-52	$8,000,000		
Totals							$8,000,000		$9,000,000

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S:\MAD\1500--1599\1547\001\Spr\Technical Memos\CFA Spreadsheets\Section 4 of CFA Tables.xlsx\Table 4.03-1 FM Projects\8/26/2009

4.04 INTERCEPTOR PROJECTED CAPITAL COSTS

Section 3.01 C presented potential projects for interceptors related to capacity needs without any watershed diversions. In addition, Section 2 presented projects requiring interceptor maintenance independent of capacity needs. Table 4.04-1 summarizes the project costs for capacity-related issues only. The costs presented in this table were based on providing a relief sewer the same size as the existing sewer. The following costs were used:

Pipe Diameter	Project Cost Per LF
18	$275
21	$300
24	450
27	$475
30	$500
36	$600
42	$700
48	$800
54	$950
60	$1,100

All costs presented in this table are based on December 2008 costs. Section 3 presented a summary of these projects. The total indicated system mileage with capacity-related issues is 42.59 miles or approximately 45 percent of the year 2010 interceptor length. Table 4.04-2 summarizes condition related projects included in the 2009 MMSD Capital Projects Budget.

TABLE 4.04-1
PROJECTED INTERCEPTOR CAPACITY COSTS(DECEMBER 2008 DOLLARS)

				Diameter (Inches)		Projected		Projected Date		Master Plan		MMSD Project		
Interceptor Segment	Upstream Manhole	Downstream Manhole	Length	Existing	Proposed Relief	Cost ($/LF)	Description	High Flow	Low Flow	Project Number	Projected Cost	Project Number	Scheduled	Budget
2Xii	02-300	02-101	3	24	24	450	WI-Spring Street Relief	2000	2000	2-1	$0			
2Bi	02-011	02-008	900	24	24	450	OWI from MH 02-012	2000	2000	2-21	$400,000			
2Bii	02-008	02-005A	1,260	24	24	450	OWI from MH 02-012	2000	2000	2-22	$600,000			
3i	03-111	03-102	3,492	12	18	275	Rimrock Interceptor	2000	2000	3-1	$1,000,000			
3ii	03-102	PS 3	308	10	18	275	Rimrock Interceptor	2000	2000	3-1	$100,000			
7B	07-437	07-426	5,510	18	18	275	FEI-Cottage Grove Extension					841-00-57		$500,000
7Ci	07-734	07-728	2,917	21	36	600	FEI-Door Creek Extension	2018	2041	7-31	$1,800,000			
7Cii	07-728	07-723	2,496	21	30	500	FEI-Door Creek Extension	2022	2048	7-32	$1,200,000			
7Ciii	07-723	07-707	7,889	24	30	500	FEI-Door Creek Extension	2025	2052	7-33	$3,900,000			
7Civ	07-707	07-426	3,474	24	30	500	FEI-Door Creek Extension	2030	2060	7-34	$1,700,000			
7Di	07-426	07-425	153	36	36	600	FEI-Upstream of Interstate	2035		7-41	$100,000			
7Dii	07-425	07-416	3,861	30	36	600	FEI-Upstream of Interstate	2018	2037	7-42	$2,300,000			
7Diii	07-416	07-415	355	42	42	700	FEI-Upstream of Interstate	2053		7-43	$200,000			
7E	07-415	07-932	8,067	42	42	700	FEI-Downstream of Interstate	2052		7-5	$5,600,000			
7Fi	07-932	07-313	14	42	60	1,100	NEI-Downstream of FEI	2000	2000	7-61	$0	839-00-79	2014	$5,300,000
7Fii	07-313	07-215	5,591	48	48	800	NEI-Downstream of FEI	2009	2018	7-62	$4,500,000	839-00-79		
7Ji	07-249	07-242	2,794	18	24	450	SEI-Blooming Grove Extension	2011	2038	7-71	$1,300,000			
7Jii	07-242	07-231	4,974	24	24	450	SEI-Blooming Grove Extension	2021	2050	7-72	$2,200,000			
7Jiii	07-231	07-228	1,347	24	24	450	SEI-Blooming Grove Extension	2029	2059	7-73	$600,000			
7Ki	07-228	07-224	2,001	30	30	500	SEI-McFarland Relief-Downstream of Blooming Grove Ext.	2034		7-81	$1,000,000			
7Kii	07-224	07-222	650	30	30	500	SEI-McFarland Relief-Downstream of Blooming Grove Ext.	2029	2058	7-82	$300,000			
7Kiii	07-222	07-218	1,647	36	36	600	SEI-McFarland Relief-Downstream of Blooming Grove Ext.	2032		7-83	$1,000,000			
7Kiv	07-218	07-215	1,606	36	36	600	SEI-McFarland Relief	2039		7-84	$1,000,000			
7Mi	07-215	07-211	2,468	60	60	1,100	SEI-Downstream of NEI	2010	2022	7-91	$2,700,000			
7Mii	07-211	PS 7	5,342	60	60	1,100	SEI-Downstream of NEI	2010	2020	7-92	$5,900,000			

TABLE 4.04-1
PROJECTED INTERCEPTOR CAPACITY COSTS(DECEMBER 2008 DOLLARS)

				Diameter (Inches)		Projected		Projected Date		Master Plan		MMSD Project		
Interceptor	Upstream	Downstream	Length	Existing	Proposed	Cost	Description	High Flow	Low Flow	Project Number	Projected Cost	Project Number	Scheduled	Budget
Segment	Manhole	Manhole			Relief	($/LF)								
8Aiii	02-545	02-538	3,121	27	27	475	WI Relief to MH 02-519	2000	2000	8-11	$1,500,000			
8Aiv	02-538	02-536	1,200	24	24	450	WI Relief to MH 02-519	2000	2000	8-12	$500,000			
8Av	02-536	02-535	600	21	21	300	WI Relief to MH 02-519	2000	2000	8-13	$200,000			
8Avi	02-535	02-532	841	21	21	300	WI Relief to MH 02-519	2025	2050	8-14	$300,000			
8C	02-531I	02-531A	2,653	21	21	300	WI-Midvale Relief	2028	2058	8-2	$800,000			
8Di	02-531A	02-519	4,363	36	36	600	WI Relief to MH 02-519 after Midvale Relief	2000	2000	8-31	$2,600,000			
8Diii	02-518	02-516	204	36	36	600	WI Relief from MH 02-519	2000	2000	8-32	$100,000			
8Div	02-516	08-228	10	36	36	600	WI Relief from MH 02-519	2000	2000	8-33	$0			
8Xi	08-207	02-503	463	24	24	450	WI Relief from MH 02-519 after Junction with Campus Relief	2048		8-91	$200,000			
8Xii	08-207	08-201	1,234	36	36	600	WI Campus Relief after Junction with Wi Relief	2049		8-92	$700,000			
8Xiii	08-121	08-120	16	30	30	500	WI Randall Relief to SWI	2000	2000	8-93	$0			
8Fvi	08-215	PB 08-214	27	27	30	500	WI-Campus Relief	2000	2000	8-5	$0			
8Ii	02-041	02-038	1,063	18	18	275	OWI to MH 02-038	2050		8-61	$300,000			
8Iii	02-038	02-034	1,460	18	18	275	OWI from MH 02-038	2020	2049	8-62		832-00-72	2011	$600,000
8Ji	02-034	02-032	816	20	21	300	OWI from MH 02-038	2012	2040	8-71		832-00-72		
8Jii	02-032	02-513	1,704	21	21	300	OWI from MH 02-038	2022	2052	8-72		832-00-72		
8R	02-150	02-145	1,215	24	24	450	SWI	2057			$500,000			
8Sii	02-142	02-136	1,669	27	27	475	SWI	2040		8-8	$800,000			
9A	09-108	09-104	1,678	24	24	450	SEI-Upstream of PS 9	2038		9-1	$800,000			
9Bi	09-104	09-101	1,373	27	27	475	SEI Upstream of MH 09-101	2058		9-21	$700,000			
9Bii	09-101	PS 9	285	24	24	450		2036		9-22	$100,000			

TABLE 4.04-1
PROJECTED INTERCEPTOR CAPACITY COSTS(DECEMBER 2008 DOLLARS)

				Diameter (Inches)		Projected		Projected Date		Master Plan		MMSD Project		
Interceptor Segment	Upstream Manhole	Downstream Manhole	Length	Existing	Proposed Relief	Cost ($/LF)	Description	High Flow	Low Flow	Project Number	Projected Cost	Project Number	Scheduled	Budget
10A	10-145	10-121	10,973	48	48	800	NEI PS 13 to PS 10	2018	2033	10-1		839-00-78	2011	$10,200,000
10Bi	10-121	10-118	874	36	36	600	NEI PS 13 to PS 10	2005	2009	10-21	$500,000			
10Bii	10-118	10-201	1,597	42	42	700	NEI PS 13 to PS 10	2000	2000	10-22	$1,100,000			
10Ei	10-201	10-115	140	42	42	700	NEI PS 13 to PS 10	2000	2000	10-31	$100,000			
10Eii	10-115	10-104A	4,412	48	48	800	NEI PS 13 to PS 10-Downstream of Lien Extension	2000	2000	10-32	$3,500,000			
10Eiii	10-104A	10-102A	1,110	48	48	800	NEI PS 13 to PS 10-Downstream of Lien Extension	2000	2000	10-33	$900,000			
10G	10-102A	10-101	959	48	48	800	NEI-Downstream of Hwy 30 Extension	2000	2000	10-4	$800,000			
10H	10-101	PS10	108	48	48	800	NEI-Downstream of Hwy 30 Extension	2000	2000	10-5	$100,000			
11Aii	11-171	11-169	812	42	42	700	NSVI PS 12 to MH 11-127	2021	2035	11-11	$600,000			
11Aiii	11-169	11-167	465	42	42	700	NSVI PS 12 to MH 11-127	2019	2030	11-12	$300,000			
11Aiv	11-167	111-161E	1,436	42	42	700	NSVI PS 12 to MH 11-127	2021	2035	11-13	$1,000,000			
11Avi	11-161A	11-159	1,321	36	36	600	NSVI PS 12 to MH 11-127	2025	2046	11-14	$800,000			
11Bi	11-159	11-158	340	36	36	600	NSVI PS 12 to MH 11-127	2023	2042	11-21	$200,000			
11Biii	11-156	11-151A	2,220	42	42	700	NSVI PS 12 to MH 11-127	2028	2052	11-22	$1,600,000			
11C	11-151A	11-145	3,784	42	42	700	NSVI PS 12 to MH 11-127	2025	2050	11-3	$2,600,000			
11Di	11-145	11-141	3,784	36	36	600	NSVI PS 12 to MH 11-127	2055		11-41	$2,300,000			
11Dii	11-141	11-137	1,648	30	33	500	NSVI PS 12 to MH 11-127	2038		11-42	$800,000			
11Diii	11-137	11-129	3,995	33	33	500	NSVI PS 12 to MH 11-127	2022	2041	11-43	$2,000,000			
11Div	11-129	11-127	733	36	36	600	NSVI PS 12 to MH 11-127	2031	2060	11-44	$400,000			
11Dv	11-127	11-116A	4,855	54	54	950	NSVI to PS 11	2021	2040	11-45	$4,600,000			
11Fi	11-116A	11-111A	2,788	54	54	950	NSVI to PS 11 after Syene Extension	2021	2037	11-51	$2,600,000			
11Fii	11-111A	11-106A	2,716	54	54	950	NSVI to PS 11 after Syene Extension	2019	2030	11-52	$2,600,000			
11Fiii	11-106A	11-104	1,689	54	54	950	NSVI to PS 11 after HWY 14 Extension	2018	2028	11-53	$1,600,000			
11Fiv	11-104	PS11	1,525	54	54	950	NSVI to PS 11 after HWY 14 Extension	2016	2027	11-54	$1,400,000			
12Hi	12-110	12-101	3,484	48	48	800	NSVI to PS 12	2018	2029	12-11	$2,800,000			
12Hii	12-101	PS 12	38	48	48	800	NSVI to PS 12	2017	2028	12-12	$0			
	CTH PD	12-207	3,600		18	275	NSVI - Morse Pond Extension					838-00-62	2012	$700,000
13G	13-132	13-122A	4,397	48	48	800	NEI PS 14 to PS 13	2056		13-1	$3,500,000			
13A-Ei	13-122A	13-116H	153	48	48	800	NEI PS 14 to PS 13	2020	2030	13-2	$100,000			
13Hi	13-105A	13-105	125	46.5	48	800	NEI PS 14 to PS 13	2037	2051	13-31	$100,000			
13Hii	13-105	PS 13	1,758	48	48	800	NEI PS 14 to PS 13	2026	2051	13-32	$1,400,000			

TABLE 4.04-1
PROJECTED INTERCEPTOR CAPACITY COSTS(DECEMBER 2008 DOLLARS)

				Diameter (Inches)		Projected		Projected Date		Master Plan		MMSD Project		
Interceptor Segment	Upstream Manhole	Downstream Manhole	Length	Existing	Proposed Relief	Cost ($/LF)	Description	High Flow	Low Flow	Project Number	Projected Cost	Project Number	Scheduled	Budget
14B	14-196	14-193	1,203	21	21	300	NEI-Deforest Extension	2023	2045	14-1	$400,000			
14D	14-182	14-171	5,724	21	21	300	NEI-Deforest Extension	2053		14-12	$1,700,000			
14E	14-171	14-166	2,351	21	21	300	NEI-Deforest Extension	2051		14-13	$700,000			
14Fi	14-166	14-165	488	21	21	300	NEI-Deforest Extension	2052		14-21	$100,000			
14Fii	14-165	14-162	1,401	24	24	450	NEI-Deforest Extension	2056		14-22	$600,000			
14G	14-162	14-156	2,687	24	24	450	NEI-Deforest Extension	2050		14-3	$1,200,000			
14Jii	14-415	14-407	2,241	15	18	275	NEI-Hwy 19 Extension	2005	2010	14-15	$600,000			
14Jv	14-407	14-134	3,059	18	18	275	NEI-Hwy 19 Extension	2050		14-16	$800,000			
14K	14-134	14-102	16,679	36	36	600	NEI:DeForest Extension after HWY 19 Extension	2050		14-5	$10,000,000			
14Li	14-362	14-358	775	10	24	450	NEI-Waunakee Extension	2025	2040	14-6	$300,000			
14Lii	14-358	14-356	674	24	24	450	NEI-Waunakee Extension	2054			$300,000			
14Mi	14-356	14-345	4,659	24	24	450	NEI-Waunakee Extension	2011	2015	14-7	$2,100,000			
14Mii	14-345	14-338	2,859	21	21	300	NEI-Waunakee Extension	2028	2048	14-17	$900,000			
14Miii	14-338	14-333	2,110	30	30	500	NEI-Waunakee Extension	2045		14-18	$1,100,000			
14Miv	14-333	14-323	4,889	30	30	500	NEI-Waunakee Extension	2030			$2,400,000			
14N	14-323	14-315	4,055	30	30	500	NEI-Waunakee Extension	2022	2030	14-8	$2,000,000			
14Oi	14-315	14-301	5,251	30	30	500	NEI-Waunakee Extension	2040		14-9	$2,600,000			
15A	05-116	05-115	2,099	14	18	275	WI-West Extension (Siphon)	2051		15-3	$600,000			
15Ci	05-113	05-106	227	24	24	450	WI-West Extension	2050		15-1	$100,000			
16Aii	05-315	05-310	1,002	18	18	275	WI to PS 16	2048		16-11	$300,000			
16Aiv	05-306	05-236	1,771	24	24	450	WI to PS 16	2024	2046	16-12	$800,000			
17 B, C		17-128					Remainder of LBMCI from Norther Lights Trail to Midtown Rd.					843-00-50	2014	$5,400,000
			230,605								$119,400,000			$22,700,000

MADISON METROPOLITAN SEWERAGE DISTRICT
50-YEAR MASTER PLAN (2010-2060)

TECHNICAL MEMO 3
CONVEYANCE FACILITIES ANALYSIS

TABLE 4.04-2

				Diameter		Projected Date		Master Plan		MMSD Project		
Interceptor	Upstream	Downstream	Length	(Inches)	Description	High Flow	Low Flow	Project Number	Projected Cost	Project Number	Scheduled	Budget
Segment	Manhole	Manhole										
2E	08-106	02-606	1438	24	SWI-Haywood Street Extension					831-00-51	2018	$700,000
5A	05-111	PS 15	3681	14,16,30	WI-Upstream of PS 5					832-00-70	2011	$1,000,000
6Ai	06-209	06-206	1240	15	East Monona Interceptor					836-00-73	2012	$500,000
4A	04-437	04-312	1420		SI-Baird Street Replacement					840-00-50	2011	$300,000
					Annual Condition Repair Estimates							$25,000,000
					NSVI Relining							$7,800,000
												$32,800,000

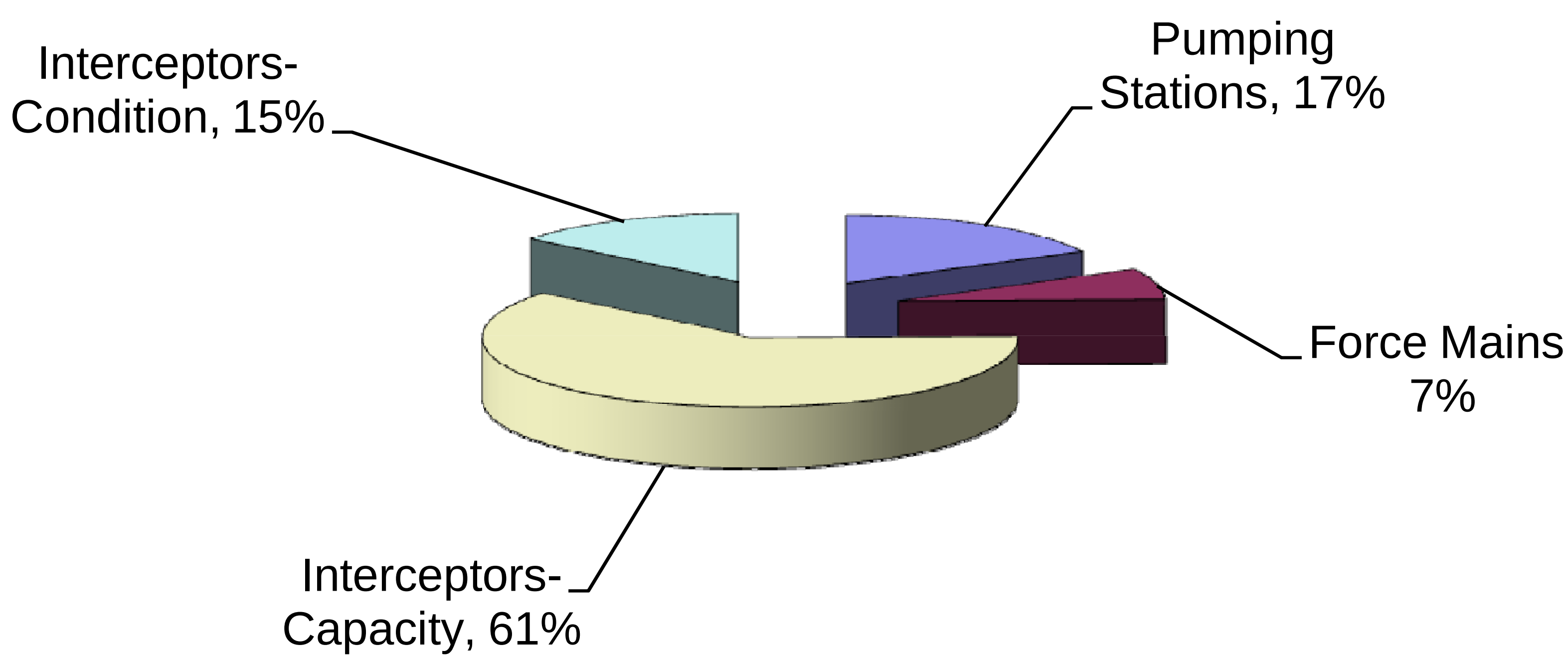
4.05 PROJECTED OVERALL CONVEYANCE CAPACITY COSTS

Figure 4.05-1 summarizes the percentage of the projected 2010 to 2060 conveyance system costs by type of infrastructure. The projected capital costs for each category are as follows:

Infrastructure Element	Projected Cost
Pumping Stations	$ 40,140,000
Force Mains	$ 17,000,000
Interceptors-Capacity	$ 142,100,000
Interceptors-Condition	$ 35,300,000
Total	**$ 234,540,000**

Costs presented in this table are from Tables 4.02-1 (Pumping Stations), 4.03-1 (Force Mains), 4.04-1 (Interceptors-Capacity), and 4.04-2 (Interceptors-Condition).

Figure 4.05-1
Distribution of Projected Conveyance Costs



APPENDIX A

TABLES A1 AND A2 (SEE CHAPTER 4 OF *MMSD COLLECTION SYSTEM EVALUATION* FOR INTERCEPTOR SEGMENT FLOWS)

MADISON METROPOLITAN SEWERAGE DISTRICT
50-YEAR MASTER PLAN (2010-2060)

TECHNICAL MEMO 3
DEVELOPMENT OF FLOWS

TABLE A 1
SERVICE AREA FLOWS

	Year									
	2000		2030				2060			
	Wastewater Flow		Wastewater Flow-TAZ		Wastewater Flow-High		Wastewater Flow			
Pumping Station	Average Daily	Peak Flow	Average Daily	Peak Flow		Peak Flow	Average Daily Flow		Peak Flow	
Service Area	(mgd)	(mgd)	(mgd)	(mgd)	(mgd)	(mgd)	(mgd)	(mgd)	(mgd)	(mgd)
							Low	High	Low	High
1	6.39	19.06	5.22	16.08	5.54	16.90	5.54	6.14	16.91	0.01
2	3.99	12.83	4.47	14.10	5.21	16.04	5.21	6.42	16.06	19.14
3	0.31	1.24	0.32	1.29	0.35	1.40	0.35	0.35	1.40	1.40
4	0.97	3.89	0.98	3.93	1.03	4.09	1.03	1.09	4.10	4.29
5	0.65	2.59	0.60	2.40	0.63	2.52	0.63	0.67	2.52	2.68
6	1.55	5.77	1.73	6.36	1.74	6.37	1.74	1.99	6.38	7.14
7	2.81	9.55	4.72	14.77	7.66	22.21	7.66	10.36	22.21	28.63
8	6.79	20.06	6.41	19.12	6.85	20.21	6.85	7.29	20.22	21.31
9	0.81	3.22	1.07	4.24	1.28	4.93	1.28	1.75	4.92	6.39
10	2.46	8.54	3.23	10.73	4.12	13.18	4.12	4.12	13.18	13.18
11	3.09	10.34	3.96	12.74	4.54	14.30	4.54	5.68	14.30	17.28
12	2.29	8.03	3.41	11.23	4.02	12.89	4.02	4.52	12.91	14.24
13	2.25	7.92	2.75	9.37	3.88	12.52	3.88	3.88	12.53	12.52
14	3.32	11.00	4.65	14.58	5.26	16.18	5.26	6.83	16.19	20.16
15	1.43	5.42	1.50	5.63	1.83	6.65	1.83	2.13	6.65	7.57
16	1.51	5.67	2.46	8.53	3.05	10.24	3.05	3.17	10.23	10.55
17	0.67	2.69	2.22	7.82	3.41	11.25	3.41	4.27	11.24	13.57
NSWTP	41.29	103.22	49.70	124.25	60.40	151.00	60.40	70.66	151.00	176.65
Notes	1. All flows presented in this table were prepared by CARPC. Peak flows are based on average daily flows multiplied by the MDC except for flows greater than 20 mgd (2.5 x) or flows less than 1 mgd (4x).									
	2. The Uncertainty Factors for the year 2030 were developed by CARPC analysis.									

TECHNICAL MEMO 3
DEVELOPMENT OF FLOWS

TABLE A 2
PUMPING STATION FLOWS

		Year									
		2000		**2030**				**2060**			
		Wastewater Flow		**Wastewater Flow-TAZ**		**Wastewater Flow-UF**				**Wastewater Flow**	
Pumping Station		**Average Daily**	**Peak Flow**	**Average Daily**	**Peak Flow**	`	**Peak Flow**	**Average Daily**	**Average Daily**	**Peak Flow**	**Peak Flow**
Service Area		**(mgd)**	**(mgd)**	**(mgd)**	**(mgd)**	**(mgd)**	**(mgd)**	**(mgd)**	**(mgd)**	**(mgd)**	**(mgd)**
								Low	**High**		
1		6.39	19.06	5.22	16.08	5.54	16.90	5.54	6.14	16.91	18.44
2		10.38	28.69	9.69	27.06	10.74	29.53	10.74	12.56	29.52	33.69
3		0.31	1.24	0.32	1.29	0.35	1.40	0.35	0.35	1.40	1.40
4		0.97	3.89	0.98	3.93	1.03	4.09	1.03	1.09	4.10	4.29
5		0.65	2.59	0.60	2.40	0.63	2.52	0.63	0.67	2.53	2.68
6		1.55	5.77	1.73	6.36	1.74	6.37	1.74	1.99	6.38	7.14
7		13.20	35.13	18.14	45.90	23.94	59.85	23.94	28.92	59.85	72.30
8		8.87	25.13	8.51	24.27	9.31	26.17	9.31	10.09	26.18	28.02
9		0.81	3.24	1.07	4.24	1.28	4.93	1.28	1.75	4.92	6.39
10		8.04	23.13	10.62	29.25	13.26	35.26	13.26	14.83	35.26	38.74
11		7.56	21.98	12.04	32.51	15.03	39.17	15.03	17.63	39.18	44.82
12		4.47	14.12	8.08	23.24	10.48	28.93	10.48	11.95	28.92	32.30
13		5.58	17.00	7.40	21.56	9.14	25.77	9.14	10.71	25.77	29.44
14		3.32	11.00	4.65	14.58	5.26	16.18	5.26	6.83	16.19	20.16
15		1.43	5.42	1.50	5.63	1.83	6.65	1.83	2.13	6.65	7.57
16		1.51	5.67	2.46	8.53	3.05	10.24	3.05	3.17	10.23	10.55
17		0.67	2.69	2.22	7.82	3.41	11.25	3.41	4.27	11.24	13.57
NSWTP		41.29	103.23	49.68	124.20	60.40	151.00	60.40	70.64	151.00	176.60
Notes	1.	All flows presented in this table were prepared by CARPC. Peak flows are based on average daily flows multiplied by the MDC except for average daily flows less than 1									
		daily flow) or flows greater than 20 mgd (2.5 times average daily flow).									
	2.	Flows for PS 1 in this analysis are all routed to PS 2. Typical operation routes about 97% of flow from PS 1 to PS 2 and about 3% of flow to PS 6.									

APPENDIX B
2008 POWER SCHEDULE FOR MMSD PUMPING STATIONS

2008 Power Schedule for District Operated Lift Stations

Updated, 28 February 2008

MMSD Control Room, 222-1201 ext 310, or wireless phone, 225-8470

Madison Gas and Electric, 252-7111, or key customer line, 252-1550

Alliant Energy, 1-800-862-6261

Owner	Station Name	Address	Utility	Account #	Circuit One ID #	Circuit Two ID #	Generator	Normal Outage	High Flow Outage
MMSD	Nine Springs Plant	1610 Moorland Road	MG&E	11224672	NSP 1310	NSP 1313	*None*	30 Minutes	**30 Minutes**
MMSD	Pumping Station 01	104 N. First Street	MG&E	11213857	BLD 1304	RKN 1337	*None*	1 Hour	**30 Minutes**
MMSD	Pumping Station 02	833 W. Washington	MG&E	11212594	ECA 1311	WGA 1313	*None*	1 Hour	**30 Minutes**
MMSD	Pumping Station 03	1610 Moorland Road	MG&E	11208998	NSP 1320	*None*	208 V, 3 Phase	3 Hours +	**1 Hour**
MMSD	Pumping Station 04	522 John Nolen Drive	MG&E	11203098	NSP 1317	NSP 1318	480 V, 3 Phase	3 Hours	**1 Hour**
MMSD	Pumping Station 05	5221 Lake Mendota Dr.	MG&E	16112120	BLK 1335	BLK 1332	480 V, 3 Phase	2 Hours +	**1 Hour**
MMSD	Pumping Station 06	402 Walter Street	MG&E	10602357	MIL 443	RYS 443	*None*	1 Hour if PS 1 is pumping to PS 6	**30 Minutes**
MMSD	Pumping Station 07	6300 Metropolitan Lane-Monona	MG&E	11218260	NSP 1309	PFL 1306	*None*	1 Hour	**30 Minutes**
MMSD	Pumping Station 08	967 Plaenert Drive	MG&E	11208501	WGA 1314	SOM 431	*None*	2 Hours	**1 Hour**
MMSD	Pumping Station 09	4612 Larsen Beach Road-McFarland	**Alliant**	448501	CODN 7253	MCFN 1112	480 V, 3 Phase	2.5 Hours	**1 Hour**
MMSD	Pumping Station 10	110 Regas Road	MG&E	11209012	RYS 1312	RKN 1338	*None*	2.5 Hours	**1 Hour**
MMSD	Pumping Station 11	4760 E. Clayton Road	MG&E	11225026	NSP 1320	NSP 1319	*None*	5 Hours	**3 Hours**
MMSD	Pumping Station 12	2739 Fitchrona Road	MG&E	11226628	FCH 1319	FCH 1316	*None*	2.75 Hours	**1 Hour**
MMSD	Pumping Station 13	3634 Amelia Earhart Dr.	MG&E	11224821	AMN 1313	ETN 1335	*None*	4 Hours	**2 Hours**
MMSD	Pumping Station 14	5000 School Road	MG&E	11209574	HKP 1307	AMN 1311	*None*	6 Hours	**2 Hours**
MMSD	Pumping Station 15	2115 Allen Blvd.-Middleton	MG&E	11213956	PHB 1306	PHB 1305	*None*	1.0 Hours	**30 Minutes**
MMSD	Pumping Station 16	1301 North Gammon Road - Middleton	MG&E	10083723	PHB 1314	PHB 1313	*None*	4 Hours	**2 Hours**
MMSD	Pumping Station 17	407 Bruce Street-Verona	**Alliant**	554395	VER N88	On Site Generator	480 V, 3 Phase	2 Hours	**1 Hour**
City of Madison	American Family	4951 Portage Road	**Alliant**	592629-001	TOC N2443	*None*	208 V, 3 Phase	1.5 Hours	**30 Minutes**
City of Madison	Arbor Hills	2714 W. Beltline Hwy.	MG&E	11195286	WGA 1319	*None*	208 V, 3 Phase	1.5 Hours	**30 Minutes**
City of Madison	Atlas	702 Atlas Ave.	MG&E	11194990	RYS 1310	*None*	240 V, 3 Phase	3.5 Hours	**1 Hour**
City of Madison	Carroll	621 North Carroll Street	MG&E	11196581	NWF 24	*None*	208 V, 3 Phase	1 Hour	**30 Minutes**
City of Madison	Cherokee No. 1	5119 Commanche Way	MG&E	11198124	AMN 1311	*None*	208 V, 3 Phase	3.5 Hours	**1 Hour**
City of Madison	Cherokee No. 2	1550 Commanche Glen	MG&E	11198132	AMN 1311	*None*	208 V, 3 Phase	3.5 Hours	**1 Hour**
City of Madison	Commodore	3100 Lake Mendota Dr.	MG&E	11221462	SHW 432	*None*	208 V, 3 Phase	2 Hour	**30 Minutes**
City of Madison	Debs	407 Debs Road	MG&E	12125605	HKP 1307	*None*	230 V, **1 Phase**	2.5 Hours	**1 Hour**
City of Madison	Diemer	5002 Lake Mendota Dr.	MG&E	11202991	BLK 432	*None*	*None*	5.5 Hours	**2 Hours**
City of Madison	Fayette	5201 Fayette Ave.	MG&E	11199874	NSP 1311	*None*	240 V, 3 Phase	4 Hours	**2 Hours**
City of Madison	Fremont	2405 Fremont Avenue	MG&E	11200417	RKN 1333	*None*	240 V, 3 Phase	1.5 Hours	**30 Minutes**
City of Madison	Gettle	5414 Gettle Avenue	MG&E	11200466	BLK 432	BLK 451	240 V, 3 Phase	1 Hour	**30 Minutes**
City of Madison	Harper	3400 Harper Road	MG&E	10242857	GRE 451	*None*	208 V, 3 Phase	5.75 Hours	**2 Hours**
City of Madison	Hermina	201 Clyde Gallagher Street	MG&E	11197803	FAO 443	*None*	208 V, 3 Phase	4.5 Hours	**2 Hours**
City of Madison	Hoboken	1814 Waunona Way	MG&E	11212602	NSP 1311	*None*	240 V, 3 Phase	4.5 Hours	**2 Hours**
City of Madison	James	3139 James Street	MG&E	11202223	FAO 443	*None*	208 V, 3 Phase	1.5 Hours	**30 Minutes**

Owner	Station Name	Address	Utility	Account #	Circuit One ID #	Circuit Two ID #	Generator	Normal Outage	High Flow Outage
City of Madison	Lois Lowry	7834 Lois Lowry Lane	**Alliant**	564495-001	WTN N7156	*None*	230 V, **1 Phase**	2.5 Hours	**1 Hour**
City of Madison	Lost Pine Trail	9432 Lost Pine Trail	**Alliant**	679476	PLVN8067	*None*	208 V, 3 Phase	4.0 Hours	**3 Hours**
City of Madison	Nelson Road	5950 Nelson Road	**Alliant**	592627-001	BKE N7214	*None*	208 V, 3 Phase	2.5 Hours	**1 Hour**
City of Madison	Regent	3933 Regent Street	MG&E	11209061	WLT 1322	*None*	*None*	6 Hours	**3 Hours**
City of Madison	Shady Point	1842 Shady Point Drive	**Alliant**	622911-001	CCSN5962	*None*	480 V, 3 Phase	2.5 Hours	**1.5 Hours**
City of Madison	Soaring Sky Run	10002 Soaring Sky Run	**Alliant**	645642-001	PLVN8067	*None*	208 V, 3 Phase	3.5 Hours	**2 Hours**
City of Madison	South Point	452 South Point Road	**Alliant**	633849-001	PLVN8067	*None*	480 V, 3 Phase	6 Hours	**3 Hours**
City of Madison	Truax Lift	2701 Anderson Street	MG&E	11194545	AMN 1313	*None*	208 V, 3 Phase	5.5 Hours	**1 Hour**
City of Madison	Veith	4101 Veith Avenue	MG&E	15555246	HKP 1308	*None*	208 V, 3 Phase	1.0 hours	**30 Minutes**
City of Madison	Waunona	3061 Waunona Way	MG&E	11212610	NSP 1317	*None*	208 V, 3 Phase	4.5 Hours	**2 Hours**
City of Madison	Woodley	2712 Waunona Way	MG&E	10774719	NSP 1317	*None*	208 V, 3 Phase	5.5 Hours	**30 Minutes if Waunona is on**
City of Madison	Wright	2722 Wright Street	MG&E	15319627	AMN 1313	*None*	208 V, 3 Phase	6 Hours	**3 Hours**
City of Madsion	Westport	42 Knutson Drive	MG&E	11202876	HKP 1308	*None*	208 V, 3 Phase	5.5 Hours	**2 Hours**
Dane County Parks	Lake Farm Park	3113 Lake Farm Road	MG&E	18709618	NSP 1319	*None*	*None*	4.5 Hours	**3 Hours**
Dunn S.D. #1	Dunn No. 1	2816 Waubesa Avenue	MG&E	16557225	NSP 1320	*None*	240 V, 3 Phase	2.5 Hours	**1 Hour**
Dunn S.D. #1	Dunn No. 2	2917 Waubesa Avenue	MG&E	10834125	NSP 1320	*None*	208 V, 3 Phase	2.5 Hours	**1 Hour**
Dunn S.D. #1	Dunn No. 3	3060 Waucheeta Tr.	MG&E	10835387	NSP 1319	*None*	208 V, 3 Phase	2.5 Hours	**1 Hour**
Dunn S.D. #1	Dunn No. 4	3159 Waucheeta Tr.	MG&E	10835379	NSP 1319	*None*	208 V, 3 Phase	2 Hours	**1 Hour**
Dunn S.D. #3	Bible Camp	2874 Bible Camp Road, McFarland	**Alliant**	306388	CODN 7253	On Site Generator	208 V, 3 Phase	2.5 Hours	**1 Hour**
Dunn S.D. #3	Jordan	4370 Jordan Drive, McFarland	**Alliant**	336508	CODN7255	On Site Generator	208 V, 3 Phase	2.5 Hours	**1.5 Hour**
Dunn S.D. #3	Maple	2684 Maple Drive, McFarland	**Alliant**	349392	CODN7255	On Site Generator	208 V, 3 Phase	2.5 Hours	**1 Hour**
Maple Bluff	Baywood	20 Bayside Drive	MG&E	10165843	JON 421	*None*	240 V, 3 Phase	4.5 Hours	**2 Hours**
Maple Bluff	Boathouse	1321 Farwell Dr / Maple Bluff Park	MG&E	10282267	GRE 451	*None*	240 V, 3 Phase	3.5 Hours	**2 Hours**
Maple Bluff	Jonas	530 Summit Road	MG&E	10552073	MEN 446	*None*	208 V, 3 Phase	3.5 hours	**2 Hours**
Town of Madison	Badger	2200 Badger Lane	MG&E	10899540	NSP 1319	*None*	240 V, 3 Phase	3.5 Hours	**1 Hour**
Town of Madison	Lake Forest	2021 Dickson Place	MG&E	10800316	WGA 1316	*None*	240 V, 3 Phase	2 Hours	**1 Hour**
Town of Madison	Mayflower	2318 South Park Street	MG&E	10381499	WGA 1316	*None*	240 V, 3 Phase	3.5 Hours	**1 Hour**

60 END OF LIST

APPENDIX C
2009 MMSD CAPITAL PROJECTS BUDGET

MADISON METROPOLITAN SEWERAGE DISTRICT

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Jon W. Schellpfeffer
Chief Engineer & Director

Protecting Public Health and the Environment

September 15, 2008

Commissioners and General Public
Madison Metropolitan Sewerage District
Madison, WI 53713

Subject: **2009 Proposed Capital Projects Budget**

Commissioners and General Public:

The proposed 2009 MMSD Capital Projects budget includes expenditures of $15,213,000, revenues of $12,452,000, and a projected 2009 year-end operating reserve of $4,525,000.

Enclosure No. 1 is the Capital Projects Budget Summary for the years 2007 through 2009. For the current year, the summary shows the budgeted amount, the actual expenses through June, and the estimated year-end totals. The anticipated expenditures for 2008 are $4,723,000 less than budgeted.

Included below are specific comments on each of the individual major projects for 2008 and 2009:

1. *Sugar River Plant Site Purchase*: $3 million was included in the budget for 2008 for possible purchase of a future plant site. The District does not anticipate spending this amount in 2008. However, a similar amount is included in the 2009 budget to be used in the event that suitable property becomes available at a reasonable price. The ongoing Master Planning effort should help determine when and if a Sugar River Plant is feasible.
2. *USGS Gauging Station:* The budget includes a one-time capital expense of $12,200 for construction of a new USGS gauging station on the Sugar River. Future support for operation of the gauging station will be expensed from the operating budget on an annual basis. The data gathered from the gauging station will support future efforts related to the discharge of effluent to the Sugar River.
3. *Tenth Addition Design and Construction*: Construction of the Tenth Addition project began in 2003 and is expected to be complete by the end of 2008. District staff anticipates the project's 2008 construction costs will be about $1.3 million, which is $400,000 less than originally budgeted. The difference is due primarily to the timing

of expenditures between years 2007 and 2008, and a decision to drop several project enhancements that were felt to be unnecessary at this time. Total Tenth Addition construction costs will be roughly $32.7 million, almost all of which the District will have financed through a Clean Water Fund Loan. Total Tenth Addition expenditures are anticipated to be about $37.3 million, consisting of the $32.7 million construction cost and planning and design costs of $4.6 million.

4. *Solids Handling Facility Planning and Improvements*: The District began a facility planning effort in 2008 to review its current digestion process, screen various digestion alternatives, and recommend the necessary facilities to produce a reliable and sustainable anaerobic digestion process resulting in Class A biosolids. This facility planning effort will end in early 2009 at a total cost of $282,000. Design and construction of facilities related to the planning effort's recommendations will follow in 2009. $2.0 million dollars has been included in the 2009 budget with an additional $4.0 million in expenditures forecast for 2010. We anticipate that this project will be financed with a Clean Water Fund Loan.

5. *Septage Receiving Improvements*: The District has included $110,000 in the 2009 budget to investigate and design improvements to its septage receiving area. Grease and inorganic materials have caused significant problems with the existing septage receiving operation, the screens, and the overall headworks facility operation. The budget anticipates construction costs of $1,000,000 in 2010 for these improvements. This project will be financed with a Clean Water Fund Loan starting in 2010. Because of its status as a septage receiving area, the loan rate may be 0%.

6. *Process Control System Upgrade*: The District's process control system was installed in 1996 as part of the Ninth Addition. The system, although fully functional, is now obsolete and, minimally, in need of software upgrades. Funds were included in the 2008 budget to begin planning. However, due to Tenth Addition related work, the planning effort was delayed and will now occur in 2009. The extent of the overall project's scope will be defined during the planning process and will help determine the ultimate capital costs for the project. Design will follow the planning process and is anticipated to begin during the latter half of 2009. Construction is expected to start in the latter half of 2010. This project will likely be a Clean Water Fund project. $375,000 is budgeted for 2009.

7. *Long Range Planning*: The 2008 expenditures for the three on-going long-term planning projects listed below total $627,000. Budgeted expenditures in 2009 are $213,000. Specific financial details are provided under each project heading:

   a. *District Long-Range Master Plan*: The main intent of this project is a long-range (50 years) evaluation of the collection system and treatment facility needs of the District. In addition to evaluating the existing facilities for capacity and condition, significant areas being investigated during the planning effort include:

      - Decentralized treatment - should the District construct new satellite treatment plants in other areas of the District, such as above Lake Mendota or in the

Sugar River basin, and if so, when and where should these facilities be constructed?

- Effluent discharge/reuse - can effluent from Nine Springs or other treatment plants be returned to the Yahara River watershed at a location other than Badfish Creek that will provide a net environmental benefit; and if so, what degree of treatment may be required and at what cost?

In 2007, the District hired the consulting team of Malcolm Pirnie and Strand Associates to provide engineering services for this project at an estimated cost of $680,000. Total 2007 expenditures on the planning effort were $207,000. In 2008, we anticipate spending $444,000; and in 2009, we expect to spend about $95,000. The facility planning effort will be completed in early 2009 at a total cost, including District salaries, of roughly $746,000.

b. *Update Collection System Facilities Plan*: The District's engineering staff prepared a Collection System Facilities Plan in 2002. The plan relied heavily upon information provided by the Dane County Regional Planning Commission (DCRPC). One of the main recommendations of the 2002 Plan was to keep the plan up-to-date, and this project is a follow-up effort to abide with that recommendation. Presently, we are in the second year of a $170,000 two-year contract with the Community Analysis and Planning Division (CAPD) and its successor agency, the Capital Area Regional Planning Commission (CARPC), to provide the same information used in preparing the first facilities plan. District staff members have been and will continue to be heavily involved in updating the plan. The CARPC contract should be completed by the end of 2008; however, the District's work on this update will continue throughout 2009. We anticipate total expenditures in 2008 of $178,000 with another $88,000 in 2009.

c. *Treatment Plant Asset Management Plan*: The District has been preparing a comprehensive asset management plan for the treatment plant. The work involves reviewing the capacity and condition of the systems and equipment at the District's Nine Springs Wastewater Treatment Plant. The information from this effort has been beneficial to the Master Planning effort. Going forward, we anticipate that the Plant Asset Management Plan, like the Collection System Facilities Plan, will prove to be a significant resource for scheduling future upgrades and rehabilitation efforts at the plant. Initially, the intent was to complete the plan internally by the end of 2008. Although the preliminary groundwork and reviews have been completed, we found that schedule to be somewhat aggressive. The work is now scheduled for completion by the end of 2009. For the year 2008, we anticipate spending $5,000 with an additional $30,000 budgeted in 2009.

8. *West Interceptor Campus Relief – Phases 5, 6, & 7*: Planning and design were underway for Phases 5, 6, and 7 of the West Interceptor Campus Relief Project. These three phases would have included approximately 15,000 feet of relief sewer

from Walnut Street to Whitney Way. However, recent information obtained from CARPC during the collection system evaluation indicates that capacity improvements are not likely to be needed for the foreseeable future. Therefore, this project has been postponed indefinitely. Beginning in 2009, no additional costs have been budgeted.

9. *West Interceptor Upstream of Pumping Station 5*: The West Interceptor above Pumping Station 5 is scheduled for replacement or rehabilitation in 2010. We anticipate that a portion of this interceptor along Lake Mendota will be abandoned after local sewers are rerouted. The remaining portion will be either replaced or relined. The cost of this project is estimated at roughly $1.1 million. $112,000 has been included in the 2009 budget to begin planning and design.

10. *West Interceptor Replacement at Old University Avenue*: This project was intended to be completed in conjunction with a City of Madison street reconstruction project in 2009. However, the City has delayed their work so our schedule now anticipates project construction in 2010. The project will rehabilitate the West Interceptor along a section of Old University Avenue. In anticipation of construction in 2010, $30,000 has been included in the 2009 budget to begin planning and design. The total project cost is estimated at $565,000.

11. *West Interceptor Extension Replacement*: The West Interceptor Extension, built in 1957 using 24" reinforced concrete pipe, was nearing capacity and some pipe sections had settled below their original grade. Work included replacing approximately 2,800 feet of this sewer from Mendota Avenue to near Century Avenue in Middleton. The project incurred $1.9 million of costs in 2007 and an additional $30,000 in 2008. A $10,000 retainer is expected to be paid to the contractor in 2009. The District financed this project with a Clean Water Fund loan.

12. *Northeast Interceptor – DeForest Monitoring Manhole*: In response to the southerly growth of the Village of DeForest, a new monitoring manhole is being installed on the Northeast Interceptor near its intersection with Windsor Road in the Town of Windsor. This monitoring point will replace the current monitoring point located further north on this interceptor and allow the abandonment of another monitoring point on a village sewer. The total cost for the installation is anticipated to be $32,000, and the work should be completed in 2008.

13. *Northeast Interceptor – Airport Reconstruction and Truax Extension Rehabilitation*: At no cost to the District, Dane County relocated a portion of the Northeast Interceptor in conjunction with other required security and safety related changes at the Dane County Regional Airport. That work was completed last year. The Northeast Interceptor Truax Extension Rehabilitation project consists of relining other portions of this same interceptor. Design began in late 2007 and construction should be completed in 2008. Total project costs will be $1.9 million. The District is financing this project with a Clean Water Fund loan.

14. *Northeast Interceptor Relief Upstream of Pumping Station 10*: The intercepting sewers above Pumping Station 10 are in need of capacity relief. This project includes

over 9,000 feet of replacement sewer at an estimated cost of $10.4 million. District staff began planning and design in late 2007 and design will continue into early 2009. Construction will occur in two phases; Phase 1 in 2009 followed by Phase 2 in 2010. Anticipated expenditures are $175,000 in 2008 and $5.1 million in both years 2009 and 2010. This project will be financed with a Clean Water Fund loan.

15. *South Interceptor at Baird Street*: Televising records from the District's sewer maintenance program identified the South Interceptor at Baird Street as needing improvement. This section of sewer will be reviewed and either replaced or relined. The 2009 budget includes $20,000 for design related to this work. Construction would occur in 2010.

16. *Door Creek Extension at Gaston Road*: In conjunction with anticipated City of Madison development in the Gaston Road area, the District constructed an additional 1,700 feet of sewer as part of its Door Creek Extension. Total project costs are anticipated to be $657,000.

17. *Far East Interceptor – Cottage Grove Extension Lining*: Portions of the FEI – Cottage Grove Extension are exhibiting significant corrosion problems. We have included $20,000 in the 2009 budget to begin design for a relining project to take place in 2010. Overall costs for the project are estimated at $530,000.

Report w/ map - John

18. *Lower Badger Mill Creek Interceptor*: The Lower Badger Mill Creek Interceptor is being constructed in several phases. Phase 2 is presently under construction and should be completed in 2008. This section begins where Phase 1 ended at Edwards Street and ends at Northern Lights Trail in Verona. Procurement of easements for the remaining portions of this interceptor will continue over the next several years. Future interceptor construction will include the City of Madison's segment (north of Midtown Road – anticipated for 2009 construction) and the remaining District section between Northern Lights Road and Midtown Road (anticipated in 2014 or later). The budget includes expenditures of $865,000 for 2008 and $147,000 for 2009.

19. *Pumping Stations 6 and 8 Rehabilitation*: The District constructed Pumping Station 6 between 1948 and 1950. It was part of the original East Interceptor project which also included Pumping Stations 1 and 7. Pumping Station 8 began operation in 1964. Major renovations to both pumping stations are necessary to improve reliability and meet present day standards. The project was bid in June of 2008. Construction is underway and will run through 2010. It is estimated that $2.1 million will be spent in 2008, $3.8 million in 2009, and $1.1 million in 2010. The total project cost is estimated at $7.3 million. The District will finance this project with a Clean Water Fund loan.

20. *Pumping Station 7 – Back-up Power*: In June, an automobile struck a power pole on Bridge Road taking down power lines that caused an extended outage at Pumping Station 7. District staff investigated several options to improve the reliability of the power system at the pumping station including an on-site generator and a third utility feed. The 2009 budget includes funds to install a third utility feed to the site. The

third feed will be routed to the site independently of the existing feeds and will be supplied from a different substation. The cost for this improvement is estimated at $141,000.

21. *Pumping Capacity Improvements at Stations 13 and 14*: Firm capacity improvements were necessary at Pumping Stations 13 and 14. The project included new pumping units at each of the two stations. Those were installed in 2008. Work continues on rebuilding existing pumps at both stations. The project is expected to be completed by the end of this year. Anticipated spending for 2008 is $486,000. Total project costs are estimated at $560,000. This project is being financed with a Clean Water Fund loan.

The anticipated 2008 revenues of $5,514,000 are $638,000 less than budgeted. This difference is comprised of the following items:

- Clean Water Fund loan disbursements of $708,500 for the Tenth Addition project will be $12,000 less than budgeted.
- Clean Water Fund loan disbursements of $517,000 for the West Interceptor Extension Replacement and Pumping Stations 13 and 14 Firm Capacity Improvements will be $237,000 more than budgeted due to the timing of construction for both projects.
- Clean Water Fund loan disbursements of $3,433,000 for the Rehabilitation of Pumping Stations 6 & 8 will be $217,000 less than budgeted.
- Estimated interceptor and treatment plant connection charge revenue of $600,000 is $500,000 less than budgeted.
- Estimated investment interest of $255,000 is $146,000 less than budgeted.

In summary, the 2008 Capital Projects budget showed 2008 expenses exceeding revenues by $6,992,000. The actual 2008 expenses are expected to exceed revenues by $2,906,000. The 2008 year-end reserve balance is anticipated to be $7,286,000, which is $4,275,000 more than budgeted. This difference is due primarily to not purchasing a Sugar River plant site in 2008. Lower costs than anticipated for Phase 2 of the Lower Badger Mill Creek Interceptor and a slower start to the construction of the Pumping Stations 6 and 8 Rehabilitation project than anticipated also contribute to this difference.

The proposed 2009 Capital Projects budget includes expenditures of $15,213,000 and revenues of $12,452,000. The resulting 2009 year-end reserve balance is anticipated to be $4,525,000, which represents a decrease of $2,761,000 from the prior year.

As detailed on Enclosure 1, the five largest expense items for the 2009 budget include the following projects:

- Northeast Interceptor – Relief Upstream of Pumping Station 10 ($5.1 million)
- Rehabilitation of Pumping Stations 6 and 8 ($3.8 million)
- Sugar River Treatment Plant Site Purchase ($3.0 million)

- Solids Handling Improvements ($2.0 million)
- Process Control System Upgrade ($0.4 million)

The anticipated 2009 revenues include $11,382,000 in Clean Water Fund loan proceeds for the Rehabilitation of Pumping Stations 6 and 8, the Northeast Interceptor Truax Extension Rehabilitation, the Solids Handling Improvements, and the Northeast Interceptor Relief Upstream of Pumping Station 10. Other revenues include $800,000 in interceptor and treatment plant connection charges and $270,000 in interest on investments.

Enclosure No. 2 is a 10-year projection of capital project costs for the period 2009-2018. This projection includes completion of the projects currently underway and future projects that were identified as high priorities in MMSD's Collection System Facilities Plan and Plant Asset Management discussions. Those projects currently underway or scheduled to begin in 2009 were discussed earlier.

Additional projects listed by the year detailed planning will begin include the following:

- 2010 Project Starts
    - Telemetry System – Third Upgrade ($150,000)
    - East Monona Interceptor at Fair Oaks ($472,000)
    - Morse Pond Extension ($670,000)
    - Pumping Station 18 ($8,500,000)
    - Pumping Station 18 Force Main ($8,500,000)
- 2011 Project Starts
    - Operations Building HVAC System ($750,000)
- 2012 Project Starts
    - Northeast Interceptor – FEI to PS 18 ($5,260,000)
- 2013 Project Starts
    - Pumping Station 7 Improvements ($1,110,000)
    - Rehabilitation of Pumping Station 11 ($4,260,000)
    - Rehabilitation of Pumping Station 12 ($4,260,000)
- 2014 Project Starts
    - Lower Badger Mill Creek Interceptor to Midtown Road ($5,000,000)
    - Rehabilitation of Pumping Station 15 ($4,270,000)
- 2015 Project Starts
    - Rehabilitation of Pumping Station 13 ($4,260,000)
    - Rehabilitation of Pumping Station 14 ($4,260,000)
    - Undefined Collection System Projects ($6,000,000)
- 2016 Project Starts

  - Southwest Interceptor – Haywood Extension ($670,000)
  - Undefined Collection System Projects ($6,150,000)

- 2017 Project Starts
  - Rehabilitation of Pumping Stations 3, 4, and 9 ($3,270,000)
  - Undefined Collection System Projects ($6,304,000)

- 2018 Project Starts
  - Undefined Collection System Projects ($6,462,000)

Beginning in 2015, Enclosure 2 shows expenditures for undefined collection system projects. Estimated costs start at $6 million and escalate by 2.5 percent each year. These represent longer-range projects that will result from the District's ongoing interceptor inspection and maintenance programs, and projects resulting from requirements for additional capacity. Identification and prioritization of these projects will be addressed in future budgets as specific projects are defined.

The projection does not include a future plant addition to address lower effluent phosphorus levels. The potential budgetary impacts of such an addition are discussed later in this letter.

The total capital projects expenditures from 2009 through 2018 are estimated at $114 million. The total revenues over this period are also estimated at $114 million. The reserve balance at the end of the year 2018 is estimated to be $8.0 million.

We anticipate continued use of the Wisconsin Clean Water Fund loan program to fund larger projects and to ensure adequate capital reserves to address any unforeseen major capital costs. The District to date has borrowed $96 million from this program for the following projects:

- Modifications to Pumping Station No. 7 ($1.9 million).
- Eighth Addition to Nine Springs ($19.9 million).
- Replacement of Pumping Station No. 5 ($1.2 million).
- Verona Force Main and Pumping Station ($2.7 million).
- Ninth Addition to Nine Springs ($14.9 million).
- Badger Mill Creek Effluent Return Project ($4.7 million).
- Pumping Station No. 2 Force Main Replacement ($3.8 million).
- Rehabilitation of PS's 1, 2, and 10 ($8.0 million).
- Tenth Addition to Nine Springs ($34.7 million to date, $35.4 million total anticipated).
- Effluent Equalization/Aeration Tanks 1-6 Rehab. ($1.7 million).
- WI Ext. Replacement & PS 13/14 Firm Cap. Improvements ($2.4 million to date and 2.9 million total anticipated).
- Rehabilitation of PS 6 and 8 and NEI - Truax Extension Liner (Expect initial loan draw in October, $9.0 million total anticipated)

The following future projects are also expected to be funded with Clean Water Fund loans:

- Solids Handling Improvements ($6.0 million from 2009 – 2011)
- NEI – Upstream of PS 10 – 2 phases ($10.4 million from 2009 – 2010)
- Nine Springs Process Control System Upgrades ($2.3 million in 2010 – 2011)
- Septage Facilities Improvements ($1.1 million in 2010 – 2011)
- West Interceptor Upstream of PS 5 ($1.1 million in 2010)
- FEI – Cottage Grove Extension Liner ($0.5 million in 2010)
- Pumping Station No. 18 and Force Main ($17.0 million in 2012 – 2014)
- NEI – FEI to SEI Junction ($5.3 million in 2013 to 2014)
- 2014 Collection System Projects ($9.6 million in 2014 – 2015)
- Later Collection System Projects ($34.2 million in 2015 – 2018)

Enclosure No. 3 shows the effect of the Clean Water Fund loans on the amount of debt service that will be collected and paid from service charge revenues during the period 2009 through 2018. The upper table on Enclosure No. 3 shows the principal and interest payments required in each year for the various Clean Water Fund loans. The lower table shows the amount of funds that must be collected through service charge rates each year to retire the debt. The amount of debt service payments included in each year's service charge rate structure include that year's second interest payment and the following year's first interest payment and annual principal payment. This earlier collection is necessary to comply with the adopted bond ordinances that require inclusion of the debt service on the tax rolls if sufficient funds are not available to cover the debt service for the following year at the time the tax roll is prepared.

In 2004 the District transferred $1,515,000 from the General Fund to the Debt Service Fund above the amount necessary to satisfy the 2004 debt service requirements. Those extra funds have been used, beginning in 2005, to limit the annual increase in the future debt service expenses raised through service charges. In 2005 and 2006 these funds were used to limit the increase to 2.0 percent. Due to projected increases in the cost of future collection system and treatment plant projects, beginning in 2007 the annual increase was raised to 3.4 percent.

Enclosure No. 4 is a chart showing the annual amounts collected through service charges and used to fund capital projects or pay debt service since 1997. It also shows the amount of funds required to be deposited in a debt service reserve each year to satisfy the bond ordinances and the projected annual amounts to be collected through service charges over the next twenty years. For the next twelve years, the regular annual increase of 3.4 percent for this portion of the budget will generate the necessary amount of funds to pay for the annual debt service expenses projected in this budget. This will help to limit the overall impact on charges due to higher cost increases for certain operating expenses and will result in a smoother annual increase in service charges over that time. Beginning in 2021, the amount of funds collected through services charges to fund debt service will exceed the actual debt service requirements. This will increase the debt service reserves. Based on historical interest and inflation rates, these reserves will be sufficient to fund future collection system projects and future treatment plant expansions of $50 million in 2021 ($34 million in 2008 dollars).

The Wisconsin Department of Natural Resources is currently developing new phosphorus water quality criteria and a TMDL (total maximum daily load) for phosphorus in the Rock River watershed. It is unlikely that the District's new WPDES permit issued next year will contain requirements for lower effluent phosphorus concentrations. However, the following permit, issued in 2014, may contain such limits. How would this impact the District's long-term financial plan? Assuming the new limits would require the addition of chemical polishing and effluent filtration processes, the capital cost at that time could be $100 million. This would increase the District's annual debt service requirements by $7 million and result in a 25 percent service charge increase. Enclosure 5 shows the impact on the debt service requirements. When the project was complete in 2018, the increase for a typical household would be about $3 per month, from $13 to $16, for District-provided services.

A large increase in one year would be reasonably easy to justify in this scenario. With the length of time it would take to plan, design, and construct such an addition, modifications to the current financial plan could be made to provide a somewhat more gradual increase. Since the future requirements are unknown at this time, it is too soon to modify the current financial plan to address them.

It is not possible to anticipate all projects that may become necessary in the future, but by maintaining adequate capital project reserves, MMSD should be able to meet the District's long-term construction project needs with reasonable increases for our users.

Respectfully submitted,

Michael E. Simon
Assistant Chief Engineer &
Director of Planning

Jon W. Schellpfeffer
Chief Engineer and Director

Enclosures: No. 1, No. 2, No. 3, No. 4, No. 5

CAPITAL PROJECTS BUDGET SUMMARY

EXPENDITURES

Acct #	Project	2007 Actual	2008 Budgeted Amount	Actual 2008 Expenses Thru June	2008 Estimated Year-End	2009 Budgeted Amount	2009 Budgeted Increase From 2008 Budgeted Amount
82240	Sugar River Plant Site Purchase	$ 0	$ 3,003,188	$ 0	$ 0	$ 3,001,522	-0.06%
44021	Sugar River USGS Gauging Station	0	0	0	0	12,200	NMF
82251	Tenth Addition Construction	2,169,617	1,698,124	836,168	1,300,000	0	-100.00%
82252	Effluent Equalization Project	9,073	0	0	0	0	NMF
82253	Space Needs Study/Personnel Facilities Expansion	328,717	0	0	0	0	NMF
82254	Aeration Tanks 1-6 Rehabilitiation	5,017	0	0	0	0	NMF
82255	Solids Handling Improvements	0	0	0	2,000	2,000,237	NMF
82256	NSWTP Stormwater Management	0	2,394	0	0	0	-100.00%
82256	Septage Receiving Improvements	0	0	0	0	109,781	NMF
82258	East Primary Influent Line Rehab	19,780	0	0	0	0	NMF
82259	Process Control System Upgrade	0	173,939	0	10,000	375,199	115.71%
44000	District Long Range Facility Plan	207,178	513,183	156,640	444,000	94,708	-81.55%
44000	Update Collection System Facilities Plan	75,794	127,204	49,365	178,000	88,000	-30.82%
44000	Treatment Plant Asset Management Plan	0	17,722	246	5,000	30,000	69.28%
82858	Solids Handling Facilities Plan	0	0	21,587	225,000	56,816	NMF
83150	Southwest Interceptor Relief - North and South Legs	454,948	0	0	0	0	NMF
83268	West Interceptor Campus Relief - Phases 5, 6, & 7	41,018	219,354	5,718	39,000	0	-100.00%
83270	West Interceptor upstream of PS5	0	0	0	0	112,052	NMF
83272	West Interceptor Replacement at Old University Avenue	0	22,950	0	4,000	29,929	30.41%
83273	West Interceptor Replacement at Park Street	39,778	0	0	0	0	NMF
83550	West Interceptor Extension Replacement	1,891,074	30,000	30,308	30,308	10,000	-66.67%
83862	NSVI - Morse Pond Extension	0	4,910	0	0	0	-100.00%
83950	NEI - DeForest Monitoirng MH	848	27,321	6,433	32,000	0	-100.00%
83975	NEI - Pflaum Road Relief Sewer	15,332	0	0	0	0	NMF
83976	NEI - Airport Replacement Sewer	898	2,574	129	129	0	-100.00%
83977	NEI - Truax Extension Rehabilitation	4,607	1,281,869	22,338	1,875,000	15,537	-98.79%
83978	NEI - Relief Upstream of Pumping Station 10	957	189,498	5,588	175,000	5,099,951	2591.30%
84050	South Interceptor - Baird Street Replacement	0	0	0	0	20,080	NMF
84156	FEI - Door Creek Extension at Gaston Road	14,570	464,010	48,476	632,000	10,000	-97.84%
84157	Far East Int - Cottage Grove Extension Lining	0	0	0	0	19,601	NMF
84350	Lower Badger Mill Creek Interceptor	689,281	1,754,276	28,440	865,000	147,543	-91.59%
85150	Rehabilitation of PSs 1, 2, and 10	6,219	0	0	0	0	NMF
85151	Pumping Station No. 1 Force Main Air Release MH	182	0	0	0	0	NMF
85252	Crosstown Force Main Replacement - Phase 2	91	0	0	0	0	NMF
85651	P.S. 6 & 8 Rehabilitation	190,306	3,397,613	195,301	2,118,000	3,835,361	12.88%
44020	PS 7 Back-up Power	0	0	0	0	141,516	NMF
86150	Pumping Capacity Improvements at PSs 11-12-13-14	73,137	213,617	235,459	486,000	3,317	-98.45%
	TOTAL EXPENDITURES	$ 6,238,422	$ 13,143,746	$ 1,642,195	$ 8,420,437	$ 15,213,350	15.75%

REVENUES

Revenue Source	2007 Actual	2008 Budgeted Amount	Actual 2008 Revenues Thru June	2008 Estimated Year-End	2009 Budgeted Amount	2009 Budgeted Increase From 2008 Budgeted Amount
CWF Loan - Tenth Addition to Nine Springs	$ 1,771,437	$ 720,639	$ 0	$ 708,545	$ 0	-100.00%
CWF Loan - Effluent Equalization	68,878	0	0	0	0	NMF
CWF Loan - West Int Ext Replacement and PSs 13 and 14	2,074,842	280,000	84,827	517,400	0	-100.00%
CWF Loan - Rehab of PSs 6 and 8 and NEI Truax Ext Liner	0	3,650,000	0	3,433,000	4,407,000	20.74%
CWF Loan - Solids Handling Improvements	0	0	0	0	1,700,000	NMF
CWF Loan - NEI Replacement Upstream of PS 10 - Phase 1	0	0	0	0	5,275,000	NMF
Interceptor and Treatment Plant Connection Charges	889,141	1,100,000	177,331	600,000	800,000	-27.27%
Interest on Investments	538,216	401,000	146,859	255,000	270,000	-32.67%
TOTAL REVENUES	$5,342,513	$6,151,639	$409,016	$5,513,945	$12,452,000	102.42%

RESERVE BALANCE

CAPITAL PROJECTS RESERVES	2007 Actual	2008 Budgeted Amount	Actual 2008 Balance Thru June	2008 Estimated Year-End	2009 Budgeted Amount	2009 Budgeted Increase From 2008 Budgeted Amount
Beginning Reserve Balance	$11,088,101	$10,003,000	$10,192,192	$10,192,192	$7,286,000	-27.16%
Ending Reserve Balance	$10,192,192	$3,011,000	$8,959,013	$7,286,000	$4,525,000	50.28%

Enclosure No. 2

Madison Metropolitan Sewerage District

CAPITAL PROJECTS BUDGET

2009 - 2018

EXPENDITURES

Acct #	Project	2009	2010	2011	2012	2013	2014	2015	2016	2017	2018
822-00-40	Sugar River Plant Site Purchase	3,001,522									
440-00-21	Sugar River USGS Gauging Station	12,200									
822-00-55	Solids Handling Improvements	2,000,237	4,000,000								
822-00-56	Septage Receiving Improvements	109,781	1,000,000								
822-00-57	Eleventh Addition										
822-00-59	Process Control System Upgrade	375,199	725,000	1,200,000							
822-00-65	Operations Building HVAC Rehab			750,000							
828-55/440	District Long Range Master Facility Plan	94,708									
828-56/440	Update Collection System Facilities Plan	88,000									
828-57/440	Treatment Plant Asset Management Plan	30,000									
828-00-58	Solids Handling Facility Planning	56,816									
830-00-54	Telemetry System - Third Upgrade		100,000	50,000							
831-00-51	Southwest Interceptor - Haywood Extension Rehab or Replacement								60,000	600,000	10,000
832-00-70	West Interceptor - Upstream of Pumping Station No. 5	112,052	1,100,000	10,000							
832-00-72	West Interceptor Replacement at Old University Avenue	29,929	525,000	10,000							
835-00-50	West Interceptor Extension Replacement	10,000									
836-00-73	East Monona Interceptor at Fair Oaks u/s of Starkweather Creek		42,000	420,000	10,000						
838-00-62	NSVI - Morse Pond Extension		60,000	600,000	10,000						
839-00-77	NEI - Truax Extension Rehabilitation	15,537									
839-00-78	NEI - Relief Upstream of Pumping Station No. 10	5,099,951	5,100,000	10,000							
839-00-79	NEI - Far East Int. to Southeast Int. Junction				250,000	5,000,000	10,000				
840-00-50	South Interceptor - Baird Street Replacement	20,080	300,000	10,000							
841-00-56	Far East Int - Door Creek Extension at Gaston Road	10,000									
841-00-57	Far East Int - Cottage Grove Extension Lining	19,601	500,000	10,000							
843-00-50	Lower Badger Mill Creek Interceptor Project	147,543	100,000	120,000			5,000,000	10,000			
853-00-50	Pumping Stations 3, 4, & 9 Revisions									250,000	3,000,000
856-00-51	Pumping Stations 6 & 8 Rehabilitation	3,835,361	1,147,000	20,000							
440-00-20	PS 7 Back-up Power	141,516									
856-00-52	Gravity or Force Main Tie between PS6 and PS10				0	0	0				
857-00-70	PS7 - Improvements (in conjunction with PS18 construction)					100,000	1,000,000	10,000			
861-00-52	Pumping Stations 11 & 12 Rehabilitation					500,000	4,000,000	4,000,000	20,000		
863-00-50	Pumping Stations 13 & 14 Rehabilitation	3,317						500,000	4,000,000	4,000,000	20,000
864-00-50	Pumping Station 15 Rehabilitation						250,000	2,500,000	1,500,000	20,000	
867-00-50	Pumping Station 17 Upgrade (Completed in conjunction with LBMCI or SRTP)					200,000	2,000,000				
868-00-51	P.S. No. 18 Construction		287,000	250,000	4,000,000	4,000,000	20,000				
868-00-52	P.S. No. 18 Force Main Construction		261,000	250,000	4,000,000	4,000,000					
	Future Collection System Projects							6,000,000	6,150,000	6,304,000	6,462,000
	Total	$ 15,213,350	$ 15,247,000	$ 3,710,000	$ 8,270,000	$ 13,800,000	$ 12,280,000	$ 13,020,000	$ 11,730,000	$ 11,174,000	$ 9,492,000

Revenues

Revenue Source	2009	2010	2011	2012	2013	2014	2015	2016	2017	2018
CWF Loan - Rehab of PSs 6 and 8 and NEI Truax Ext Liner	4,407,000	1,320,000								
CWF Loan - Solids Handling Improvements	1,700,000	4,220,000	80,000							
CWF Loan - NEI Replacement Upstream of PS 10 - Phase 1	5,275,000									
CWF Loan - Nine Springs Process Control System Upgrades		1,100,000	1,225,000							
CWF Loan - Septage Facilities Improvements		950,000	50,000							
CWF Loan - NEI Replacement Upstream of PS 10 - Phase 2		5,100,000								
CWF Loan - West Interceptor Upstream of PS 5		1,100,000								
CWF Loan - FEI Cottage Grove Ext Liner		500,000								
CWF Loan - P.S. No. 18 and PS 18 Force Main				7,800,000	9,050,000	150,000				
CWF Loan - Northeast Interceptor FEI to SEI Replacement Project					5,150,000	100,000				
CWF Loan - Nine Springs Maintenance Facilities					0	0				
CWF Loan - 2014 Collection System Projects						4,160,000	5,390,000	50,000		
CWF Loans for Future Collection System Projects							5,300,000	11,430,000	10,520,000	6,910,000
Interceptor Connection Charges	800,000	1,000,000	1,200,000	2,000,000	2,100,000	2,200,000	2,300,000	2,400,000	2,500,000	2,600,000
Interest on Investments	270,000	180,000	170,000	180,000	270,000	220,000	110,000	160,000	240,000	290,000
Total	$ 12,452,000	$ 15,470,000	$ 2,725,000	$ 9,980,000	$ 16,570,000	$ 6,830,000	$ 13,100,000	$ 14,040,000	$ 13,260,000	$ 9,800,000

Construction Account Cash Flow Summary

Constructioin Account Activity	2009	2010	2011	2012	2013	2014	2015	2016	2017	2018
Beginning of Year Reserve Balance	$ 7,286,000	$ 4,520,000	$ 4,740,000	$ 3,760,000	$ 5,470,000	$ 8,240,000	$ 2,790,000	$ 2,870,000	$ 5,180,000	$ 7,270,000
+ Revenues	12,452,000	15,470,000	2,725,000	9,980,000	16,570,000	6,830,000	13,100,000	14,040,000	13,260,000	9,800,000
- Expenditures	15,213,350	15,247,000	3,710,000	8,270,000	13,800,000	12,280,000	13,020,000	11,730,000	11,174,000	9,492,000
End of Year Cash Reserve Balance	$ 4,520,000	$ 4,740,000	$ 3,760,000	$ 5,470,000	$ 8,240,000	$ 2,790,000	$ 2,870,000	$ 5,180,000	$ 7,270,000	$ 7,580,000

JWS, 9/8/2008, 2009 Capital Budget .xls, 20-YEAR - 2009

Madison Metropolitan Sewerage District

Enclosure No. 3

CAPITAL PROJECTS BUDGET

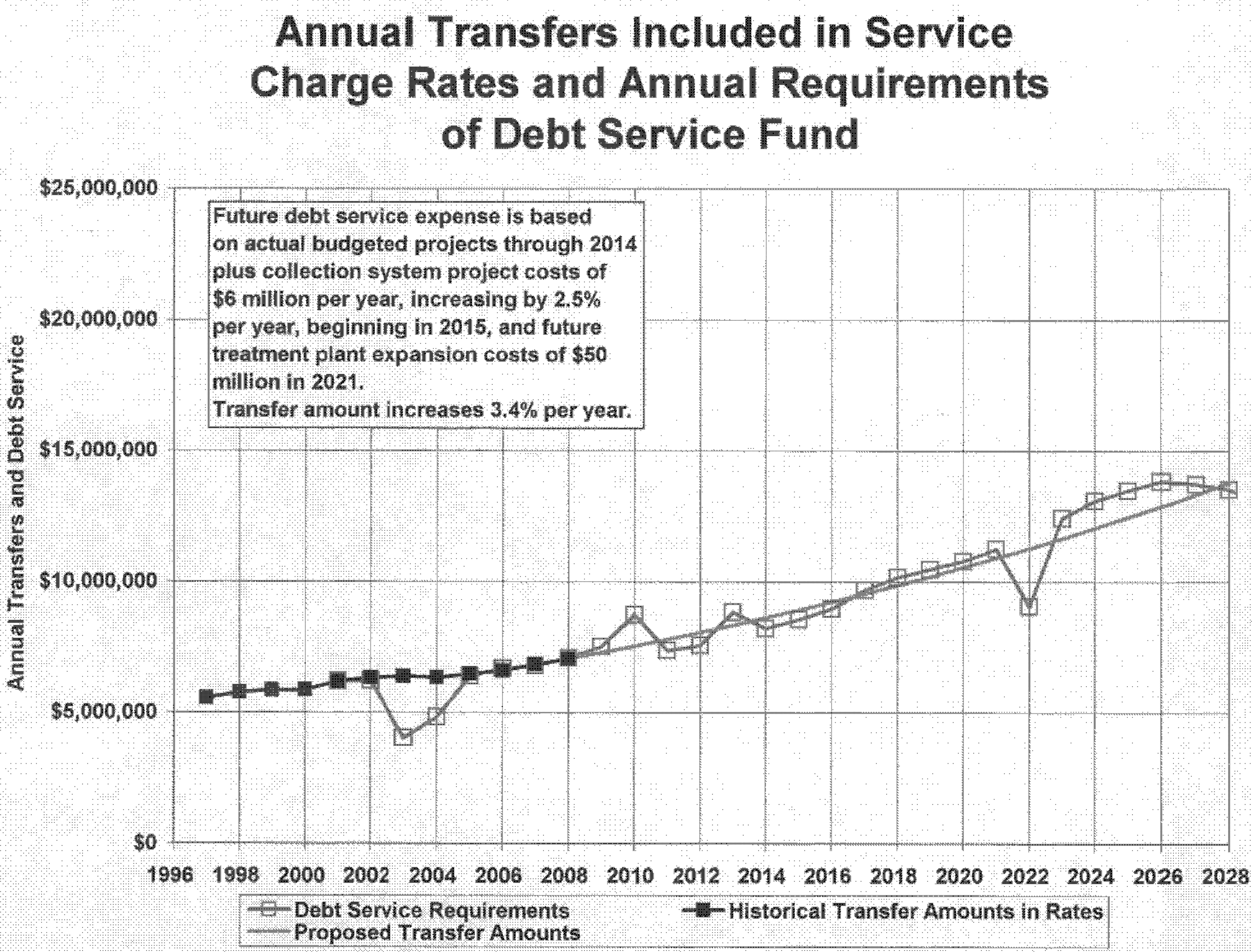
2009 - 2018

Debt Service Payments

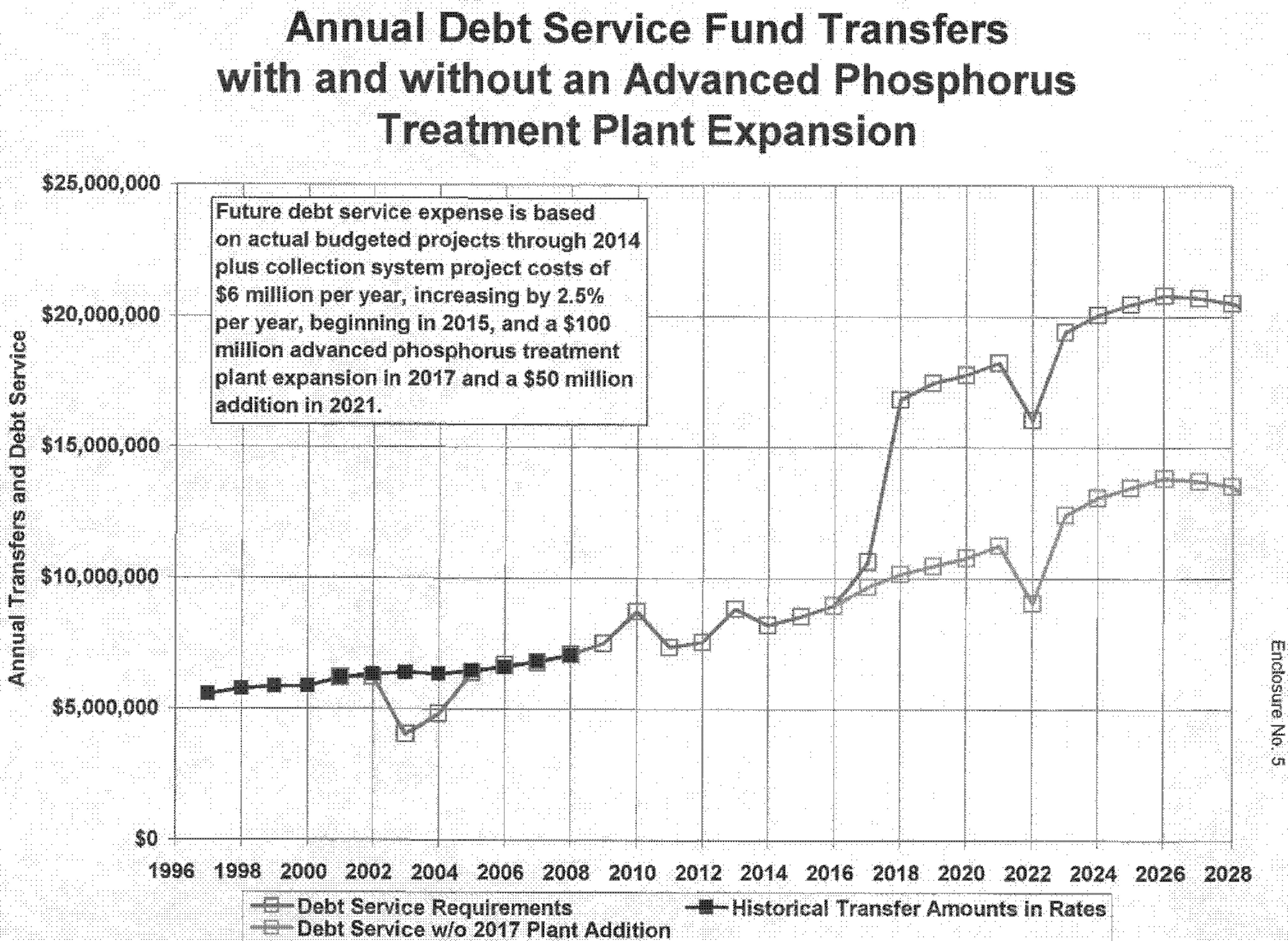
Debt Instrument	1997	1998	1999	2000	2001	2002	2003	2004	2005	2006	2007	2008	2009	2010	2011	2012	2013	2014	2015	2016	2017	2018
CWF Loan - Pumping Station No. 7	141,333	141,272	141,208	141,142	141,073	141,002	140,928	140,851	140,771	140,688	140,602	140,512	140,418	140,323	140,222							
CWF Loan - Eighth Addition to NSWTP	1,485,397	1,484,786	1,484,152	1,483,493	1,482,809	1,482,099	1,481,361	1,480,594	1,479,798	1,478,972	1,478,113	1,477,221	1,476,295	1,475,333	1,474,333	1,473,296						
CWF Loan - Pumping Station No. 5	84,708	84,903	84,876	84,849	84,821	84,792	84,763	84,732	84,700	84,667	84,633	84,598	84,562	84,524	84,486	84,446	84,405	84,362				
CWF Loan - Verona Force Main and Pumping Station	180,224	181,812	190,899	190,837	190,774	190,708	190,641	190,571	190,498	190,424	190,346	190,267	190,184	190,099	190,011	189,920	189,826	189,729	189,628			
CWF Loan - Ninth Addition to NSWTP	351,475	1,032,000	1,109,954	1,105,959	1,106,473	1,106,102	1,105,719	1,105,323	1,104,914	1,104,491	1,104,055	1,103,604	1,103,139	1,102,658	1,102,162	1,101,649	1,101,119	1,100,572	1,100,007			
CWF Loan - Badger Mill Creek Effluent Return	47,000	141,500	344,500	327,525	327,431	327,335	327,235	327,132	327,026	326,917	326,804	326,688	326,568	326,445	326,317	326,185	326,050	325,910	325,765	325,616	325,463	
CWF Loan - PS #2 Force Main Replacement - Phase 1					41,589	126,032	125,968	125,931	125,893	125,853	125,813	125,771	125,728	125,683	125,637	125,589	125,540	125,490	125,437	125,383	125,328	125,270
CWF Loan - PS #2 Force Main Replacement - Phase 2			0	0	20,140	139,342	139,686	139,646	139,606	139,563	139,520	139,475	139,428	139,380	139,331	139,280	139,227	139,172	139,116	139,058	138,999	138,937
CWF Loan - Tenth Addition to NSWTP	0						27,610	344,028	737,264	2,268,223	2,341,274	2,470,228	2,527,722	2,530,088	2,529,409	2,528,712	2,527,995	2,527,257	2,526,500	2,525,721	2,524,920	2,524,097
CWF Loan - PS 1, PS 2, and PS 10 Rehabs							6,247	108,989	404,962	203,991	524,204	524,073	523,938	523,799	523,656	523,509	523,358	523,203	523,043	522,879	522,711	522,537
CWF Loan - PS 1, PS 2, and PS 10 Rehabs Amendment										11,101	17,973	17,969	17,966	17,962	17,959	17,955	17,951	17,947	17,943	17,939	17,935	17,931
CWF Loan - Effluent Equalization											104,655	108,928	108,908	108,888	108,868	108,846	108,825	108,803	108,780	108,757	108,733	108,709
CWF Loan - WI Ext Replacement and PSs 13 and 14											0	49,687	172,512	172,511	172,475	172,438	172,400	172,361	172,321	172,280	172,238	172,195
CWF Loan - PS 6 and PS 8 Rehabs and NEI Truax Ext Liner												0	132,571	217,218	637,795	637,698	637,560	637,417	637,271	637,121	636,968	636,810
CWF Loan - Solids Handling Improvements													1,906	80,590	399,219	400,174	400,090	400,002	399,913	399,822	399,728	399,631
CWF Loan - NEI Replacement Upstream of PS 10 - Phase 1													12,055	334,238	338,228	338,158	338,086	338,013	337,937	337,860	337,780	337,699
CWF Loan - Nine Springs Process Control System Upgrade														2,784	95,008	109,382	153,288	153,255	153,222	153,188	153,153	153,117
CWF Loan - NEI Replacement Upstream of PS 10 - Phase 2														11,911	323,884	327,007	326,939	326,870	326,799	326,726	326,651	326,574
CWF Loan - 2010 Septage Facilities Improvements														2,602	63,354	64,119	64,106	64,092	64,078	64,064	64,049	64,034
CWF Loan - West Interceptor Upstream of PS 5														2,935	69,907	70,531	70,516	70,501	70,486	70,470	70,454	70,438
CWF Loan - FEI Cottage Grove Ext Liner														586	31,571	32,060	32,053	32,046	32,039	32,032	32,025	32,017
CWF Loan - PS 18 and Force Main																26,433	317,688	1,171,417	1,176,379	1,176,065	1,175,742	1,175,409
CWF Loan - NE Int, FEI to SEI, Replacement Project																	14,504	343,423	349,864	349,773	349,680	349,584
CWF Loan - 2014 Collection System Projects																	14,235	180,516	663,182	664,308	664,131	663,948
CWF Loan - Future Collection System Projects																		18,525	428,033	1,222,448	1,899,578	2,332,317
Total (Rounded)	$ 4,615,000	$ 5,404,000	$ 5,742,000	$ 5,729,000	$ 5,809,000	$ 6,063,000	$ 6,105,000	$ 4,048,000	$ 4,735,000	$ 6,074,890	$ 6,577,992	$ 6,759,000	$ 7,084,000	$ 7,591,000	$ 8,894,000	$ 8,797,000	$ 7,686,000	$ 9,051,000	$ 9,868,000	$ 9,372,000	$ 10,046,000	$ 10,151,000
Percent Change from Previous Year	11.1%	17.1%	6.3%	-0.2%	1.4%	4.4%	0.7%	-33.7%	17.0%	28.3%	8.3%	2.8%	4.8%	7.2%	17.2%	-1.1%	-12.6%	17.8%	9.0%	-5.0%	7.2%	1.0%

Debt Service Reserves Included in Service Charge Rates

Debt Instrument	1997	1998	1999	2000	2001	2002	2003	2004	2005	2006	2007	2008	2009	2010	2011	2012	2013	2014	2015	2016	2017	2018
CWF Loan - Pumping Station No. 7	142,902	142,902	142,902	142,902	142,902	142,902	142,902	142,902	142,902	142,902	142,902	142,902	142,902	902								
CWF Loan - Eighth Addition to NSWTP	1,501,206	1,501,206	1,501,206	1,501,206	1,501,206	1,501,206	1,501,206	1,501,206	1,501,206	1,501,206	1,501,206	1,501,206	1,501,206	1,501,206	206							
CWF Loan - Pumping Station No. 5	85,642	85,711	85,711	85,711	85,711	85,711	85,711	85,711	85,711	85,711	85,711	85,711	85,711	85,711	85,711	85,711	711					
CWF Loan - Verona Force Main and Pumping Station	197,000	193,700	192,739	192,739	192,739	192,739	192,739	192,739	192,739	192,739	192,739	192,739	192,739	192,739	192,739	192,739	192,739	739				
CWF Loan - Ninth Addition to NSWTP	1,124,000	1,041,000	1,130,000	1,115,000	1,117,777	1,117,777	1,117,777	1,117,777	1,117,777	1,117,777	1,117,777	1,117,777	1,117,777	1,117,777	1,117,777	1,117,777	1,117,777	777				
CWF Loan - Badger Mill Creek Effluent Return	113,000	353,000	328,000	330,501	330,501	330,501	330,501	330,501	330,501	330,501	330,501	330,501	330,501	330,501	330,501	330,501	330,501	330,501	330,501	501		
CWF Loan - PS #2 Force Main Replacement - Phase 1					140,102	127,122	127,122	127,122	127,122	127,122	127,122	127,122	127,122	127,122	127,122	127,122	127,122	127,122	127,122	127,122	127,122	127,122
CWF Loan - PS #2 Force Main Replacement - Phase 2			0	0	127,735	140,926	140,926	140,926	140,926	140,926	140,926	140,926	140,926	140,926	140,926	140,926	140,926	140,926	140,926	140,926	140,926	140,926
CWF Loan - Tenth Addition to NSWTP							317,000	703,510	2,227,930	2,408,367	2,351,318	2,551,389	2,554,361	2,554,361	2,554,361	2,554,361	2,554,361	2,554,361	2,554,361	2,554,361	2,554,361	2,554,361
CWF Loan - PS 1, PS 2, and PS 10 Rehabs							79,000	489,700	524,877	528,855	528,855	528,855	528,855	528,855	528,855	528,855	528,855	528,855	528,855	528,855	528,855	528,855
CWF Loan - PS 1, PS 2, and PS 10 Rehabs Amendment										24,875	18,112	18,112	18,112	18,112	18,112	18,112	18,112	18,112	18,112	18,112	18,112	18,112
CWF Loan - Effluent Equalization										84,919	109,815	109,760	109,760	109,760	109,760	109,760	109,760	109,760	109,760	109,760	109,760	109,760
CWF Loan - WI Ext Replacement and PSs 13 and 14											133,935	188,457	173,922	173,922	173,922	173,922	173,922	173,922	173,922	173,922	173,922	173,922
CWF Loan - PS 6 and PS 8 Rehabs and NEI Truax Ext Liner												59,756	186,313	639,319	643,078	643,078	643,078	643,078	643,078	643,078	643,078	643,078
CWF Loan - Solids Handling Improvements													26,510	381,013	403,465	403,465	403,465	403,465	403,465	403,465	403,465	403,465
CWF Loan - NEI Replacement Upstream of PS 10 - Phase 1													280,888	340,938	340,938	340,938	340,938	340,938	340,938	340,938	340,938	340,938
CWF Loan - Nine Springs Process Control System Upgrade														78,573	100,181	154,548	154,548	154,548	154,548	154,548	154,548	154,548
CWF Loan - NEI Replacement Upstream of PS 10 - Phase 2														272,560	329,628	329,628	329,628	329,628	329,628	329,628	329,628	329,628
CWF Loan - 2010 Septage Facilities Improvements														53,557	64,633	64,633	64,633	64,633	64,633	64,633	64,633	64,633
CWF Loan - West Interceptor Upstream of PS 5														59,203	71,096	71,096	71,096	71,096	71,096	71,096	71,096	71,096
CWF Loan - FEI Cottage Grove Ext Liner														25,958	32,316	32,316	32,316	32,316	32,316	32,316	32,316	32,316
CWF Loan - PS 18 and Force Main																146,575	1,124,115	1,186,836	1,186,836	1,186,836	1,186,836	1,186,836
CWF Loan - NE Int, FEI to SEI, Replacement Project																	282,107	352,882	352,882	352,882	352,882	352,882
CWF Loan - 2014 Collection System Projects																	81,358	638,307	670,213	670,213	670,213	670,213
CWF Loan - Future Collection System Projects																			322,394	1,056,143	1,773,409	2,256,174
CWF Loan - Future Treatment Plant Construction																				0	0	0
General Fund Additional Transfers to (from) Sinking Fund								1,515,000	82,309	(82,420)	47,079	(35,216)	(217,607)	(1,184,817)	439,509	504,137	(477,482)	425,501	366,079	265,668	(137,447)	(295,898)
Total	$ 5,561,012	$ 5,771,607	$ 5,849,770	$ 5,864,059	$ 6,195,873	$ 7,413,384	$ 6,384,884	$ 6,347,094	$ 6,474,000	$ 6,603,480	$ 6,828,000	$ 7,060,000	$ 7,300,000	$ 7,548,200	$ 7,804,838	$ 8,070,203	$ 8,344,590	$ 8,628,306	$ 8,921,668	$ 9,225,005	$ 9,538,655	$ 9,862,969
Percent Change from Previous Year	23.4%	3.8%	1.4%	0.2%	5.7%	19.7%	-13.9%	-0.6%	2.0%	2.0%	3.4%	3.4%	3.40%	3.40%	3.40%	3.40%	3.41%	3.39%	3.41%	3.40%	3.40%	3.40%



Enclosure No. 4



Annual Debt Service Fund Transfers
with and without an Advanced Phosphorus
Treatment Plant Expansion
Future debt service expense is based on actual budgeted projects through 2014 plus collection system project costs of $6 million per year, increasing by 2.5% per year, beginning in 2015, and a $100 million advanced phosphorus treatment plant expansion in 2017 and a $50 million addition in 2021.
Annual Transfers and Debt Service
$25,000,000
$20,000,000
$15,000,000
$10,000,000
$5,000,000
$0
1996
1998
2000
2002
2004
2006
2008
2010
2012
2014
2016
2018
2020
2022
2024
2026
2028
Debt Service Requirements
Debt Service w/o 2017 Plant Addition
Historical Transfer Amounts in Rates
Enclosure No. 5

Technical Memorandum No.4

Date: July 14, 2008

To: Madison Metropolitan Sewerage District

From: Steve McGowan, P.E., BCEE
Project Manager, Malcolm Pirnie, Inc.
Eric Wang, P.E.
Project Engineer, Malcolm Pirnie Inc

Subject: 50-Year Master Plan
TM-4: Planning Variables (Final)

Project No.: MMSD No. 8425001
MPI No. 6100-001

1. Purpose

This memorandum documents a workshop held with the Technical Advisory Committee (TAC) and key MMSD staff to identify and discuss major planning variables that will govern or impact MMSD's available options for continuing to provide high quality services over the 50-year Master Planning period.

2. Workshop Description

The workshop was held on May 19, 2008 at MMSD headquarters. The workshop agenda, attendance sheet, and handout are included as Appendices A, B and C respectively. The workshop was designed to identify the major planning variables that will govern or impact MMSD's ability to continue to provide high quality services over the planning period. These services include: wastewater conveyance; wastewater treatment and the return of clean water to the environment; and biosolids management. The planning variables and driving forces identified in this workshop will serve as the basis for the subsequent planning process.

The workshop participants represented a broad range of experiences and perspectives, including:

- Staff from MMSD management and operations.
- Area planning staff from the Wisconsin Department of Natural Resources (WDNR).

- Water resource experts from University of Wisconsin – Madison.
- Experts from governmental agencies of the State of Wisconsin and Dane County.
- Consultant staff from Malcolm Pirnie and Strand & Associates, Inc.

The workshop began with a presentation of the preliminary list of planning variables identified by the consultants and MMSD staff. A discussion was then held to identify additional planning variables and their impacts on the master planning process.

3. Identified Planning Variables

The following planning variables were identified and discussed:

<u>Location of Treatment Plants</u>

Based upon a preliminary analysis of the current treatment capacities and anticipated future loadings, the existing wastewater treatment facilities at the Nine Springs WWTP will not have adequate capacity for the 50-year planning period. Capacity could be addressed by expanding the current plant, and or constructing satellite treatment facilities. The following issues pertaining to expanded and new facilities were discussed:

- New satellite treatment facilities should be close to population centers.
- Proximity to wetlands for the use of effluent polishing could be desirable.
- Effluent reuse could be a major initiative in the future. An example of this could be the use of effluent as cooling water at a power plant. In this case, treatment facilities could be strategically placed near the intended reuse applications.
- Is there enough land available for construction of the new facilities at the Nine Springs plant site? The land requirements for new facilities will be highly dependent on the level of treatment required to comply with water quality regulations and resulting effluent limitations in place at the time of construction.
- Future wastewater treatment systems may consist of many smaller, decentralized treatment facilities that operate on a local or neighborhood level. These systems have the potential to save on conveyance system infrastructure costs because they use smaller pipes. They may also provide better opportunities for local water reuse applications such as irrigation.
- With the TMDL being developed for the Rock River, there may be opportunities for effluent credit trading with local farmers.

- Water quantity and hydrology within the watershed will likely be a major issue in the future. Following its use, all groundwater used in the District's service area is currently pumped to a single central location for treatment. The treated wastewater effluent is then diverted around the Madison lakes by pumping most of it to Badfish Creek. Future treatment systems could provide a higher level of treatment, allowing the cleaned water to be used to provide a sustainable water flow to the lakes, rivers and aquifers in the upper part of the Yahara River watershed.

Biosolids Management

The District currently uses anaerobic digestion to produce biosolids that are recycled to agricultural land. An initiative is underway to add flexibility by developing a soil like product that can be used in non-agricultural settings. Accepting solid waste such as manure for treatment was discussed. It was indicated during the workshop that the January, 2007 MMSD issue paper on Agricultural Waste Management recommended the District not be involved in day-to-day manure management. Doing so is not considered a core business for the District and may not be a permissible function of a metropolitan sewerage district under state statutes. However, the District could provide technical assistance if on-farm or regional manure digestion is utilized within the county. The issue paper does state that future evaluations of biosolids management options, including evaluations conducted as part of the Master Plan, should include at least a cursory look at the feasibility of enhancing the District's biosolid products using animal waste. The following issues were also discussed:

- Many obstacles exist if the District was interested in accepting agricultural waste. Regulatory concerns and the affect on rates are two such obstacles.
- The biosolids currently contain significant levels of phosphorus. There is already an excess level of phosphorus within certain portions of the Yahara watershed. For this reason, there may be a need in the future to export biosolids from the watershed.
- If biosolids are produced at separate satellite plants, there may be a difference in quality from varying sources.
- Attention could be paid to the internal distribution of phosphorus within the watershed. Areas that are hypersensitive to phosphorus, such as areas close to the lakes, might not receive any biosolids, while areas poor in phosphorus might receive increased amounts.
- Changing agricultural practices may reduce the amount of fertilizer used by farmers and decrease the demand for biosolids.

- Emerging compounds of concern in biosolids may drive future regulations and limit the ability to beneficially reuse biosolids. This may also apply to WWTP effluent.
- Can biosolids be exported for alternate uses in the future? Using biosolids as compost or an add mixture to other products or for producing manufactured goods could become possible. If so, what type of capital infrastructure would be needed for these alternate uses?

Effluent Discharge and Reuse

Increasing regulatory pressure and energy costs may limit the long term viability of pumping all treated effluent to Badger Mill Creek and Badfish Creek. Also, as noted earlier, water conservation within the watershed is considered a primary issue to address in the future. The volumes and locations at which the District discharges its effluent will be a major factor in sustaining water levels in streams and aquifers throughout the watershed. The following issues on effluent discharge and reuse were discussed.

- The most apparent variable is the ability to discharge effluent into the Madison Lake system. This will depend heavily on effluent quality limits, regulatory judgement and public perception. Legislative changes may also be required.
- Decentralized local treatment plants could be a direction in the future. These facilities could reduce inter-basin water transfers by reusing effluent within the basin that it was generated. They also would eliminate the need to pump effluent long distances, thereby reducing energy costs associated with pumping.
- An area near St. Louis, Missouri has employed a decentralized treatment system to supplement base flow in a local stream. It was not the lowest cost solution, but the community still decided to go in that direction. Obtaining more information on this effort would be helpful.
- Who would ultimately be responsible for running the decentralized facilities? If operational responsibilities remain with the District, there may be workforce availability issues associated with operating multiple facilities. Greater levels of automation may be necessary in response to a shrinking workforce, and workers' skills will have to adapt to dealing with more remote automated control systems.
- Conservation of water on the intake side of the water system will be essential to achieve sustainability. Current pumping of groundwater is lowering the groundwater table and reducing baseflow to streams and springs.
- Energy conservation and water conservation should be considered equally important.
- Augmenting low water flow areas with treated effluent is an option, but the ability is needed to divert or manage the effluent in some other manner during high flow events. Nine Springs WWTP can utilize its lagoons for storage, but they can only

hold 66 million gallons, a volume of water equal to approximately one and one-half days worth of dry-weather plant influent volume.

- Reintroduction of treated effluent back into the groundwater through infiltration or recharge could be a viable option to address water quantity concerns.
- Microconstituents found in treated effluent such as pharmaceuticals, disinfection byproducts and viruses may be subject to increased regulation and create public perception issues that could limit the viability of using effluent for groundwater recharge.
- Would there be enough available land area to implement effluent reuse options involving infiltration to an extent that it would have a significant impact on groundwater quantity?
- From an ecological perspective it may be better to augment existing base flows than to recharge aquifers.
- Use of wetlands for effluent polishing and use of effluent in reclaiming wetlands need to be further investigated.
- The reuse of 'grey water' in non drinking applications appears to be a sensible option for the reduction of water consumption. How to go about implementing and integrating such systems remains an issue.
- Major water consumers such as industrial parks and golf courses should be targeted first for instituting water reuse systems.
- Public perception can influence the ability to institute water reuse options, and information/education efforts will need to be undertaken to impact public perception. The discussion in 2003 related to using effluent for cooling water at the new co-gen facility on campus highlighted the need for information/education activities. Staff from the University of Wisconsin expressed concerns related to reusing effluent because of perceptions that use could impact human health.
- The majority of wastewater flow is generated by residential sources. The residential capacity to take on new grey water systems needs to be investigated.

Regulatory Trends

Jane Carlson summarized the regulatory issues that could impact the master planning process. These issues were summarized in a draft technical memorandum that was distributed to the advisory committee in advance of the meeting.

Higher concentrations of organic waste in plant influent will not greatly affect the plant operations. What will greatly affect plant operations is the lowering of effluent discharge limits on phosphorus and nitrogen, and/or imposition of limits for chlorides or emerging contaminants.

Stormwater Management

Currently communities served by the District have separate storm and sanitary sewer systems. Stormwater is captured in dedicated storm sewers and discharged to detention basins or directly into adjacent water bodies. Currently the District has no involvement in stormwater management. The following issues were discussed:

- Under its current policy, the District will not get involved in stormwater management unless the following three conditions are met:
    a. A stormwater problem with water quality implications requires a regional solution;
    b. The involved municipalities are unable to implement a coordinated plan; and
    c. There is consensus that the District is the appropriate agency to deal with the issue.
- The District has to indirectly deal with stormwater flow because of inflow and infiltration into the sanitary collection system. The District generally has sufficient capacity to treat increased flows resulting from I/I, but has limited capacity to pump treated effluent away from the plant. The lagoon system is currently used for effluent equalization purposes.
- A white paper produced by NACWA on climate change and potential impacts on precipitation events will be sent out to the group. Are there any plans to adapt to possible environmental change that brings larger storms and potentially higher flows at the treatment plant?

Environmental Impacts

The overall environmental impact of the District's facilities and operations will be considered in the planning efforts. Carbon footprints, air quality and generation of hazardous materials are examples of the kinds of items to be examined under this variable as summarized in the Planning Variables Descriptions document that was distributed before the meeting.

Future Flow Projections

Future flow projections have significant impacts on capacity requirements for both the collection system and treatment facilities. The following items were discussed:

- Different means of transporting wastewater could reduce wastewater flows.
- Water conservation efforts could have a major impact on future flows.
- Conservation efforts may be offset by population growth.

- What has been the accuracy of population projections in the past? Is there a tendency to over or under estimate?
- Flow projections developed by MMSD in the past have proven to be fairly accurate, usually within ±5% of the actual flows.
- Should MMSD consider using a more conservative peaking factor when designing interceptor sewers and pumping stations? The current "Madison Design Curve" is based on data from fifty years ago. Do the public's current expectations on level of service (frequency of sanitary sewer overflows) require a more conservative approach? Will climate change result in more intense storm events with resulting higher peak flows?
- The population growth rates could drastically change in the future. Climate change could lead to major migration patterns. There could also be a decrease in population as is the case in more highly developed regions such as Western Europe.
- Inflow and infiltration of groundwater into the collection system is a major cause of peak flows to Nine Springs.
- Exfiltration from the District's interceptor system is unlikely because they are generally located below the groundwater level. Exfiltration from local collection systems and building laterals may be an issue since viruses have been detected in the deep aquifer used as the area's drinking water source. The master plan will have a section assessing the general condition of the District's interceptors.

Construction/Operational Costs

- Construction and operational costs will be major drivers for all scenarios and alternatives.
- Availability of skilled manpower will impact the ability to implement new services using multiple small-sized satellite treatment plants.

Public Acceptance

Phase 1 of the public involvement initiative has been completed. Strand is developing a memo on the topic of Phase 1 activities and results. The following items were discussed:

- A two-pronged method was used involving surveys and public presentations.
- Surveys were mailed to 260 stakeholders in the Madison area. The response rate was approximately 70%, which is excellent. The survey asked responders what level of acceptance they had for various water reuse applications and the additional monthly service charge amount they would be willing to pay to implement new water reuse alternatives.

- From February to April, 14 presentations were given in various communities, primarily to public works committees/boards. The purpose of the presentations was to introduce the District, introduce the master plan and to introduce some of the major water issues facing the area.
- Most attendees at the presentations were supportive of the introduction and public relations efforts and also supportive of water conservation and the concept of wastewater reuse. Groundwater appeared to be a bigger concern than surface water. In general the opinions and ideas expressed at the presentations were similar to those brought up in the earlier workshops.
- One disappointment was that there was only one response from the 40 environmental groups that were contacted to set up a presentation, and no group requested a presentation. Any ideas on how to engage these groups is welcome.
- It was suggested that MMSD should be put on the list of parties to be contacted as part of the Yahara Clean initiative and other related local initiatives.
- More public involvement efforts should be extended to average citizens rather than public works personnel.
- A comment was made during the planning variables meeting that many people might not know who MMSD is and may think that they are part of the City of Madison.

Miscellaneous

- System flexibility, maintainability, and similar operational characteristics shall be evaluated as part of the planning efforts.

Technical Memorandum No.5

Date:	May 30, 2008
To:	Madison Metropolitan Sewerage District
From:	Jane Carlson, P.E.
	Project Engineer, Strand Associates, Inc.
	Randy Wirtz, PhD, P.E.
	Project Manager, Strand Associates, Inc.
Subject:	50-Year Master Plan
	TM-5: Regulatory Review and Analyses (Final)
Project No.:	MMSD No. 8425001
	MPI No. 6100-001

1.01 TECHNICAL MEMO OVERVIEW

This Technical Memo reviews existing and foreseeable future regulatory issues potentially affecting Madison Metropolitan Sewerage District's (MMSD's) planning and operations in the next 50 years. This review is conducted for various potential alternatives, such as continued treatment at a single Nine Springs Wastewater Treatment Plant (NSWTP), or potential new facilities north of Lake Mendota (Mendota Plant) and/or on the Sugar River near Verona. Potential treatment at NSWTP with discharge to Lake Waubesa or an increased discharge to Badger Mill Creek and treatment of some MMSD wastewater flows at the Sun Prairie or Stoughton Wastewater Treatment Plants (WWTPs) are also reviewed. A meeting to discuss relevant water quality and regulatory issues was held with the Wisconsin Department of Natural Resources (DNR), Capital Area Regional Planning Commission (CARPC), Dane County, and others on December 7, 2007. Information from that meeting forms the basis for much of this memo.

1.02 SURFACE WATER REGULATIONS POTENTIALLY IMPACTING ALL ALTERNATIVES

Current rules affecting effluent limitations for a discharge to surface waters are contained in Wisconsin Administrative Code Chapters NR 102, NR 104, NR 105, NR 106, NR 207, NR 210, and NR 217. Additional regulatory or quasi-regulatory initiatives at the local, state, or federal levels (e.g., DNR listing of impaired waters) could impact surface water discharge. Issues of particular importance relative to alternatives being considered in the Master Planning process are summarized below.

1. Phosphorus (P) criteria: The District's Wisconsin Pollutant Discharge Elimination System (WPDES) permit currently contains an effluent limit of 1.5 mg/L to comply with requirements specified in NR 217. The DNR is undertaking a new regulatory initiative that could lead to the establishment of P water quality criteria through revisions to NR 102 and 106. Proposed limits for rivers and streams range from 0.075 to 0.105 mg/L. Proposed limits for reservoirs and lakes (excluding the Great Lakes) range from 0.015 to 0.04 mg/L. These requirements could result in MMSD needing to further reduce effluent P levels and/or offset P loads through a watershed-based trading program. MMSD is a

member of the DNR's technical advisory committee developing the P criteria and companion implementation language. The DNR currently anticipates that revisions to NR 102 and 106 will be complete by mid-2009.

2. Total nitrogen (TN) criteria: The DNR does not currently have a schedule for TN criteria development. However, the United States Environmental Protection Agency (EPA) is interested in states developing these criteria soon. It is anticipated that these criteria may be developed by around 2010 or 2011. If developed, TN criteria would likely be promulgated under NR 102 and 106 and may result in MMSD needing to provide an additional level of treatment and/or offset TN loads through a watershed-based trading program.

3. P and TN effluent standards: The Natural Resources Defense Council (NRDC) has petitioned the EPA to revise the definition of secondary treatment to include nutrient removal. The NRDC has recommended effluent standards on the order of 0.3 mg/L for total P and 3 mg/L for TN. The future of this initiative is unclear.

4. Chlorides: Chlorides are currently addressed in NR 106: Chloride concentrations in MMSD effluent continue to increase primarily because of the use of in-home water softeners. MMSD does not yet have a numeric effluent limit for chloride in its WPDES discharge permit, although future requirements are likely. These requirements could take multiple forms, including the establishment of an interim limit, a target value, and the requirement to establish a source reduction program. MMSD currently provides public education regarding optimizing water softener salt usage.

5. Mercury and other metals: Mercury and other toxics are currently addressed in NR 105, with additional requirements for mercury in NR 106. MMSD does not currently have a mercury limit in its WPDES permit. However, consistent with the mercury variance language in NR 106, MMSD has developed a mercury minimization program. In addition, mercury sampling of effluent is required. A 1.3 mg/L limit for mercury may be included in a future WPDES permit, pending EPA requirements. MMSD's WPDES permit does not include limits for other metals, and none are anticipated in the near future. However, effluent monitoring is required for several metals, including cadmium, chromium, copper, lead, nickel, and zinc.

6. Thermal standards: The DNR has issued draft revisions to NR 102 and NR 106 regarding temperature impacts from certain point source dischargers including WWTPs. The DNR accepted public comments on the draft rules in February 2008 and is now developing its response to the comments. These rules may become final in late 2008. As currently written, the draft rules allow an existing WWTP to apply for a variance if they have a continued or increased discharge to an existing outfall. Therefore, the rules should not affect the NSWTP Badfish Creek or Badger Mill Creek outfalls as long as MMSD applies for the variance. New discharge locations may need to comply with the standards, however, and this could require installation of heat exchangers, cooling towers, or other facilities.

7. Microconstituents: Microconstituents are gaining attention across the country because of their potential negative impact on aquatic and other communities. This may eventually result in promulgation of new or more restrictive effluent limits requiring additional monitoring and perhaps additional treatment. There is no Wisconsin rulemaking process underway yet, but NR 105 secondary value language could be used to regulate pollutants that are demonstrated to cause harm.

8. Water quality assessment and impaired waters listing: Impaired waters are those waters that are not meeting state water quality standards as defined by Section 303(d) of the federal Clean Water Act. Every two years, the DNR is required to submit a list of impaired waters to EPA for approval. Waters on the 303(d) list are given high priority for regulatory action, which could include development and implementation of total maximum daily loads (TMDLs) to address the cause of impairment. Information from the DNR's 2006 and draft 2008 assessment document and list of impaired waters (303(d)) for Badfish Creek is included in Table 1.03-1. As additional water quality criteria (WQC) are developed, the pollutants of concern for which local water bodies are listed may increase; for example, P may be added to the list of potential causes of impairment for additional water bodes in the 2010 303(d) list. In this case, additional TMDLs could eventually be developed by the DNR, or other DNR-approved studies and measures could be taken to address the impairments.

9. Water balance issues: Water balance issues will receive increased attention, with the expected drivers occurring at the local or regional level. For example, water balance and the impacts of groundwater pumping are discussed in the *Dane County Groundwater Protection Plan*, prepared by the Dane County Regional Planning Commission (DCRPC; 1999). An increased discharge to Badfish Creek will exacerbate existing issues related to groundwater table decline and surface water base flows in the greater Madison area, unless other measures are taken to offset the increased discharge. The groundwater table decline and effluent diversion currently reduce base flows in some area rivers and streams. It also impacts area springs and wetlands. Impacts are becoming apparent in the Rock River and Sugar River Basins. Some City of Middleton officials have expressed concern that groundwater pumping will begin to impact base flows in Black Earth Creek in the Wisconsin River Basin if current trends continue. Offset measures could include discharge of effluent to appropriate locations in the affected watersheds, water conservation, reduced groundwater pumping, increased stormwater infiltration, or other means to replace water discharged to Badfish Creek via the NSWTP. These latter measures would traditionally be led by area municipal water supply and stormwater agencies; however, a cooperative approach among multiple agencies including MMSD has been suggested by the DNR and Dane County. Additional detailed information about this issue can be found in various publications including the *Dane County Groundwater Protection Plan*, 1999, and the MGE publication titled *MGE-UW West Campus Cogeneration Facility–Final Environmental Impact Statement*, June 2003.

1.03 SURFACE WATER REGULATIONS IMPACTING NSWTP ALTERNATIVES

A. NSWTP Continued Discharge to Badfish Creek

A summary of the DNR's listing and assessment information is shown in Table 1.03-1, and potential effluent limits related to a continued discharge to Badfish Creek are summarized in Table 1.03-2. Potential future issues besides those noted previously are summarized below.

1. Rock River Basin P and sediment TMDL: The TMDL is being developed by consultants under contract with the EPA. The draft TMDL report is scheduled to be issued for public comment late 2008 or early 2009. It appears MMSD will have a waste load allocation (WLA) for total P as a result of this TMDL. The magnitude of the WLA will be dependent upon other sources of P loading in Badfish Creek, lower Yahara River, and Rock River and will also depend on the method(s) used by the EPA to allocate the TMDL between point and nonpoint sources. At this time, since Badfish Creek itself is not listed as impaired because of P, we expect that MMSD's WLA will be set assuming a NSWTP design average flow of 50 million gallons per day (mgd) and a target P concentration in the Yahara River around 0.1 to 0.125 mg/L. The EPA will also look at other sources of P in the Yahara and Rock Rivers when determining the WLA. It is anticipated that the DNR will develop a companion Implementation Plan after the TMDL report becomes final, and it appears likely the DNR will involve MMSD and other stakeholders in its preparation. Among other things, the Implementation Plan may be used to further refine allocations [WLAs and load allocations (LAs)], schedules, and methods for incorporating WLA-related effluent limits into permits.

2. P criteria: The proposed P criteria for Badfish Creek is currently 0.075 mg/L. It appears the Yahara River P WQC downstream of the confluence of Badfish Creek will be around 0.1 mg/L. Depending on the background concentration of P in Badfish Creek (i.e., from groundwater or other sources of dilution water), some dilution may be allowed when determining the associated water quality-based effluent limit (WQBEL) for P.

B. NSWTP with Increased Discharge to Badger Mill Creek

The MMSD is presently permitted to discharge up to 3.6 mgd to Badger Mill Creek and may consider alternatives that increase this discharge. Badger Mill Creek is a tributary to the Sugar River. The Sugar River has been designated an exceptional resource water (ERW). Water quality assessment and listing information for Badger Mill Creek and downstream Sugar River are shown in Table 1.03-1. Current WPDES permit limits for MMSD's discharge to Badger Mill Creek are summarized in Table 1.03-2; however, for an increased discharge, the effluent limits could be impacted by the more stringent rules related to the Sugar River. The initiatives listed for surface water discharges (Section 1.02 A) would apply to an increased discharge to Badger Mill Creek, as would the following.

1. P criteria: The current draft administrative code language for P criteria would result in a P WQC around 0.075 mg/L for Badger Mill Creek. Depending on background concentrations, some dilution may be allowed when determining the WQBEL for P.

However, the P concentration for an increased discharge at this location may be limited further because of the downstream Sugar River ERW designation.

2. DNR interpretation of antidegradation requirements: Antidegradation rules are contained in NR 207. Since the Sugar River is an ERW, it is subject to more stringent antidegradation requirements. In general, a new discharge to an ERW needs to meet upstream water quality. Regulations are not as stringent for an increased existing discharge; however, the permittee would still need to demonstrate there will either be no significant lowering of water quality or that the project has sociological and economic benefits. Because of this, MMSD may need to perform modeling, stream studies, or other analysis to demonstrate that an increased discharge to Badger Mill Creek will not result in significant lowering of water quality in the Sugar River or that the project is otherwise justified. Such demonstration may be particularly important for effluent parameters like ammonia, chloride, and P.

3. Water balance issues: This discharge location would help offset some of the water balance issues caused by the pumping of water supply wells located in west Madison and Verona. It will help maintain base flows in Badger Mill Creek and the Sugar River.

C. NSWTP with Discharge to Lake Waubesa via Nine Springs Creek

One alternative that MMSD may consider is discharge of highly treated effluent to Nine Springs Creek or wetlands tributary to Mud Lake and Lake Waubesa. MMSD is presently allowed to discharge to Nine Springs Creek on an emergency basis only and has only done so on rare occasions when the capacity of effluent pumps and on-site storage structures is exceeded. Water quality assessment and listing information for Nine Springs Creek and Lake Waubesa are shown in Table 1.03-1. Current potential effluent limitations for a discharge to Nine Springs Creek are also summarized in Table 1.03-2; however, the effluent limits would likely be most impacted by the more stringent statutes related to Lake Waubesa. In addition to those noted in Section 1.02, a discharge to Lake Waubesa would be affected by the issues summarized below.

1. Thermal standards: If this discharge location is construed as an existing outfall for MMSD, it is possible that it would be eligible for a variance to the proposed thermal standards outlined in draft revisions to NR 102 and NR 106. Otherwise, some mitigation of effluent temperature may need to be included for a discharge at this location.

2. P criteria: The current draft administrative code language for P criteria would result in a P WQC around 0.040 mg/L for shallow lakes like Lake Waubesa. Depending on the background concentration of P in the lake, some dilution may be allowed when determining the WQBEL for P.

3. DNR interpretation of requirements in Wisconsin State Statute 281.47: This statute was the driver for MMSD diverting effluent around the Madison lakes beginning in the late 1950s. The statute does not explicitly prohibit direct discharge of effluent to the chain of lakes including Lake Waubesa, but it does place conditions that must be met for direct discharges to occur. The DNR is given authority to determine whether these conditions

are met. Based on DNR discussions during Madison Gas and Electric's (MGE's) cogeneration facility planning, it appears the effluent quality would need to be close to background surface water quality for P prior to approval of a Lake Waubesa discharge. Background concentrations may be close to the 0.040 mg/L proposed shallow lake criteria.

4. Water balance issues: This discharge location would help offset some of the water balance impacts of the discharge to Badfish Creek. Specifically, it would increase dry weather base flows in the Yahara River south of Lake Waubesa.

In summary, implementation of a potential discharge at this general location will depend on addressing the following issues:

1. Public acceptance of a discharge to Lake Waubesa, Lake Kegonsa, and the Yahara River upstream of Stoughton.

2. DNR approval of the discharge based on requirements in the Wisconsin State Statutes.

3. Technical and economic feasibility of constructing additional facilities that will meet proposed P and N standards for a lake discharge.

1.04 SURFACE WATER REGULATIONS IMPACTING SATELLITE WWTP ALTERNATIVES

The satellite wastewater treatment plant (WWTP) concept could include one of the following approaches:

1. Full treatment of all flows generated in a particular area at a satellite WWTP: This could include expansion of an existing WWTP or construction of new WWTPs.

2. Full treatment of dry weather flows generated in a particular area at a satellite WWTP: Peak wet weather flows could be diverted from smaller treatment plants and treated at the NSWTP.

3. Use of "cluster" WWTP systems providing full treatment of wastewater from smaller or remote subdivisions: Examples would include a community mound-type septic system or small package aeration system. Such systems could potentially be owned and/or operated by MMSD.

For any of these approaches, administration, laboratory, and biosolids management services could continue to take place at MMSD's main NSWTP, assuming this is most cost-effective. Surface water regulations affecting such WWTPs are described below. Satellite WWTPs could also have a discharge to the groundwater; the associated regulations are discussed in Section 1.05.

A. <u>Mendota Plant (Upper Lake Mendota Watershed Discharge)</u>

MMSD may consider constructing a satellite WWTP with discharge of highly treated effluent to the upper Yahara River or wetlands tributary to Lake Mendota. Water quality assessment and listing information for this segment of the Yahara River and Lake Mendota are shown in Table 1.03-1. Anticipated effluent limitations for a discharge in this location are also summarized in Table 1.03-2; however, the effluent limits would likely be subject to the more stringent state statutes related to Lake Mendota.

Many of the initiatives listed in Section 1.02, as well as those listed for a Lake Waubesa discharge (1.03 C) would apply to a discharge to Lake Mendota. According to the current draft rules, a new discharge at this location would probably not be eligible for a variance according to NR 102 and NR 106 thermal standards. In addition, a discharge to Lake Mendota would be affected by the issues summarized below.

1. P criteria: The current draft administrative code language for P criteria would result in a P WQC around 0.015 mg/L for Lake Mendota. Depending on the background concentration of P in the lake, some dilution may be allowed when determining the WQBEL for P. The DNR has noted that a TMDL-like approach could be required before setting WLAs, LAs, and WQBELs for a Lake Mendota discharge so that load and wasteload allocations can be assigned to all the sources of P to the lake.

2. Water balance issues: This discharge location would help offset some of the water balance impacts of the existing NSWTP discharge to Badfish Creek. Specifically, it would increase dry weather base flows through the Madison Lakes and in the Yahara River.

In summary, implementation of a potential Mendota plant will depend on addressing the following issues:

1. Public acceptance of a discharge to Lake Mendota and other lakes and Yahara River segments upstream of Stoughton.

2. DNR approval of the facility based on requirements in the Wisconsin State Statutes.

3. Technical and economic feasibility of constructing a facility that will meet proposed P and N standards for a lake discharge.

4. Technical and economic feasibility of constructing a facility that will meet proposed thermal discharge standards.

B. Sugar River Plant (Sugar River Watershed Discharge)

MMSD is considering construction of a satellite WWTP with discharge of highly treated effluent to the Sugar River or its tributaries.

Water quality assessment and listing information for the Sugar River are shown in Table 1.03-1. Anticipated effluent limitations for a discharge directly to or impacting the Sugar River are summarized in Table 1.03-2. The initiatives listed in Section 1.02 would apply to a discharge to the Sugar River. An increased discharge to one of the Sugar River tributaries, Badger Mill Creek, was discussed in Section 1.03 B, and the requirements would be similar for other Sugar River tributaries with the exception that any new discharge (as opposed to an increased Badger Mill Creek discharge) would be subject to additional requirements. For example, it is likely that it would not be eligible for a variance according to the draft revisions to NR 102 and NR 106 for thermal standards. In addition, a discharge to the Sugar River would be affected by the issues summarized below.

1. P criteria: The current draft administrative code language for P criteria would result in a P WQC around 0.075 mg/L for the Sugar River; however, antidegradation requirements contained in NR 207 would also apply. For an Exceptional Resource Water (ERW), this essentially means the new discharge would need to meet background water quality. For example, if the background P concentration in the Sugar River is 0.050 mg/L, the effluent limit could be 0.05 mg/L.

2. Chlorides: Since the Sugar River is designated an ERW, it is possible the chloride concentrations in the discharge would need to meet background concentrations in accordance with NR 207. The DNR has expressed some willingness to discuss this issue further with the MMSD, particularly if there is a net environmental benefit associated with the discharge such as restoration of water balance or other benefits.

3. Ammonia, biochemical oxygen demand (BOD), and other limits: It is possible that the effluent limit for ammonia, BOD, total suspended solids (TSS), and other parameters may need to be equal to background concentrations of these parameters because of the ERW designation for the Sugar River. The DNR Guidance on the "13 pound rule" contains calculations related to assimilative capacity and may impact BOD limits for non-variance streams; this guideline may apply if the background concentration does not.
4. Water balance issues: This discharge location would be used to offset groundwater withdrawals from the Sugar River Basin, as is currently being done with the Badger Mill Creek discharge.

In summary, implementation of a potential Sugar River plant will depend on addressing the following issues:

1. Public acceptance of a new or increased Sugar River discharge.

2. Technical and economic feasibility of constructing a facility that will meet proposed P and N standards for a discharge to the Sugar River, which is an ERW.

3. Technical and economic feasibility of constructing a facility that will meet proposed thermal discharge standards for a Class II trout stream that is an ERW.

4. Technical and economic feasibility of constructing a facility that will meet potential effluent chloride and other limitations based on a new or increased discharge to an ERW.

C. <u>Sun Prairie WWTP (Koshkonong Creek Discharge)</u>

Another alternative MMSD may consider is a cooperative agreement with Sun Prairie to treat a portion of MMSD's wastewater flow. This would result in an increased discharge to Koshkonong Creek. Water quality assessment and impairment listings for Koshkonong Creek and downstream Lake Koshkonong are shown in Table 1.03-1. Sun Prairie is presently permitted to discharge. Current WPDES permit limits for Sun Prairie's discharge are also summarized in Table 1.03-2. The initiatives listed in Section 1.02 would generally apply to an increased discharge to Koshkonong Creek. Since this discharge location is an existing outfall for Sun Prairie, it is likely that it would be eligible for a variance to the proposed thermal standards outlined in draft revisions to NR 102 and NR 106. In addition, an increased discharge to Koshkonong Creek may be affected by the issues summarized below.

1. P criteria: The current draft administrative code language for P criteria would result in a P WQC around 0.075 mg/L for Koshkonong Creek and 0.040 mg/L for Lake Koshkonong. Depending on background P concentrations, some dilution may be allowed when determining the WQBEL for P.

2. Water balance issues: Currently, flow is being diverted from the Yahara River Basin to the Koshkonong Creek Basin because of discharges from the Sun Prairie treatment plant. If future growth in MMSD in the Koshkonong Creek Basin results in the net diversion going to the Yahara River Basin, this discharge location would help offset some of the water balance impacts. Specifically, it would increase dry weather base flows in Koshkonong Creek.

A potential option for the Sun Prairie WWTP would be the return of highly treated effluent from the Sun Prairie WWTP to Token Creek to provide base flows lost in Token Creek because of areas in the Sun Prairie urban service area (USA) that are located in this watershed. This return could be done either directly or through wetlands as a method of restoring the wetlands base flow lost through groundwater depletion as a result of water supply withdrawals for Sun Prairie.

Use of a portion of the capacity of the Sun Prairie WWTP or the implementation of construction of a high quality effluent facility at the Sun Prairie WWTP will depend on addressing the following issues:

1. Intergovernmental cooperation between the City of Sun Prairie and MMSD.

2. Public acceptance of an increased Koshkonong discharge.

3. Technical and economic feasibility of constructing a facility that will meet proposed P and N standards for a discharge to Koshkonong Creek or Token Creek.

4. Technical and economic feasibility of constructing a facility that will meet proposed thermal discharge standards for Koshkonong Creek or Token Creek, which is an ERW of the State of Wisconsin.

5. Technical and economic feasibility of constructing a facility that will meet potential effluent chlorides limitations based on a new discharge to Token Creek, an ERW, or increased discharge to Koshkonong Creek.

D. Stoughton WWTP (Lower Yahara River Discharge)

Another alternative MMSD may consider is a cooperative agreement with Stoughton to treat a portion of MMSD's wastewater flow. This would result in an increased discharge to the Yahara River at

Stoughton. Stoughton is presently permitted to discharge up to 1.65 mgd design average flow at this location and may soon begin facilities planning to increase the design average flow to about 2.35 mgd. Water quality assessment and impairment listings for the Yahara River are shown in Table 1.03-1. Current WPDES permit limits for Stoughton's discharge to the Yahara River are summarized in Table 1.03-2. If the WWTP is expanded, it is anticipated that Stoughton will have more stringent limits for weekly average summer BOD and TSS and more stringent ammonia limits. The initiatives listed in Section 1.02 would apply to an increased discharge to the Yahara River. Since this discharge location is an existing outfall for Stoughton, it is likely that it would be eligible for a variance to the proposed thermal standards outlined in draft revisions to NR 102 and NR 106.

An increased discharge at this location would provide less relief from the water balance issues in the Yahara River, since the water balance concerns in the Yahara River system are greater upstream of Stoughton's discharge point.

Constructing new capacity at the Stoughton WWTP or construction of a new facility to treat both MMSD and Stoughton wastewater near Stoughton will depend on addressing the following issues:

1. Intergovernmental cooperation between the City of Stoughton and MMSD.

2. Public acceptance of an interceptor corridor near Door Creek, if wastewater flows are conveyed from the Cottage Grove area.

E. <u>Oregon WWTP (Badfish Creek Discharge)</u>

MMSD has considered a possible cooperative agreement with the Village of Oregon to treat a portion of MMSD's wastewater flow. Oregon discharges to the Oregon Branch of Badfish Creek, with limits as described in Table 1.03-2. This alternative would be subject to regulations and have similar impacts as those described for an NSWTP discharge to Badfish Creek (Section 1.03 A.). This alternative would provide a minimal water balance benefit.

F. <u>Other Surface Water Discharge Locations Including Stream Base Flow Augmentation</u>

Other surface water discharge locations may be considered, such as a new discharge to the Yahara River just downstream of Lake Waubesa. A discharge at this location would likely have similar issues and benefits as those discussed above for a discharge to Nine Springs Creek and Lake Waubesa.

Base flow augmentation using highly treated WWTP effluent may also be considered in the future, particularly for urban streams. For example, relatively small volumes of effluent could be further treated at the Sun Prairie or a future north MMSD WWTP and discharged to streams in the northeast portion of the Lake Mendota or north Lake Monona watersheds. Starkweather Creek has experienced a reduction in dry weather base flows over the years, possibly caused by the high percentage of impervious surfaces in the watershed and pumping of groundwater in Madison, and could be a good candidate to receive flow augmentation in this manner. A discharge of treated effluent at this location would have similar issues and benefits as those discussed above for a discharge to the upper Yahara River and Lake Mendota.

1.05 WATER REGULATIONS IMPACTING A GROUNDWATER DISCHARGE OR EFFLUENT REUSE

A. Groundwater Recharge

Groundwater recharge using effluent is being practiced in several locations around the state, particularly in the Wisconsin River Valley and other locations where soils are sandy and thus conducive to infiltration. A typical method of effluent groundwater recharge is to use seepage cells (also called absorption ponds), which are regulated under NR 206. Current effluent limitations for discharge to absorption ponds include:

BOD	50 mg/L
TN	10 mg/L
TDS	500 mg/L
Chloride	250 mg/L

Groundwater monitoring is usually required for absorption ponds and the relevant groundwater standards at the design management zone boundary (250 feet from the seepage cell boundary) or at the property line would apply. These are contained in NR 140. The groundwater preventive action limit (PAL) for chloride is 125 mg/L and the enforcement standard (ES) is 250 mg/L.

Groundwater recharge using stormwater infiltration galleries is being practiced at the Odana Hills Golf Course in southwestern Madison. Since the discharge is to the subsurface and therefore cannot rely as much on aerobic and facultative bacteria, plant uptake, and other processes for treatment, the effluent limits for this type of discharge are more stringent. Typically, effluent limits are set equal to groundwater standards or upgradient groundwater quality. For the system in Madison, this has required pretreatment of the stormwater using membrane filtration prior to discharge to the infiltration galleries.

For this type of discharge, it appears the largest hurdles for MMSD to overcome would be TN and chloride effluent concentrations. Biological nitrogen removal can be used to reduce TN to below 10 mg/L. If a variance could not be obtained, chloride concentrations would need to be reduced through source reduction or reverse osmosis treatment prior to discharge to an infiltration gallery and may also need to be reduced prior to a discharge to absorption ponds.

Favorable groundwater infiltration locations were explored as part of the MGE West Campus Cogeneration facility environmental impact review. Four sites were identified in west and south Madison with the projected ability to recharge 120-million gallons per year of stormwater. A discussion can be found in the *MGE-UW West Campus Cogeneration Facility Final Environmental Impact Statement* (MGE, 2003). These sites would need to be reviewed from the perspective of wastewater quality to determine if they would be effective for effluent infiltration. Assuming they were suitable, these sites would be able to accept only a small fraction of MMSD’s total effluent flow. A large, potentially favorable infiltration site has also been identified in Fitchburg.

The use of injection wells is another method of groundwater infiltration. Federal drinking water regulations include five types of injection well permits. Effluent would need to meet NR 140 standards before injection, unless it could be shown that the aquifer receiving the effluent was nonpotable and isolated from water supply aquifers.

Depending on the location of groundwater absorption ponds, infiltration galleries, or injection wells, it may be necessary to provide additional treatment to remove additional pathogens (such as viruses) or microconstituents from the effluent prior to recharge. The upper sandstone aquifer in the Madison area is no longer used for human consumption, and recharge of this aquifer could help provide restoration of local springs and wetlands. However, the lower water supply aquifer is not protected everywhere in Dane County because the shale layer below the sandstone aquifer is discontinuous in some locations. Therefore, infiltration sites would need to be carefully selected if higher levels of treatment are not provided.

B. Nonresidential Irrigation

The current MMSD permit contains provisions related to use of effluent on the Nine Springs Golf Course in Fitchburg as a demonstration project. This type of discharge would be regulated under NR 206. Current regulations include a BOD effluent limitation of 50 mg/L. Hydraulic loading rates and load and rest cycles are determined on a case-by-case basis and generally depend on the soil type. Likewise, TN and fecal coliform limits are determined on a case-by-case basis. Groundwater monitoring is often required for these systems, particularly when significant pretreatment is not provided. Groundwater standards for chloride (125 mg/L PAL and 250 mg/L ES) may be of greatest concern for MMSD's effluent.

Nonresidential irrigation would generally involve spray or drip irrigation of treated wastewater onto agricultural fields, grass lands, golf courses, or similar areas. Spray irrigation onto agricultural land has been practiced in Wisconsin for many years, primarily for industrial or small (less than 1 mgd) municipal wastewater treatment systems. Muskegon County, Michigan, has a large effluent irrigation and rapid infiltration facility near the shores of Lake Michigan that has been operating successfully for many years. Generally TN applications are limited to crop uptake rates, which are on the order of 165 lb/acre-year for corn and 300 lb/acre-year for certain grasses like reed canarygrass. Groundwater monitoring is often required for determining compliance with groundwater standards.

Crops from a spray irrigation field are typically harvested to remove nutrients. Crops may be used for cattle feed or bedding. More recently, crops like switchgrass are being explored as a potential biofuel for energy generation. The crops are not typically used for human consumption if domestic wastewater is used for irrigation, unless the effluent meets very stringent reuse standards such as California Title 22. Title 22 standards include a turbidity of 2.0 nephalometric turbidity units (NTU) and two total coliforms per 100 mL. In general, this requires advanced filtration and may require a chlorine residual.

Golf course irrigation using WWTP effluent is particularly attractive in areas where fresh water is scarce, such as the Southwest United States. However, it is gaining popularity in Illinois and elsewhere in the Midwest as a way to reduce discharges to receiving streams and reduce the use of fresh groundwater or surface water on golf courses. High salts can be a concern for land application because of the reduced-permeability impact on clayey soils from sodium (particularly if the sodium is high in proportion to other cations in the wastewater) and because of the concern about chloride applications resulting in an exceedance of groundwater quality standards. High salt concentrations can also result in "burning" of foliage or stunted growth; however, this is not typically a concern for domestic wastewater irrigation in temperate climates. After four years of operation, MMSD's Nine Springs Golf Course test plot has not shown adverse effects from salts, according to the greens keeper.

Irrigation using treated wastewater may be beneficial for the water balance issues in Dane County. First, irrigation of cropland may result in increased groundwater recharge. This is less likely for irrigation on golf courses because of high turfgrass evapotranspiration rates. Second, if golf course irrigation using effluent reduces the use of fresh water from rivers, lakes, or the ground, then it will also help improve water balance.

Implementation of any facilities for effluent reuse for irrigation, particularly onto food crops, or athletic fields or parks, will depend on addressing the following issues:

1. Public acceptance of the use of effluent for irrigation.
2. DNR approval.
3. Potential Wisconsin Department of Health and Family Services (DHFS) approval related to any public health concerns (total coliform limitations).
4. Availability and quantity of potential irrigation sites.
5. Proximity of irrigation sites to facilities that could provide highly treated effluent.
6. Available corridors to convey the treated effluent to the irrigation sites.

C. Industrial or Commercial Reuse

Wastewater effluent is being used for industrial noncontact cooling and other noncontact uses. An example in Wisconsin is the Heart of the Valley wastewater treatment plant in Kaukauna where highly treated effluent is being used by a nearby power plant for cooling. Wisconsin currently has no standards for the treatment of effluent for use in an industrial facility.

In 2002, MGE briefly explored the use of MMSD effluent for cooling at its new West Campus Cogeneration facility. A discussion is contained in *MGE-UW West Campus Cogeneration Facility Final Environmental Impact Statement* (MGE, 2003). The use of effluent would have offset MGE's 2.75 mgd proposed full build-out water withdrawal from Lake Mendota. The cost for additional disinfection and filtration to remove protozoans, and a pipeline to convey effluent to the Cogeneration facility, was determined to be approximately $9.5 million at that time. Annual operational costs were projected to be $135,000. There were also concerns from the UW regarding use of the effluent in a residential and campus setting for a facility of the size being considered, so the concept was not pursued further.

It may also be possible for effluent to be reused for noncontact industrial cooling water. Several individuals responding to the MMSD interest survey indicated that commercial car wash use may be another viable alternative; however, the locations of such facilities may be too diffuse for cost-effective conveyance of the treated effluent. The concept should be initially explored with the largest water users in Dane County who are believed to use fresh water for nonpotable uses. Potential users are listed here:

- Oscar Mayer.
- Golf courses, particularly in or just north of Madison.
- MGE Blount Street or other location.
- UW Physical Plant.
- Lycon Corporation.
- Wingra Stone.

Ethanol production consumes approximately five gallons of water per gallon of ethanol produced, and consideration could be given to colocating an ethanol production plant so that it can cost-effectively use WWTP effluent. This would still result in a net loss of water from the basin, but from a water resources standpoint, this would be preferable to an ethanol plant using fresh water. CARPC has indicated that it may be difficult to site an ethanol production plant in Dane County if they want to use groundwater or surface water. In Wisconsin, outside of Dane County, the communities of Cambria, Milton, Jefferson, and Monroe all have ethanol production plants being planned.

Other potential uses identified at the MMSD regulatory review meeting include sod farms and large agricultural operations that currently use fresh water for flushing systems in barns and other purposes.

D. <u>Residential Reuse</u>

It has been proposed by several individuals that treated effluent could be reused for toilet flushing, residential lawn irrigation, and other residential nonpotable water uses. Such a concept would require effluent treatment to a very high level (potentially California Title 22 standards as noted above for food crop irrigation), require force mains to convey the treated effluent to the residential developments, and require a new infrastructure similar to the “purple pipe” reuse water distribution systems used in the Southwest and elsewhere. This concept may be worth considering for new developments where installation costs would be lower compared to existing developments. However, it is likely that costs of such systems would outweigh the benefits, at least in the short term in the Madison area. For the short term, it appears that residential water conservation measures may provide similar benefits at a significantly lower cost.

Implementation of any facilities for effluent reuse for residential irrigation or reuse will depend on addressing the following issues:

1. Public acceptance of the use of effluent for residential purposes.
2. DNR approval.
3. Potential DHFS approval related to public health concerns (total coliform limitations).
4. Availability and quantity of potential reuse sites.
5. Proximity of residential reuse sites to facilities that could provide highly treated effluent.
6. Available corridors and distribution systems to convey the treated effluent to the reuse sites.

E. Wetlands Restoration

The DNR has indicated that a discharge to wetlands may be subject to less stringent requirements than a discharge to an ERW stream or the Madison lakes, particularly for restored wetlands. A potential option for the Mendota Plant would be to discharge effluent to wetlands to provide the base flow for the wetland system that has been lost because of groundwater table lowering from water supply withdrawals in Madison, Waunakee, DeForest, Windsor, and Sun Prairie. This option may also be useful in lieu of a direct stream or lake discharge in the vicinity of the Sugar River or Nine Springs Creek/Lake Waubesa.

Wetland discharges are regulated under NR 103. NR 103 applies to natural and restored wetlands but not to constructed wetlands for wastewater treatment or polishing; the latter systems are typically constructed with liners separating them from natural waters and are considered a wastewater treatment unit process.

NR 103 addresses water quality and functional use of a wetland. Discharge of treated effluent to a natural wetland would require high levels of treatment, possibly similar to a Madison lakes discharge. Otherwise, the functional use of the natural wetland could change over time as oxygen demanding substances, nutrients, and other effluent constituents could eventually affect the types of plants and the water quality in the wetland.

Because of the concern over changed functional use of natural wetlands, an effluent discharge to a restored wetland may be more cost-effective, assuming conveyance costs are not too high. The DNR has indicated that they provide greater flexibility regarding functional use in the case of a restored wetland. Restoration of wetlands provides many other environmental benefits and could result in P trading or other credits for MMSD. Wetland restoration also tends to be viewed favorably by the public and environmental advocacy groups. CARPC and the DNR have mapped potentially restorable wetlands and have indicated that some of these areas are quite large and may provide good benefit for the cost.

1.06 OTHER KEY REGULATIONS ISSUES

A. Biosolids Management

The following biosolids regulations have been identified as possibly being applicable to MMSD's future operations. Within the next 20 years, these regulations along with increased development in the Madison area may result in the requirement for more land and increased hauling distances in the Metrogro program. These regulations may also place additional restrictions on the MetroMix program. In the longer term, MMSD may need to consider additional alternatives for at least a portion of its biosolids such as landfilling. Landfilling may still be considered a beneficial reuse option if biosolids are used as cover material, are used to facilitate decomposition, are part of a landfill bioreactor, or if biosolids additions promote the formation of landfill gas that is then recovered and used to generate electricity.

1. State (NR 204) and federal (40 CFR Part 503) biosolids regulations: These regulations may come under triennial or other review. New requirements (lower limits for existing parameters, new limits for microconstituents, development of risk-based pathogen

requirements, or different pathogen indicator organisms) could be promulgated. If so, these requirements could impact MMSD's ability to beneficially reuse biosolids through the Metrogro and MetroMix programs or though other beneficial reuse initiatives.

2. Runoff management rule (NR 151), Soil and Water Resource Management Rule (ATCP 50), and Wisconsin NRCS 590 Nutrient Management Standards: These rules could place restrictions on MMSD's Metrogro applications to agricultural land by limiting phosphorus loadings. The rules are presently being rewritten and are expected to require agricultural fields to meet a phosphorus index of 6 (an average of approximately 6 pounds P runoff per acre per year). Potentially lower indexes would be applied where needed to meet a TMDL. Eventually the state could impose additional restrictions on fertilizer P applications to agricultural land unless soil testing indicates P is needed. These types of regulations could lead to future competitions between applications of manure and biosolids.

3. Impaired waters (303(d)) listings and TMDLs: Commonly thought of in terms of impacts on effluent quality, a TMDL could also impact Metrogro applications because it would place restrictions on both point and nonpoint (e.g., agricultural runoff) loads to an impacted waterbody or stream segment. In the short term, nutrient and/or sediment-related TMDLs are most likely to cause impacts.

4. Local ordinances relating to the use of lawn fertilizers containing phosphorus: As currently written, Dane County and City of Madison ordinances would have little or no impact on MMSD's ability to use MetroMix. However, revisions to these ordinances and/or adoption of new ordinances developed by the state or other communities could have a negative impact on the ability to use MetroMix.

5. State regulations covering fertilizers and related products (ATCP 40): Current regulations reference sewage sludge and sewage sludge products. Revisions could lead to the imposition of significant fees based on mass of material produced, which may influence future decisions regarding potential biosolids management options.

B. Pollutant Minimization

Pollutant minimization will be more important in the future, particularly for compounds that are difficult or costly to remove using current treatment methods. MMSD has a pretreatment program that requires regulated industries to meet local limits for metals and other parameters. In 2007 MMSD cosponsored a pharmaceutical take-back program to reduce the chance that these compounds will end up in the wastewater. They have also developed a mercury minimization plan and an ordinance requiring dental offices to recover mercury amalgam. They are investigating sources of chloride in the wastewater in efforts to reduce this compound in the NSWTP influent. Pollution prevention and source reduction is more proactive than wastewater treatment so is often viewed more favorably by the public. There are often economic benefits of pollution prevention.

Future potential opportunities for pollutant minimization include:

1. Additional sponsorship of pharmaceutical take-back programs.
2. Ordinances against discharge of microconstituents to municipal sewers.

3. Additional mercury reduction ordinances or strategies.
4. Additional local pretreatment limits to further reduce heavy metals or other compounds in the wastewater.
5. Coordination with Madison or other water utilities for significant changes in water softening practices (centralized reverse osmosis or lime softening) to reduce chlorides in the wastewater.

C. Pollutant Load Trading

Pollutant load trading is a likely future initiative because it can reduce the cost of compliance with new water quality or other regulations. The Rock River publicly owned treatment works (POTW) group explored pollutant load trading for a pilot P TMDL in the late 1990s and early 2000. At that time, the cost to remove P to 1.0 or 1.5 mg/L at WWTPs was found to be low enough that pollutant load trading was not worthwhile. There were other issues affecting pollutant load trading at the time, such as high trading ratios being suggested by the DNR (i.e., requiring the removal of two or three pounds of agricultural P in trade for one pound of WWTP P), that contributed to the decision to remove P at the WWTPs at the time. Since then, the DNR has collected additional performance data and more sophisticated models have been developed to better predict P reductions through agricultural best management practices (BMPs), and this should reduce trading ratios. The DNR has also indicated interest in working with WWTPs on many of the other trading issues such as the length of time a trade would be permitted. Pollutant load trades such as those listed below may be beneficial to MMSD in the future.

1. Installation of agricultural BMPs or restoration of wetlands in the Badfish Creek or Yahara River watersheds to allow for a higher effluent P limit at MMSD's NSWTP. It is possible that this would reduce or eliminate the need for effluent filtration. This could also be used for nitrogen load trading.

2. Potentially, participation in sediment removal, dam removal, or stream bank restoration projects in the watershed to reduce in-stream sources of sediment and P in exchange for higher effluent limits.
3. Treatment of its wastewater to levels below MMSD's permit limit. This option would then produce credits that could be sold to downstream P dischargers.

4. Assistance to or coordination with local communities for changes to deicing practices to allow an increased chloride discharge to the groundwater or Sugar River basin surface waters. This type of cooperative effort is underway in the DuPage River/Salt Creek Watershed in Illinois.

D. Air Quality

The following air quality-related regulations or initiatives have been identified that may impact MMSD's operations.

1. State air regulations (NR 404, NR 405, NR 406, NR 407, NR 429, NR 438, NR 439, and NR 445): These regulations could impact multiple areas of MMSD's operations including emissions from unit processes, digester gas-fueled engines, biosolids processing, and

biosolids land application. Typical focus is on parameters such as carbon monoxide, nitrogen and sulfur oxides, and volatile organic compounds. These regulations may require additional permitting, sampling, covering of tanks, collection and scrubbing of exhaust from engine generators, or other measures in the future.

2. Federal air regulations (Maximum Achievable Control Technology standards): These regulations would primarily apply to new WWTP projects, or major reconstruction projects, that are over a certain size. Currently, a major source is defined as one that emits more than 10 tons per year of a hazardous air pollutant (HAP) or more than 25 tons per year of a combination of HAPs. Currently, none of MMSD's operations result in a major source of HAPs.

3. Greenhouse gas emissions and climate change: The global warming issue may not be far enough along to affect WWTPs for the next decade or so. In Wisconsin, the Governor's task force is presently focusing on larger sources. However, Wisconsin Focus on Energy and Energy Star programs are two that are actively assisting WWTPs with greenhouse gas emission reductions through grants and incentives and accounting tools. In the future there may be more opportunities for cooperative projects such as irrigation of wastewater onto "biomass" crops that are subsequently used for ethanol or direct energy production. This could reduce the need for chemical fertilizers to establish the crops, which has been one criticism of the current practice of growing corn for ethanol production, for example.

There may be regulations related to climate change and sizing of conveyance and treatment facilities in the future to address extreme weather events.

1.07 FINAL COMMENTS

On the surface, it appears that current or pending water quality regulations tend to point MMSD toward no change in its current practices. If the public approves of MMSD's current practices, and assuming it is more cost-effective to maintain the single-WWTP model, there may be little impetus for MMSD to build satellite plants or implement water reuse strategies. Water reuse and similar initiatives may require partnerships with other agencies under the direction of a state or county agency.

It is anticipated that the Great Lakes Compact and other water quantity initiatives may raise public awareness of the water quantity/balance issue in the Madison area. Also, if lawn watering or other restrictions become commonplace, the public will see more value in its water resources. This may shift trends toward mandated or voluntary, cooperative measures for interbasin water balance efforts.

Along with the MMSD, the public should continue to think about its water quality and quantity goals and how much they are willing to pay to realize those goals. For example, should MMSD operate at lowest cost possible or operate at a higher cost and be part of a holistic solution to water quantity issues in Dane County? What are the public's aquatic life and recreational use goals–is maintenance of our current fisheries a high enough goal? Are algae blooms in the Madison lakes infrequent enough that they do not cause too much of a recreational use nuisance?

Some of these questions will continue to be asked as part of future development approvals in Dane County, as well as during the DNR's triennial standards review process. As they relate to the 50-Year Master Planning process, MMSD has taken a proactive approach by asking some of these questions through its user questionnaire and public outreach process.

1.08 DEFINITIONS

Following are definitions for the acronyms and abbreviations used in this technical memo.

ATCP	Agriculture, Trade, and Consumer Protection (Wisconsin Department of)
BOD	biochemical oxygen demand
BMP	best management practice
CARPC	Capital Area Regional Planning Commission
DCRPC	Dane County Regional Planning Commission
DHFS	Wisconsin Department of Health and Family Services
DNR	Wisconsin Department of Natural Resources
EPA	United States Environmental Protection Agency
ERW	Exceptional Resource Water
ES	enforcement standard
HAP	hazardous air pollutant
LA	load allocation
LAL	limited aquatic life
LFF	limited forage fish
MGE	Madison Gas & Electric
mgd	million gallons per day
MMSD	Madison Metropolitan Sewerage District
N	nitrogen
ng/L	nanograms per liter
NRCS	Natural Resources Conservation Service
NRDC	Natural Resources Defense Council
NSWTP	Nine Springs Wastewater Treatment Plant
NTU	nephalometric turbidity unit
P	phosphorus
PAL	preventive action limit
POTW	publicly owned treatment works
TMDL	Total Maximum Daily Load
TN	total nitrogen
TSS	total suspended solids
USA	urban service area
WLA	waste Load Allocation
WPDES	Wisconsin Pollutant Discharge Elimination System
WQBEL	water quality based effluent limit
WQC	water quality criteria or criterion
WWSF	warm water sport fish
WWTP	wastewater treatment plant

TMDL: A TMDL is the maximum amount of pollutant loading that surface water can accept and still meet water quality standards including designated uses. The general TMDL equation is as follows:

$$TMDL = \sum WLA + \sum LA + MOS$$

Where ∑WLA is the sum of the waste load allocations for the pollutant, assigned to all point sources (a point source is generally any discharger that has a WPDES permit); ∑LA is the sum of the load allocations for the pollutant, assigned to all nonpoint sources (e.g., agricultural runoff, nonpermitted municipal runoff, and atmospheric deposition); and MOS is a margin of safety provided to account for uncertainties in the water quality monitoring, modeling, or science.

Microconstituents: Include but are not limited to pharmaceuticals, hormones, and other endocrine disrupting compounds and various bactericides.

Groundwater table: The groundwater table is the point where subsurface soils become saturated, and its surface is at atmospheric pressure. In this report, references to the decline in the groundwater table also encompass the decline in the piezometric surface of the partially confined aquifer below the water table aquifer.

Table 1.03-1
DNR Listing and Assessment Information for Possible Discharge Locations
MMSD 50-Year Master Plan

Wastewater Treatment Plant	Discharge Location (Downstream Waterbody)	Basin (Watershed)	Designated Use	303(d) Listed Impairments and (Potential Causes)	Key Recommendations from DNR Water Quality Management Plan Reports (1)
Nine Springs	Effluent Ditch to Oregon Branch of Badfish Creek	Lower Rock (Badfish Creek - LR 07)	LAL	Not listed	MMSD should continue monitoring.
Nine Springs	(Badfish Creek to CTH A)	Lower Rock (Badfish Creek - LR 07)	LFF	FCA (PCBs)	
Nine Springs	(Badfish Creek to Yarhara River)	Lower Rock (Badfish Creek - LR 07)	WWSF	FCA (PCBs)	Water quality has improved since the 1970s; better quality effluent.
Nine Springs	(Yahara River)	Lower Rock (Yahara River - Lake Kegonsa - LR 06)	WWSF	dhab; DO (phos; sed)	
Nine Springs	Badger Mill Creek	Sugar-Pecatonica (Upper Sugar River - SP 15)	LFF from former Verona STP to STH 69; being reclassified WWSF	Not listed	Water quality has improved since the 1970s. Control urban runoff from Verona and west Madison.
Nine Springs	Nine Springs Creek	Lower Rock (Yahara River - Lake Monona - LR 08)	WWFF	DO;temp (phos;sed)	Construction site runoff has contributed to sedimentation and lower water quality. Municipalities should enforce erosion control ordinances. Protect springs and wetlands.
Nine Springs	(Upper Mud Lake)	Lower Rock (Yahara River - Lake Monona - LR 08)	WWSF	Not listed	Boat traffic lowers water quality. Adopt and enforce no-wake ordinance.
Nine Springs	(Lake Waubesa)	Lower Rock (Yahara River - Lake Monona - LR 08)	WWSF	Not listed	Water quality has improved since MMSD effluent diversion. Additional monitoring. Control of aquatic plants.
Village of Oregon	Oregon Branch Badfish Creek	Lower Rock (Badfish Creek - LR 07)	LAL	Not listed	V. Oregon should upgrade and enforce erosion control ordinances.
Village of Oregon	(Oregon Branch Badfish Creek)	Lower Rock (Badfish Creek - LR 07)	LFF	Not listed	
Village of Oregon	(Badfish Creek to CTH A)	Lower Rock (Badfish Creek - LR 07)	LFF	FCA (PCBs)	Water quality is gradually improving.
City of Stoughton	Yahara River	Lower Rock (Yahara River-Lake Kegonsa - LR 06)	WWSF	dhab; DO (phos; sed)	Evaluate return of MMSD effluent to increase base flow. C. Stoughton and V. McFarland enforce erosion control ordinances.
City of Sun Prairie	Koshkonong Creek	Lower Rock (Upper Koshkonong Creek - LR 12)	LAL	DO; eutr;dhab; sed (phos; sed)	Acquire land for wetlands. Triennial water quality standards review. C. Sun Prairie enforce erosion control ordinance. Improve creek.
City of Sun Prairie	(Koshkonong Creek 2.5 miles from outfall)	Lower Rock (Upper Koshkonong Creek - LR 12)	WWSF	DO; eutr;dhab; sed (phos; sed)	Chemical water quality has improved.
City of Sun Prairie	(Koshkonong Creek)	Lower Rock (Lower Koshkonong Creek - LR 11)	WWSF	DO; eutr;dhab; sed (phos; sed)	Aquire land for wetlands.
City of Sun Prairie	(Lake Koshkonong)	Lower Rock (Lower Koshkonong Creek - LR 11)	WWSF	DO; eutr;dhab; sed (phos; sed)	High nutrient and sediment loads, poor water quality. Form lake district. Improve public access. Additional monitoring.
Mendota (North Madison)	Yahara River	Lower Rock (Yahara River - Lake Mendota - LR 09)	WWSF	Not listed	Aquire lands for public access.
Mendota (North Madison)	(Lake Mendota)	Lower Rock (Yahara River - Lake Mendota - LR 09)	WWSF	FCA (PCBs)	Municipalities should enforce erosion control ordinances. Control of aquatic plants. Aquire land for wetlands and access.
Sugar River	Sugar River	Sugar-Pecatonica (Upper Sugar River - SP 15)	CW/ERW	Not listed	Water quality has improved since the 1970s and is good except for fecal coliform. Groundwater diversion poses a threat to base flow. Control urban and agricultural runoff.
Sugar River	(Lake Belle View)	Sugar-Pecatonica (Upper Sugar River - SP 15)	WWSF	Not listed	Shallow reservoir with poor water quality.

Notes: 1 Non-binding or Early Planning Recommendations from Wisconsin DNR, 2001 (Lower Rock Basin) and 2006 (Upper Sugar River Watershed)

Abbreviations:
FCA = fish consumption advisory impairment
PCBs = polychlorinated biphenyls are a potential cause of the indicated impairment
DO = dissolved oxygen impairment
Temp = temperature impairment
phos = phosphorus is a potential cause of the indicated impairment
sed = sediment is a potential cause of the indicated impairment
eutr = eutrophication impairment
dhab = degraded habitat impairment

Table 1.03-2
Potential Effluent Limits for Possible Discharge Locations
MMSD 50-Year Master Plan

Wastewater Treatment Plant	Discharge Location (Downstream Waterbody)	Designated Use	Minimum Dissolved Oxygen (mg/L)	BOD (1) Monthly (mg/L)	BOD (1) Weekly (mg/L)	TSS (1) Monthly (mg/L)	TSS (1) Weekly (mg/L)	Ammonia NItrogen Limits (mg/L)	Total Nitrogen Monthly Potential (2) (mg/L)	Fecal Coliform Monthly (No/ 100 mL)	Phosphorus Current Monthly (mg/L)	Phosphorus Potential Monthly (3) (mg/L)
Nine Springs	Effluent Ditch to Oregon Branch of Badfish Creek	LAL	5	19	20	20	23	Yes	8	400	1.5	0.075
Nine Springs	(Badfish Creek to CTH A)	LFF										0.075
Nine Springs	(Badfish Creek to Yarhara River)	WWSF										0.075
Nine Springs	(Yahara River)	WWSF										0.105
Nine Springs	Badger Mill Creek	LFF from former Verona STP to STH 69; being reclassified WWSF	5	N/A	7/16	10/16	N/A	Yes	8	400	1.5	0.075
Nine Springs	Nine Springs Creek	WWFF	7	5	5	5	5	Yes	8	400	1.5	0.075
Nine Springs	(Lake Waubesa)	WWSF										0.040
Village of Oregon	Oregon Branch Badfish Creek	LAL	4	20	20	20	30	Yes	8	400	1.5	0.075
Village of Oregon	(Oregon Branch Badfish Creek)	LFF										0.075
Village of Oregon	(Badfish Creek to CTH A)	LFF										0.075
City of Stoughton(4)	Yahara River	WWSF	6	25	33/40	30	40	Possible (At Least Max Day)	8	400	1.5	0.105
City of Sun Prairie	Koshkonong Creek	LAL	7	5/10	5/10	10	N/A	Yes	8	400	1.4	0.075
City of Sun Prairie	(Koshkonong Creek 2.5 miles from outfall)	WWSF										0.075
City of Sun Prairie	(Koshkonong Creek)	WWSF										0.075
City of Sun Prairie	(Lake Koshkonong)	WWSF										0.040
Mendota (North Madison)	Yahara River	WWSF	7	5	5	5	5	Yes	3	400	N/A	0.075
Mendota (North Madison)	(Lake Mendota)	WWSF										0.015
Sugar River (5)	Sugar River	CW/ERW	7	5	5	5	5	Yes	3	400	N/A	0.075
Sugar River	(Lake Belle View)	WWSF										0.040

Notes:
1 Lower limit applies during summer. Limits shown as 5 mg/L are estimates based on meeting "background" concentrations.
2 Nitrogen limits shown are estimated technology-based limits; Mendota and Sugar River limits assume a higher level of TN removal will be required at these locations.
3 Phosphorus potential limits are based on proposed P criteria in draft NR 102 with no allowance for mixing and dilution. Actual limits will likely be higher.
4 Stoughton limits shown are based on current 1.65 mgf DAF; BOD limits shown are actually CBOD.
5 A new Sugar River discharge would need to meet background water quality because of antidegradation requirements for ERW waters.

Technical Memorandum No.6

Date: October 8, 2008

To: Madison Metropolitan Sewerage District

From: Steve McGowan, P.E., BCEE
Project Manager, Malcolm Pirnie, Inc.
Eric Wang, P.E.
Project Engineer, Malcolm Pirnie Inc

Subject: 50-Year Master Plan
TM-6: Scenario Planning Workshops (Final)

Project No.: MMSD No. 8425001
MPI No. 6100-001

1. Purpose

This memorandum documents the two scenario planning workshops held with key MMSD staff on June 5, 2008 and the Technical Advisory Committee (TAC) on June 6, 2008. The workshops used a scenario planning process to identify factors and uncertainties that could potentially impact the District during the 50 year master planning period, with a focus on the far end of the planning period. Information gained through this process will help the District continue to provide high quality services throughout the 50 year planning period.

2. Scenario Planning Description

Scenario planning is a predictive modeling technique used for risk analysis and planning policy creation. Scenario planning identifies probable outcomes that may result from a combination of factors/planning variables and their associated uncertainties. One of the greatest values of scenario planning lies in its articulation of a common future view to enable coordinated decision-making and action. Though scenario planning does not predict the future, it enables the user to prepare for future outcomes and to identify actions that need to occur to achieve desired outcomes.

The technique grew out of defense planning in the 60's and 70's and was a key element in the successful positioning of Royal Dutch Shell after the Arab oil embargo of the early 70's. Scenario planning has since been successfully used in both the public and private sectors to create situation-specific "alternative futures" while systematically accounting for future uncertainty. Scenario planning typically includes the following steps:

1. Frame the core planning questions

   The central questions or issues that will be addressed are identified through a brainstorming session with the planning group. The planning group then discusses the various issues and arrives at consensus agreement on the central issue(s) that needs to be addressed.

2. Identify driving forces

   A second brainstorming session is held to generate a list of driving forces that have a bearing on the central questions. The goal at this stage is to initially capture all ideas without trying to gauge their relative importance. Many of the driving forces relevant to the MMSD master planning process were identified in the Planning Variable Identification Workshop, held on May 19, 2008, while others became evident in discussions held during the scenario planning workshops.

3. Identify critical uncertainties

   Once the driving forces have been identified, the planning group evaluates each driving force based on two factors: 1) importance relative to the central issue(s); and 2) the associated level of uncertainty. The driving forces of greatest interest are those that are both very important and highly uncertain.

4. Develop scenarios

   Two critical driving forces are used to create a matrix of possible scenarios. This is accomplished by identifying the polar extremes of each critical driving force. The uncertainties of those driving forces are not viewed as representing a range or spectrum of relative values. They are instead viewed as end-point extremes. The critical uncertainties are then used to create two-dimensional matrices. The quadrants defined by the combinations of the critical uncertainties are the possible future scenarios to be evaluated.

5. Map paths to each scenario

   Each characterized scenario is a future scenario that could occur. The planning group plots a pathway to each of these scenarios based upon its specific characteristics and issues. The pathway includes individual elements such as public, political, and research/technological programs as well as various construction projects that need to be sequenced over time to achieve the envisioned future scenario.

6. Identify common elements

   The pathways are developed independently from one another and are based solely on realizing each specific scenario. Nonetheless, similarities and overlaps do occur among the individual pathways developed. There are projects and programs that are present on all or many of the individual scenario pathways. This commonality indicates that such projects and programs will be useful under a wide range of possible futures. As a result, such elements are more likely to be viable as the future unfolds.

7. Screen & align alternatives

   In this step, alternatives associated with each scenario pathway are identified. Rating criteria will be generated and used in ranking these alternatives.

8. Develop signposts & triggers and implementation plan

   In this step, signposts and trigger mechanisms along the scenario pathways are generated. The implementation plan for the planning period is also developed.

3. Scenario Planning Workshops

A) General

The workshops began with an introduction to scenario planning techniques. This was followed by a presentation on factors and/or trends that are currently impacting the wastewater industry. These include:

1. Population growth

   The population of the U.S. is projected to grow by 28.9% from 282,125,000 to 363,584,000 from Year 2000 to Year 2030. The Dane County and the City of Madison populations are projected to expand by 36.0% and 27.3% respectively during the same period. Population census shows that people are now living 20 years longer than in 1970s', which implicates delayed retirement, more age-diverse workforce, and potential skills shortages.

2. Political climate

   Political climate is getting more complex with the surge of NGO advocacy, rise of weblogs, larger role of public participation, etc. It implicates the needs to manage constituencies, develop relationships, understand where the public stands on issues, improve financial and capital improvement project transparency, etc.

3. Environmental concerns

   With increasing concerns on global climate change, the evaluation of greenhouse gas emissions and carbon footprints of wastewater treatment facilities may become regulated in the future.

4. Increasingly stringent environmental regulations

   With increasing public concerns on water resource protection, more contaminants may be regulated by regulatory agencies in the future. Wastewater effluent discharge limits may become more stringent.

5. Financial constraints

   Existing infrastructure replacement and repair, energy volatility, less available federal subsidies, and resistance for raising wastewater rates will create financial constraints to the wastewater treatment agencies. It raises the needs for communicating to stakeholders, optimizing utility efficiency, documenting infrastructure/rate needs, etc.

6. Total water management

   Total water management requires considering entire water cycle as an integrated system. The current challenges include degradation of water resources, farm land loss, etc. The strategies include wastewater facility life cycle analysis, watershed/stakeholder engagement, promoting using stormwater/wastewater as resource, water supply diversification, water conservation, environmental trade-offs, etc.

7. Customer service

   The wastewater agencies will be more proactively in educating the water customers, understanding the needs and expectations of costumers, and cultivating sustainable approaches in utilizing water resource.

8. Changes in the workforce

   The U.S. workforce is getting older, multi-generational, more female and ethnically diverse. Professions in environmental engineering field are becoming more popular, information technology skills become more necessary for qualified workers. It implicates that future wastewater jobs should be able to accommodate workforce with generational differences, provide flexibility for workforce, emphasize training and apprenticeship programs, and accommodate part-time retirees.

9. Technology

   The current trends for technologies are smaller, cheaper, faster and more mobile. Automation and remote monitoring are the trends for future wastewater treatment facilities.

10. Energy

    Energy pressure may create incentives for implications of more energy-efficient treatment facilities to lower costs and interruption risk. Sustainable energy such as solar power and wind power will play bigger role in future energy market and wastewater water facilities.

11. Increasing risk

    Risks to wastewater utility are increasing due to IT and physical security issues, climate change, workforce shortage, litigation, etc. The strategies include reassessing system vulnerability to attack (physical and IT), developing specific risk management strategies for climate change, succession planning, and public outreach in the event of a terrorist attack.

A list of planning variables/driving forces identified by the consultants, MMSD staff and TAC members was then reviewed, with additional variables identified during the group discussion. Workshop attendees then ranked the variables based on level of uncertainty and importance. Top-ranked variables/driving forces were then used to generate scenario matrices, which served as the basis for identifying scenario implications.

B) Ranking of Planning Variables and Driving Forces

The following 24 planning variables and driving forces were identified and discussed by the attendees of the two workshops. The first nine variables were identified and discussed in TM 4, Planning Variables. The remaining variables were added during this workshop.

1. Location of Treatment Plant
2. Biosolids Management
3. Effluent Discharge and Reuse
4. Regulatory Trends
5. Storm Water Management
6. Environmental Impacts
7. Future Flow/Load Projections
8. Construction/Operational Costs
9. Public Acceptance
10. Workforce
11. Energy
12. Commodities
13. Water Rights
14. Competitiveness
15. Other WWTPs
16. New Technologies/Solutions
17. Population Shift
18. Ability/Willingness to Pay by Customers
19. Protect the Lakes
20. Area Growth and Distribution
21. Cost of Repair/Replacement of Existing Facilities
22. Change in Leadership/Governance
23. Total Watershed Management
24. Special Water Needs by New Industries

These variables and driving forces were ranked for their levels of uncertainty and importance by workshop attendees according to the following scale:

Level of Uncertainty: 1-5

Very Certain – 1

Certain – 2

Somewhat Certain -3

Uncertain – 4

Very Uncertain – 5

Level of Importance: 1-5

Very Important – 5

Important – 4

Less Important – 3

Partially Important – 2

Not Important – 1

The variables ranking results for both workshops are presented in Figures 3-1 and 3-2. The variables that have the highest level of importance and uncertainty are highlighted in yellow boxes.

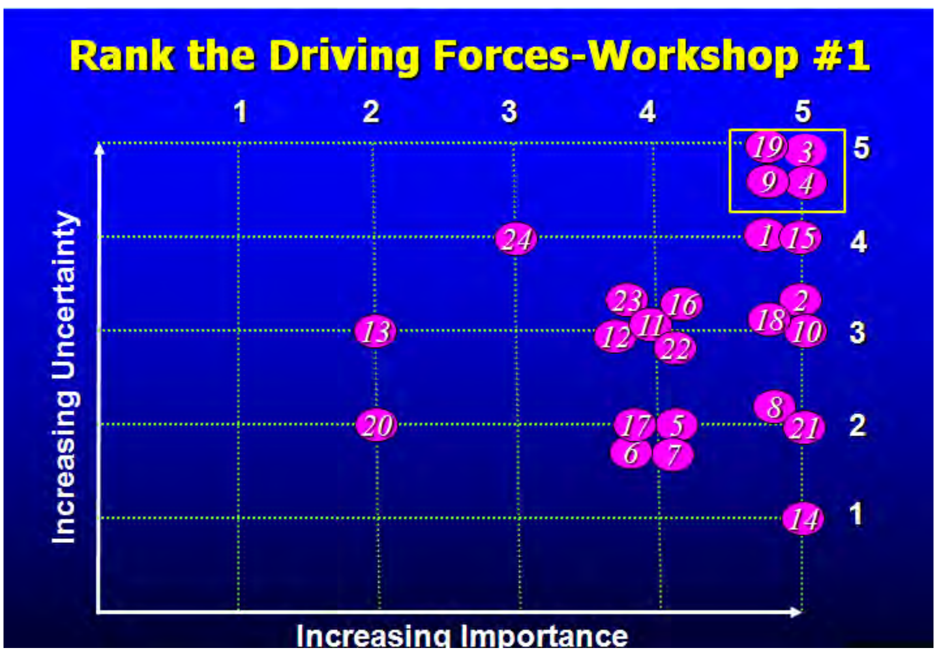


Figure 3-1. Ranking of Importance and Uncertainty of Planning Variables – Workshop #1

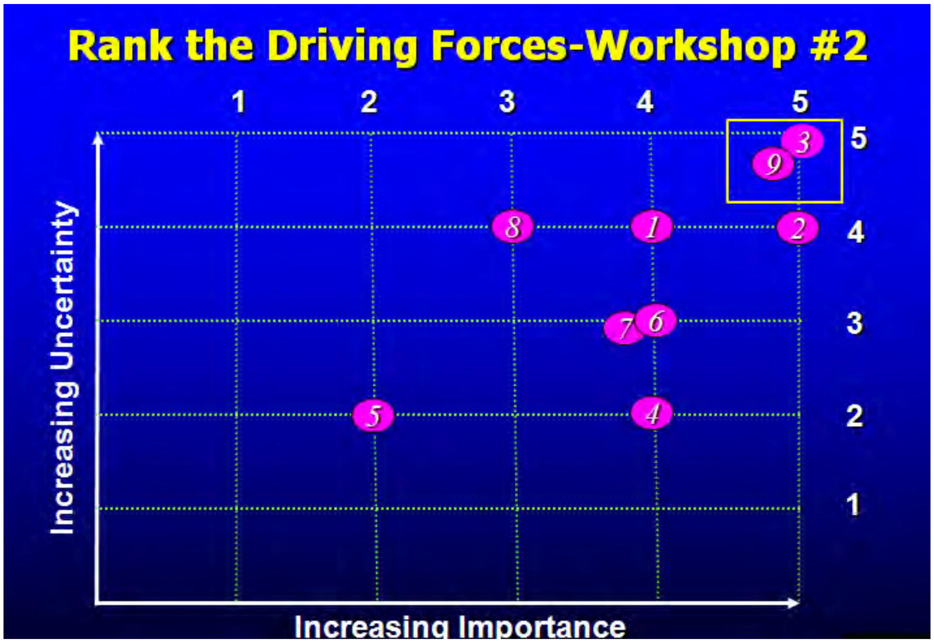


Figure 3-2. Ranking of Importance and Uncertainty of Planning Variables – Workshop #2

Although two groups of workshop attendees ranked each variable slightly differently, the following variables and driving forces were selected by both groups for the highest level of uncertainty and importance:

1. No. 3 - Effluent Discharge and Reuse

   Currently MMSD pumps plant effluent to Badger Mill Creek via a 10 mile force main and to Badfish Creek via a 5 mile force main. Peak flows exceeding the plant's pumping and equalization capacities overflow to Nine Springs Creek. Increasing regulatory and operational pressures, including the potential for more stringent effluent limits, high energy requirements for pumping, concerns on mitigation of inter-basin water transfers, ground water preservation, etc. could impact future practices. Many of the future options will require the involvement of other water resource managers, such as the Capital Area Regional Planning Commission, the Wisconsin Department of Natural Resources, the Dane County Lakes and Watershed Commission and local drinking water and storm water utilities. Local involvement of special interest groups such as watershed advocacy groups, fishermen, conservation organizations, environmental organizations, along with the general public, will be necessary to implement future reuse options and alternate discharge locations.

2. No. 4 - Regulatory

Future regulatory requirements could significantly impact MMSD's planning and operations over the planning period. Areas of particular importance include: phosphorus criteria; total nitrogen criteria; chlorides, mercury and other toxics, thermal standards, microconstituents in effluent and biosolids; water quality assessments; Rock River TMDL development; water balance issues, groundwater rules for discharges to land and subsurface, and requirements for land application of biosolids.

3. No. 9 - Public Acceptance

Public acceptance will play an important role as the District evaluates effluent reuse opportunities; construction of regional treatment plants; construction of un-manned neighborhood treatment plants; and alternative biosolids management options.

4. No. 19 - Protect the Lakes

Madison's lakes are the most visible and highly regarded resource of the local ecological system. Therefore, protecting the lakes is one of the top concerns in the master planning process.

4. Development of Scenarios

Based upon the selected planning variables and driving forces, three scenario matrices were developed in the two workshops for group discussions (Figures 4-1, 4-2, and 4-3). The variable "No. 19 - Protect the Lakes" is dependent on the effluent discharge locations, biosolids management alternatives and other planning variables, therefore was not used as an independent planning variable in the scenario matrices. Workshop attendees discussed each of these scenario matrices.

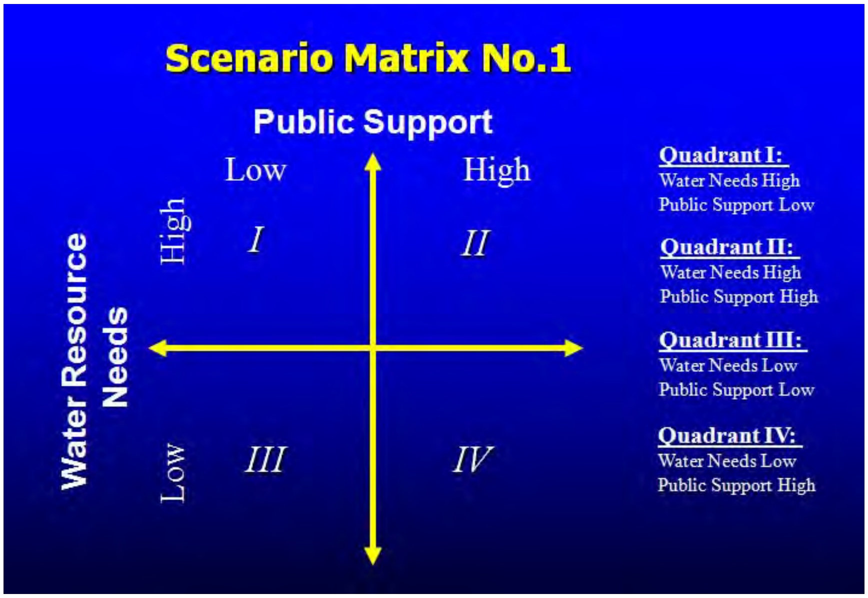


Figure 4-1. Scenario Matrix No. 1

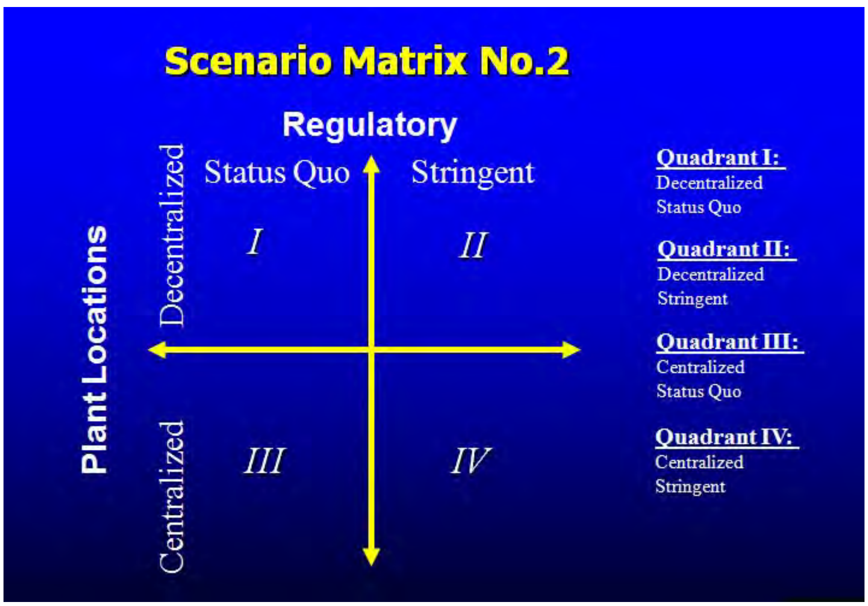


Figure 4-2. Scenario Matrix No. 2

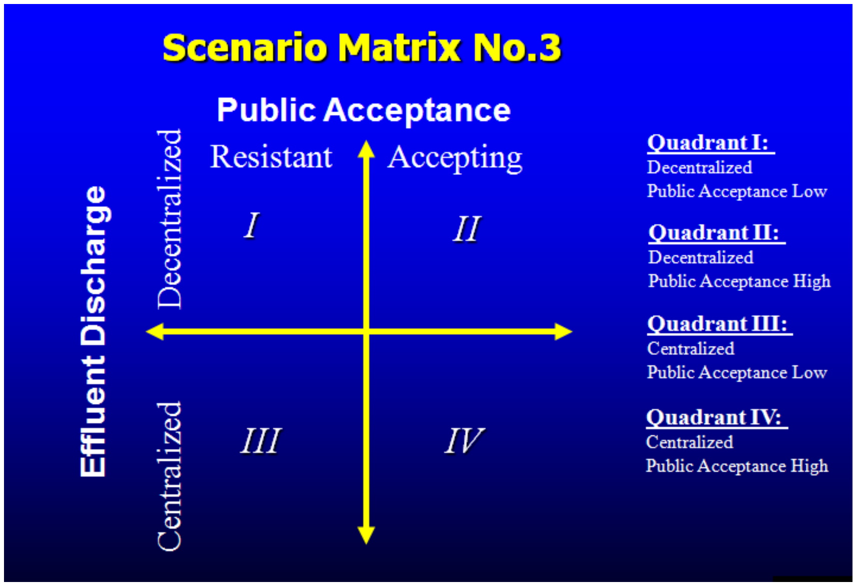


Figure 4-3. Scenario Matrix No. 3

5. Planning Scenario Implications

The following implications were identified by the workshop attendees for different planning scenarios:

A) Water Resource Needs High and Public Support Low

- Due to the low public support, it will be necessary to target potential customers for recycled effluent.
- Management of the District's effluent discharges and effluent reuse alternatives will need to be adaptive due to low public support.
- Groundwater recharge could be a viable option.
- Incremental implementation of effluent reuse alternatives will be necessary to gain public acceptance.
- It will be important to identify the lead agency for overall water resources management in the District's service area.
- It will be important to develop good relationships with the other water sector agencies, such as the Capital Area Regional Planning Commission, the Dane County Lakes and Watershed Commission, the Wisconsin DNR, and the local water and storm water utilities.
- It will be important to develop a good public education program related to reuse.
- It will be important to monitor the developments in the technical fields associated with water reuse.
- The District should identify the target environmental groups that would have an interest in water reuse and engage these groups in the water resource management discussions.

- It will be important to develop a good public relations program to better understand the information needs of the public and develop consensus/support for goals and major program elements.
- The construction of demonstration facilities to determine costs and show benefits of effluent reuse alternatives may be effective under this scenario.
- More efforts will be needed to convince the public and regulatory agencies that effluent reuse alternatives are protective of the public health and the environment.
- The District will need to establish its credibility in implementing effluent reuse alternatives.

B) Water Resource Needs High and Public Support High

- Under this scenario the District can be more selective in which reuse alternatives are implemented since infrastructure costs associated with water reuse can be high and the energy consumption for some alternatives can be high.
- Training and skills development for the new process operating procedures will be needed in implementing effluent discharge and reuse alternatives.
- Additional land may need to be purchased to accommodate additional treatment and conveyance facilities.
- A list of potential customers for recycled water will need to be developed and users identified that have needs for large volumes of recycled water.
- Seasonal water demand needs to be addressed when considering effluent reuse alternatives such as golf course or crop irrigation.
- Contingency plans will need to be provided in the effluent reuse systems to address changes in demands for reclaimed water.
- Public education will still be important under this scenario.
- Water conservation efforts may result in reduced demand for reclaimed water.
- The District will need to establish its credibility in implementing effluent reuse alternatives.

C) Water Resource Needs Low and Public Support Low

- The need for effluent reuse will require justification.
- Public education to cultivate public acceptance for new effluent discharge locations and reuse alternatives will be necessary.
- If water reuse needs are low, it will be more important to promote water conservation efforts to avoid the need for increasing the capacity of the current effluent conveyance system.
- It will be important to monitor regulatory trends and their impacts on the effluent discharge and reuse alternatives.
- The construction of demonstration facilities to determine costs and show benefits of effluent reuse alternatives will be necessary under this scenario.

- This quadrant best represents the current condition and would require the lowest level of operational changes.

D) Water Resource Needs Low and Public Support High

- This quadrant represents the best scenario for the District in the near term.
- A good public education program related to wastewater treatment and water reuse will still be important.
- If water reuse needs are low, it will be more important to promote water conservation efforts to avoid the need for increasing the capacity of the current effluent conveyance system.
- The construction of demonstration facilities to determine costs and show benefits of effluent reuse alternatives will be desirable under this scenario.
- With a higher level of public support, the promotion of green communities may be an effective approach to promoting water reuse.
- It will be important to monitor regulatory trends and their impacts on the effluent discharge and reuse alternatives.

E) Decentralized Treatment and Regulatory Status Quo

For Constructing New Satellite Plants

- The public may be more resistant to the satellite wastewater treatment facilities in their neighborhood if regulations are not felt to be stringent enough.
- Additional staff will be needed to operate the satellite plants. With changing demographics, developing a work force for the satellite plants may be difficult.
- Operation and maintenance of the overall sewerage system will be more complex. Additional redundancy will be needed.
- Lands will be required for satellite treatment plants.
- Higher levels of treatment than currently provided at the Nine Springs plant may be required at new satellite plants.
- Biosolids treatment and disposal operations will be more complex.

For Rehabilitating and Expanding Existing Plants in Nearby Municipalities

- Reaching service agreements with the nearby municipalities could be challenging.
- Initial costs could be high compared to the centralized treatment model.
- Politics (issues not directly related to the provision of sewerage service) may impact the ability to implement projects with nearby municipalities.
- The issue of ownership and control of the treatment plants may be a challenge.
- There may be major construction cost savings under this scenario.

F) Decentralized Treatment and Regulatory Stringent

For Constructing New Satellite Plants

- Higher levels of treatment than currently provided at the Nine Springs plant may be required at new satellite plants, especially if more stringent requirements are in place.
- Need to identify potential industrial customers of effluent reuse.
- The construction of satellite plants would reduce the needs for modifications at the Nine Springs WWTP and in the existing conveyance system.
- Construction costs may be high compared to the centralized treatment model, especially if more stringent requirements result in the construction of additional treatment processes.
- Although the public may still resist locating a satellite treatment plant in their neighborhood, they may be more receptive if regulations are felt to be stringent enough.
- Additional staff will be needed to operate the satellite plants. With changing demographics, developing a work force for the satellite plants may be difficult, especially if more stringent requirements result in the use of newer and more complex technology.
- It may be difficult to obtain ownership of suitable sites for satellite plants, especially if more stringent requirements result in the construction of additional treatment processes that require larger tracts of land.
- Biosolids treatment and disposal operations will be more complex, especially if more stringent requirements result in the construction of additional treatment processes.

For Rehabilitating and Expanding Existing Plants in Nearby Municipalities

- The determination of service charge rates could be challenging.
- Biosolids treatment and disposal operations could be more complex, especially if more stringent requirements result in the construction of additional treatment processes.

G) Centralized Treatment and Regulatory Status Quo

- Available lands in the proximity of the Nine Springs WWTP could be a constraint to the plant expansion.
- Effluent volumes and loadings to Badfish Creek and Badger Mill Creek could be an issue if the plant keeps expanding.
- New infrastructure at Nine Springs will be needed sooner than if flow diversion to satellite facilities or to other existing treatment plants is implemented. This includes new effluent conveyance facilities.
- Lower capital costs can be achieved due to the economy of scale.
- There is a potential for higher pumping costs than if flow was diverted to satellite plants or to other existing treatment plants.

- The older technology used at Nine Springs may result in higher energy consumption.

H) Centralized Treatment and Regulatory Stringent

- Multiple options for high quality effluent discharge and water reuse become possible.
- There is a potential to discharge to the lakes.
- Use of newer technologies may lower manpower requirements.
- A higher quality effluent that could be returned to Lake Waubesa would reduce or eliminate the need for effluent pumping. This would also simplify operation and maintenance.
- This scenario would result in centralized biosolids treatment and disposal facilities.
- Available lands in the proximity of the Nine Springs WWTP could be a constraint to the plant expansion, especially if more stringent requirements result in the need to construct additional treatment processes.
- Meeting stringent effluent limits can be challenging.
- New infrastructure at Nine Springs will be needed sooner than if flow diversion to satellite facilities or to other existing treatment plants is implemented. This includes new effluent conveyance facilities.
- The capacity of the current interceptors and pumping stations will need to be increased sooner than if flows were diverted to new satellite treatment plants or to other existing treatment plants.
- Effluent volumes to Badfish Creek and Badger Mill Creek could be an issue if the plant keeps expanding. However, with more stringent regulations, pollutant loadings may be less of a concern.

I) New Effluent Discharge/Reuse Locations and Resistant Public

- Educate the public and the other water sector agencies, including the regulators, to cultivate acceptance of new effluent discharge locations and reuse alternatives (Social marketing).
- Costs for all effluent discharge and reuse alternatives will need to be developed.
- It will be important to achieve win-win situations among multiple parties.
- The construction of demonstration facilities to determine costs and show benefits of effluent reuse alternatives will be necessary under this scenario.
- The District will need to develop effective partnerships in implementing effluent discharge and reuse alternatives.
- Due to public resistance, it will be necessary to target potential customers for recycled effluent.

J) New Effluent Discharge/Reuse Locations and Accepting Public

- Under this scenario the District can rely more on cost-effectiveness in locating new discharge locations and choosing which reuse alternatives to implement, rather than implementing the alternatives that have the least public resistance.
- When water becomes more valuable, the public will be more willing to accept effluent reuse. The demand for effluent reuse may be high.
- It will be important to establish and maintain the District's credibility in implementing water reuse alternatives to gain and keep public acceptance
- The demand for effluent reuse may be high.

K) Current Effluent Discharge Locations and Resistant Public

- If current effluent discharge locations are to be maintained and water reuse needs are low, it will be more important to promote water conservation efforts to avoid the need for increasing the capacity of the current effluent conveyance system.
- The capacity and capabilities of the current treatment and conveyance systems will need to be expanded earlier.
- Educate the public and the other water sector agencies, including the regulators, to cultivate acceptance of reuse alternatives (Social marketing).
- It will be important to establish the District's credibility in implementing water reuse alternatives to gain public acceptance.

L) Current Effluent Discharge Locations and Accepting Public

- Under this scenario the District can rely more on cost-effectiveness in selecting which reuse alternatives to implement, rather than implementing the alternatives that have the least public resistance.
- It will be important to establish and maintain the District's credibility in implementing water reuse alternatives to gain and keep public acceptance
- Although less important than in Quadrant 3, it will still be necessary to promote water conservation efforts.
- Although less critical than in Quadrant 3, the capacity and capabilities of the current treatment and conveyance systems will need to be expanded earlier.
- Effluent volumes and loadings to Badfish Creek and Badger Mill Creek could be an issue if the plant keeps expanding. It will be important to track new regulatory initiatives that could impact effluent quantity and quality levels for Badfish Creek and Badger Mill Creek.

6. Future Signposts and Triggers

Based on the results of the two workshops, signposts and trigger mechanisms were generated to provide MMSD the necessary “early warning” for preparing for future scenarios. The signposts and potential corresponding strategies are presented in Table 6-1.

Table 6-1 Signposts for Future Scenarios

No.	Signposts	Potential Strategies
1	Improvement in wastewater treatment technology for high quality effluent processes	• Discharge to Lake Waubesa, which would reduce effluent pumping costs and simplify operation and maintenance. • Discharge to Yahara River upstream of Lake Mendota to provide additional base flow • Increase effluent discharge to Sugar River to match the groundwater withdrawal from the watershed.
2	Local regional wastewater agencies show interest in joining MMSD. This could happen in the following scenarios: • More stringent future regulatory requirements make the small-scale local operations less cost-effective • Local agencies have financial or technical difficulties in meeting the higher discharge limits • The imbalanced inter-basin water transfer becomes a major concern and requires a regional solution and there is a consensus that MMSD is the appropriate agency to deal with the issue.	• Consider forming partnership with regional wastewater agencies • Determination of the provision of sewerage service structure and service charge rates • Negotiate to achieve win-win situations among multiple parties.
3	Imbalanced inter-basin water transfer becomes a major concern in the future	• A new Sugar River plant discharge to the confluence of the Sugar River and the Badger Mill Creek or/and headwater of the Sugar River will become more convincing. • Consider starting planning process for a Mendota Plant to provide additional base flow in the Yahara River upstream of Lake Mendota. • Increase effluent discharge to Starkweather Creek by constructing a new satellite treatment plant or conveying treated effluent from NSWTP to the area. • Expand the existing Sun Prairie WWTP and increase discharge to Koshkonong Creek.
4	Low public support for effluent reuse	• Target potential industrial effluent users. • Manage effluent discharges and reuse, be adaptive to different future scenarios. • Establish credibility with incremental implementation of effluent reuse alternatives • Identify the lead agency for overall water resources management in the area. Develop good relationships with other water sector

No.	Signposts	Potential Strategies
		agencies • Develop good public education program related to effluent reuse to convince the public and regulatory agencies that effluent reuse alternatives are protective for the public health and the environment. • Monitor the developments in the technical fields associated with effluent reuse • Identify the target environmental groups that would have an interest in water reuse and engage these groups in the water resource management discussions. • Construction of demonstration facilities to show benefits of effluent reuse alternatives and to determine capital and M/O costs.
5	High public support for effluent reuse	• Be selective in which alternatives to be implemented and to adopt the alternatives with high cost efficiency and environmental benefits. • Conduct training and prepare workforce for effluent reuse applications. • Purchase land for additional treatment and conveyance facilities. • Develop lists of potential customers for effluent reuse. • Address the seasonal demand variance for treated effluent. Provide contingency plans for effluent reuse systems.
6	Higher than projected peak flows due to increased precipitation and resulting higher rates of I/I and high groundwater levels	• Harden the conveyance system components to eliminate points of entrance for I/I. • Encourage sound management of collection systems in satellite communities • Increase the capacity of new and rehabilitated conveyance system components.
7	Water resource needs low due to: • Water conservation efforts • Lower than expected growth rate	• Delay construction of additional capacity for the conveyance system and treatment facilities. • Public education to cultivate public acceptance for new effluent discharge locations and reuse alternatives. • Monitor regulatory trends and their impacts on the effluent discharge and reuse alternatives. • Construction of demonstration facilities to determine costs and show benefits of effluent reuse alternatives.
8	Water resource needs high due to: • Higher than expected growth rate • Population shift	• Public education to cultivate public acceptance for new effluent discharge locations and reuse alternatives. • Construction of demonstration facilities to determine costs and show benefits of effluent reuse alternatives. • Conduct training and prepare workforce for effluent reuse applications. • Purchase land for additional treatment and conveyance facilities. • Develop lists of potential customers for effluent reuse. • Promote water conservation efforts

No.	Signposts	Potential Strategies
		• Implement programs to reduce inflow/infiltration, which will delay the need for major capital improvement projects required to expand the capacity of the conveyance system.
9	High regulatory requirements	• Upgrade the existing treatment facilities and effluent pumping system. • Diversify the treated effluent discharge locations and effluent reuse alternatives. • Diversify the biosolids utilization alternatives. • Take proactive action to identify alternative users for biosolids other than agricultural crop land. The production of a Class A biosolids material is critical to assure that a full range of alternate uses can be investigated. • Construction of new satellite treatment plants with high quality effluent processes

Technical Memo for

Madison Metropolitan Sewerage District, Wisconsin

Technical Memo 7

50-Year Master Plan

mmsd

MALCOLM PIRNIE

STRAND ASSOCIATES, INC.®
ENGINEERS

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SECTION 1
OVERVIEW

1.01 TECHNICAL MEMO OVERVIEW

This Technical Memo defines projects and groups them into potential alternatives that provide different approaches to meet the needs of the Madison Metropolitan Sewerage District (MMSD) during the next 50 years. The base plan for all alternatives is conveyance of all wastewater to the Nine Springs Wastewater Treatment Plant (NSWTP) for treatment with effluent pumped to the two existing discharge locations: Badfish Creek and Badger Mill Creek. Replacement or relief facilities for those components reaching their capacity between 2010 and 2060 are sized for the appropriate service life for the type of facility. For example, pumping station structures are sized for the projected 100-year capacity needs while interceptors are sized for the projected 75-year needs. In the conveyance system, the more conservative design flows, based on the maximum projected flows determined by the CARPC, are used. These flows are based on the full build-out of each community's service area based on the community's Comprehensive Plan. At the NSWTP, design flows are based on the extrapolation of values determined by the CARPC using the DOA projected population data for Dane County. This results in lower design flows. The accumulated design flows in the conveyance system total 70 million gallons per day (mgd) in 2060 compared to a value of 60 mgd based on projected DOA population data. This planning approach is used since the service lives of interceptors and pump station structures is 75 to 100 years, and it is uncertain where in the District service area higher growth will occur. Economies of scale in the construction of conveyance facilities make this the more cost-effective approach. At the NSWTP, incremental capacity additions are made to address twenty years of growth even though the structures will have service lives of 75 to 100 years or more. The DOA-based values reflect the more probable total population and flow data scenarios when analyzing future design conditions at the NSWTP.

The District has the capability to discharge 75 mgd of treated effluent to Badfish Creek and 3.6 mgd of treated effluent to Badger Mill Creek. Sustained flows exceeding 78.6 mgd receive ultraviolet (UV) disinfection up to 115 mgd and are then routed to a 50-million-gallon storage lagoon. Once the storage lagoon is full, effluent is discharged to Nine Springs Creek and ultimately to the Yahara River upstream of Lake Waubesa. Flow from the lagoon is pumped back to the NSWTP for treatment when the capacity is available.

An analysis of the projected needs for wastewater treatment based on treating all wastewater generated in the District at the NSWTP is presented in Technical Memo 1–Review of Existing Treatment Facilities. Flow and loading projections for this treatment plant analysis are presented in Technical Memo 2 based on information prepared by the Capital Area Regional Planning Commission (CARPC) and contained in the 2008 MMSD Collection System Evaluation. An analysis of existing conveyance facilities to convey wastewater from MMSD users to the NSWTP is contained in Technical Memo 3–Conveyance Facilities Analysis (CFA). The information contained in these technical memos is the "baseline" option for the District. Regulations impacting the existing operation and alternative projects are reviewed in Technical Memo 5–Regulatory Trends.

Technical Memo 8 presents the development of the ranking criteria and methods for evaluating alternatives for the Master Plan. Technical Memo 9 presents the ranking and detailed evaluation of alternatives for the Master Plan.

1.02 DEFINITIONS AND ABBREVIATIONS

A. Abbreviations

Following are definitions for the acronyms and abbreviations used in this technical memo.

$_7Q_{10}$	10-year, 7-day low flow
ADF	average daily flow
BOD_5	biochemical oxygen demand
CARPC	Capital Area Regional Planning Commission
CFA	Conveyance Facilities Analysis
CTH	County Trunk Highway
fps	feet per second
gcd	gallons per capita per day
MDC	Madison Design Curve
mgd	million gallons per day
MMSD	Madison Metropolitan Sewerage District
NSVI	Nine Springs Valley Interceptor
NSWTP	Nine Springs Wastewater Treatment Plant
PHF	Peak Hourly Flow
PS	pumping station
WWTP	wastewater treatment plant

B. Definitions

Madison Design Curve–Peak hourly flow (PHF) factor developed in the 1961 Greeley and Hansen Report on Sewerage and Sewage Treatment. The formula is applied to average daily flows (ADFs) in the range of 1 mgd to 20 mgd as follows: $PHF = (ADF)^{.842} x\ 4$. All PHFs presented in this technical memo were prepared this way unless specifically noted otherwise. For ADFs less than 1 mgd, the peaking factor is 4. For average daily flows greater than 20 mgd, the peaking factor is 2.5.

SECTION 2
DEVELOPMENT OF ALTERNATIVE PROJECTS

2.01 DEVELOPMENT OF MASTER PLAN PROJECTS

The current MMSD model is conveyance of all wastewater to a centralized treatment facility (NSWTP) for treatment and discharging the treated effluent to Badfish Creek (75 mgd) and Badger Mill Creek (3.6 mgd). Flows exceeding 78.6 mgd, once the 50-million-gallon effluent storage lagoon is full, are discharged to Nine Springs Creek. There may be advantages to altering this model by decentralizing treatment through the construction of satellite treatment plants or altering the conveyance system to route wastewater from certain parts of the service area to an existing municipal treatment plant in a nearby community. These advantages could include lower capital costs in the conveyance system and at the NSWTP, reduced operational costs associated with pumping the wastewater and effluent, and environmental benefits realized by returning the effluent closer to the original source of the water.

Implementation of projects to decentralize treatment will take a decade or longer to implement, either because of issues related to the receiving water into which effluent from the satellite plant would be discharged, or due to the length of time it would take to reach agreement with a community with an existing treatment plant. Due to these constraints and the fact that the District has immediate needs to address capacity and condition issues in the conveyance system, there are few near-term decentralization projects that can achieve conveyance system construction cost savings. Projects that address capacity needs of the Nine Springs Valley Interceptor (NSVI) are the exception. Additional capacity in the NSVI will be required in approximately ten years. This would allow sufficient time to implement a decentralized project in this part of the District's service area. Such a project would have the highest potential to produce capital cost savings in the conveyance system and at the NSWTP where future capacity expansions could be avoided, delayed, or reduced in size. Conveyance capacity needs on the east side of the District are more immediate, and thus decentralized projects in this part of the service area will generally be more costly overall since the opportunity to achieve near-term conveyance system construction cost savings will not be available.

The following key principles were used to develop the projects presented in this Technical Memo.

1. PHFs to Badfish Creek will not exceed the rated 75 mgd of the effluent force main.

2. The growth rate projections for the conveyance system which include an uncertainty factor to reflect the unknowable location and timing of growth will be used for determining when loadings to various conveyance components will reach the design capacity.

3. The growth rate projections based on extrapolating the Department of Administration data for Dane County will be used for determining when loadings to various treatment processes at the NSWTP reach design capacity.

4. For evaluation purposes, it is assumed that the NSWTP will need to be upgraded to achieve a lower effluent phosphorus concentration in 2020 and a lower total nitrogen effluent concentration in 2030. Also, the solids processing facilities at the NSWTP will require capacity expansion in 2030.

5. Discharge at Badger Mill Creek at a minimum of 3.6 mgd will be maintained for all alternative projects.

6. ADFs are based on the 2008 MMSD Collection System Evaluation as prepared by CARPC. Peak hourly flows were based on ADF times the Madison Design Curve (MDC).

Projects were developed based on addressing the projected future needs for either the current NSWTP (Technical Memo 1) or the existing MMSD conveyance facilities (Technical Memo 3). Projects are presented for the east side of the MMSD system (Service areas for Pumping Stations (PSs) 6, 7, 9, 10, 13, and 14) and the west side of the MMSD system (Service Areas for PSs 1, 2, 3, 4, 5, 8, 11, 12, 15, 16 and 17). Figure 2.01-1 shows the service areas included in the east and west sides and the surface water drainage divides. In an alternative operating mode, the service area for PS 1 would be located in the east side of the MMSD. Under the current operating mode, the majority of flow for PS 1 is pumped to PS 2, which is located on the west side of the MMSD system. On the east side of the MMSD system, only PS 7 discharges to the NSWTP. West side pumping stations discharging to NSWTP include PSs 2, 3, 4, 8, and 11.

Timing for implementation of the projects is dependent on the needs of the existing facilities identified in either Technical Memo 1 (Wastewater Treatment), Technical Memo 3 (Conveyance Facilities), or to improve effluent quality as described above. In some instances, capacity expansion will be necessary significantly sooner than the decentralization project could be developed and implemented. This will be noted in the discussion for each of the projects.

Projects are organized into near-term projects and long-term projects. Near-term projects are those that would address the need for capacity expansion in the conveyance system required in the next ten to fifteen years. Long-term projects are those which, while still viable, cannot be implemented prior to the time the collection system capacity improvements would be required. Examples of long-term projects would include those that would discharge highly treated effluent to Lake Mendota or Lake Monona, effluent reuse projects that would be primarily driven by the economic need to reuse water, or turf irrigation projects on a larger scale that would require the development of a distribution network for the highly treated effluent. Near-term projects are discussed in Section 3 of this Technical Memo while long-term projects are discussed in Section 4 of this Technical Memo.

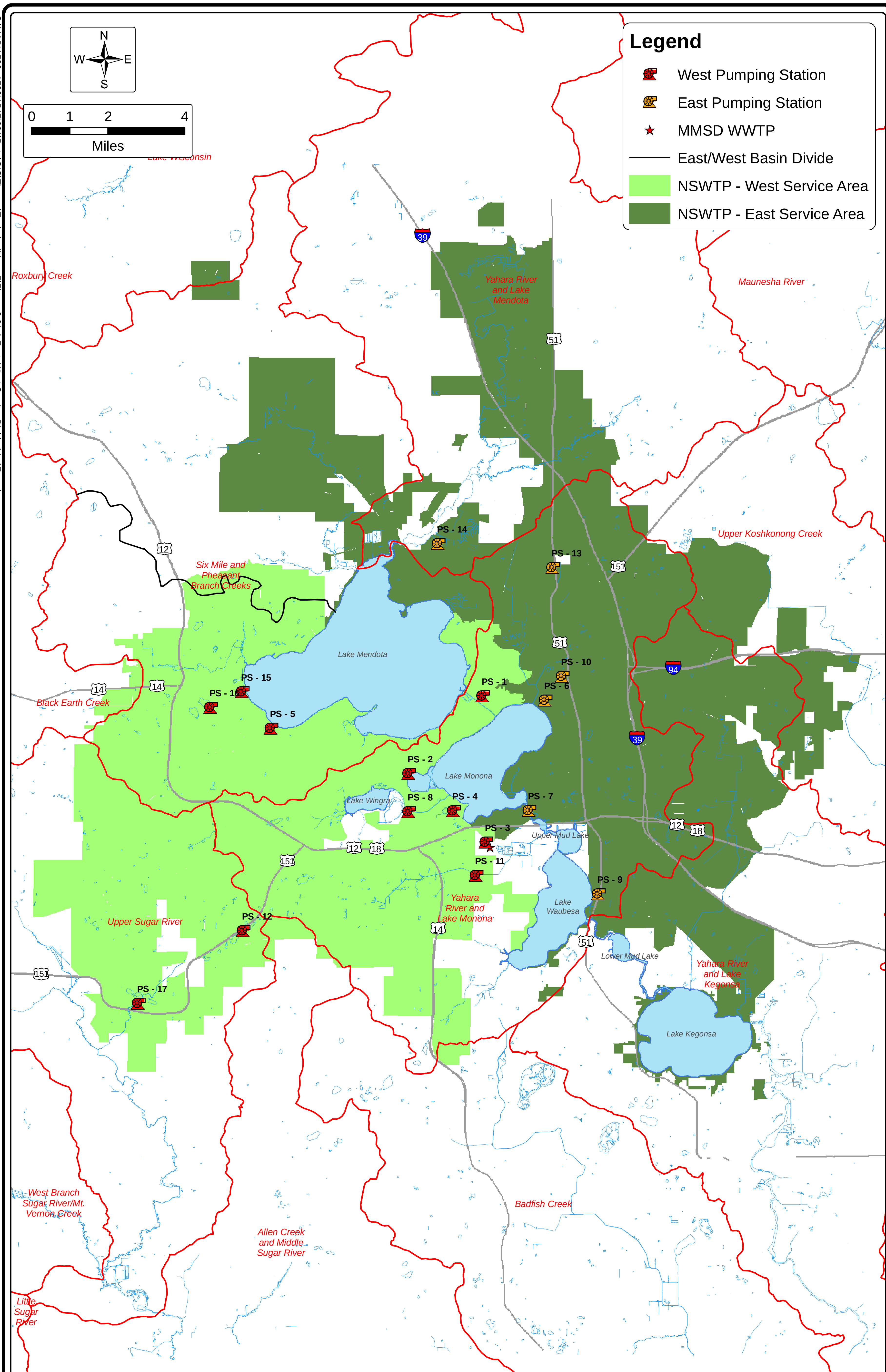

FIGURE 2.01-1
1547.001

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MALCOLM PIRNIE

MMSD EAST/WEST SERVICE DIVIDE

TECHNICAL MEMO 7
ALTERNATIVES DEVELOPMENT
MADISON METROPOLITAN SEWERAGE DISTRICT

2.02 WEST SIDE PROJECTS DEVELOPMENT

Table 2.02-1 summarizes all the capacity-related projects for the west side of the MMSD service area. The west side includes the service areas for PSs 1, 2, 3, 4, 5, 8, 11, 12, 15, 16, and 17. The segments referred to in Table 2.02-1 are defined using the following format: the major identifier (e.g., 8A) refers to a section of interceptor with the same peak hourly flows. The minor identifier (e.g., i in 8Ai) refers to different pipe size segments that have different capacities. The proposed sizing for relief sewers will be based upon either matching the size of the existing sewer, or if capacity greater than the existing sewer is required, a larger size will be included in the development of the capacity related project costs.

Based on the projected capacity needs for the interceptors and pumping stations on the west side of the MMSD service area, the following alternative projects to "baseline" operation were identified.

- Project W1–Nine Springs Valley Interceptor Relief
- Project W2–Sugar River WWTP
- Project W3–Dual Sugar River Satellite Plants (CTH PD Plant and Nesbitt Road Plant)
- Project W4–Village of Oregon Discharge to PS 11

These projects are described in more detail, including project options, in the following paragraphs.

A. Project W1–Nine Springs Valley Interceptor Relief

The Conveyance Facilities Analysis (CFA) prepared by the CARPC identified several interceptor segments of the Nine Springs Valley Interceptor (NSVI) in need of capacity improvements prior to 2060. These segments are identified in Table 2.02-1. The NSVI extends from the NSWTP to the downstream end of the PS 16 force main. The segment of this interceptor between PS 11 and the junction with the PS 17 force main (MH 12-110) will require capacity relief within ten years. The total length of interceptor relief between PS 11 and MH 12-110 is 37,630 feet. Project W1A is construction of parallel gravity sewers and force mains from MH 12-110 to the NSWTP. This project also includes major upgrades to PS 11 and PS 12 to address capacity and condition issues. The pumping station upgrades are scheduled to be completed by 2015. Alternatives to Project W1A would be construction of a parallel gravity sewer from MH 12-110 to PS 12 and a new force main from PS 12 to either PS 11 (Alternative W1B) or all the way to the NSWTP (Alternative W1C). The force main for Alternative W1C would parallel the existing force main from PS 11 to the NSWTP and cross-connected to PS 11 to provide a second force main from this critical pumping station. Figure 2.02-1 shows possible routings for Alternatives W1A and W1B.

Based on a limiting maximum velocity of 8 feet per second (fps) for the most limiting segment of the force main, the force main for Project W1B would be a 24-inch-diameter pipe, and the force main for Project W1C would be a 24-inch-diameter pipe from PS 12 to PS 11, and a 36-inch-diameter pipe from the cross-connection at PS 11 to the NSWTP.

Construction of Project W1B would eliminate the need to upgrade any of the NSVI between PS 12 and PS 11 for capacity reasons. It would also include capacity expansion for PS 11 firm pumping capacity and PS 11 force main capacity in 2015 and 2050, respectively.

TABLE 2.02-1

MMSD 2010-2060 WEST SIDE PROJECTS

Pump Station Pump Basin	From MH	to MH	Length (feet)	Existing Diameter (Inches)	Description	Capacity (mgd)	Cumulative Flows 2000	2030 TAZ	2030 UF	2060 Low	2060 High	Deficit	Timing High	Low	Project
PS 2, 3 and 4 Fm			17428	36	PS 2,3 AND 4 FM to NSWTP	36.5	31.65	30.95	33.82	33.82	38.16	1.66	2050		3-2
3i	03-111	03-102	2492	12	Rimrock Interceptor	1.08	1.24	1.29	1.4	1.4	1.4	0.32	2000	2000	3-1
3ii	03-102	PS 3	308	10	Rimrock Interceptor	1	1.24	1.29	1.4	1.4	1.4	0.4	2000	2000	3-1
PS 4					PS 4	4.2	3.88	3.92	4.10	4.10	4.30	0.10	2045		4-1
PS 2 and PS 4 Force Main				36	PS 2 and PS 4 Force Main	36.5	32.57	30.99	33.62	33.65	37.96	1.46	2050		4-2
8Aiii	02-545	02-538	3121	27	WI Relief	8.95	9.8	10.2	10.44	10.44	11.56	2.61	2000	2000	8-11
8Aiv	02-538	02-536	1200	24	WI Relief	8.52	9.8	10.2	10.44	10.44	11.56	3.04	2000	2000	8-12
8Av	02-536	02-535	600	21	WI Relief	5.97	9.8	10.2	10.44	10.44	11.56	5.59	2000	2000	8-13
8Avi	02-535	02-532	841	21	WI Relief	10.44	9.8	10.2	10.44	10.44	11.56	1.12	2030	2060	8-14
8C	02-531I	02-531A	2653	21	Midvale Relief	3.55	3.25	3.22	3.63	3.63	3.97	0.42	2022	2055	8-2
8Di	02-531A	02-519	4363	36	WI Relief	12.19	12.64	12.98	13.48	13.48	14.78	2.59	2000	2000	8-31
8Diii	02-518	02-516	204	10	WI Relief	12.19	12.64	12.98	13.48	13.48	14.78	2.59	2000	2000	8-32
8Ei	02-516	08-228	10	36	WI Relief	12.19	14.27	14.51	14.99	14.99	16.27	4.08	2000	2000	8-4
8Fvi	08-215	214X160	27	27	WI-Campus Relief	5.66	9.73	9.73	10.01	10.01	10.74	5.08	2000	2000	8-51
8Fx	08-207	08-201	1234	36	WI-Campus Relief	15.54	13.7	13.82	14.67	14.67	16.18	0.64	2047		8-52
8Ii	02-041	02-038	1063	18		2.71	1.41	1.5	2.2	2.2	2.94	0.23	2050		8-61
8Iii	02-038	02-034	1460	18		1.92	1.41	1.5	2.2	2.2	2.94	1.02	2020	2049	8-62
8Ji	02-034	02-232	816	20		2.84	2.42	2.5	3.48	3.48	4.29	1.45	2012	2040	8-71
8Jii	02-232	02-513	1704	21		3.24	2.42	2.5	3.48	3.48	4.29	1.05	2022	2052	8-72
8Kii	08-207	02-503	463	24	WI Relief	4.13	3.64	3.67	3.9	3.9	4.3	0.17	2047		8-8
8Miii	08-121	08-120	16	30	WI Randall Relief	18.8	20	19.89	20.96	20.96	22.79	3.99	2000	2000	8-9
8Sii	02-142	02-136	1669	27	SW Interceptor	5.66	5.36	5.31	5.59	5.59	5.94	0.28	2035		8-10

TABLE 2.02-1

MMSD 2010-2060 WEST SIDE PROJECTS

							Cumulative						Timing		
				Existing			Flows								Project
	From MH	to MH	Length	Diameter	Description	Capacity							High	Low	
Pump Station Pump Basin			(feet)	(Inches)		(mgd)	2000	2030 TAZ	2030 UF	2060 Low	2060 High	Deficit			
11Aii	11-171	11-169	812	42	NSVI	24.32	14.2	23.34	29.04	29.04	31.85	7.53	2022	2040	11-11
11Aiii	11-169	11-167	465	42	NSVI	24.32	15.06	24.14	29.87	29.87	32.66	8.34	2020	2035	11-12
11Aiv	11-167	11-161E	1436	42	NSVI	25.17	15.06	24.14	29.87	29.87	32.66	7.49	2025	2046	11-13
11Avi	11-161A	11-159	1321	36	NSVI	27.25	15.06	24.14	29.87	29.87	32.66	5.41	2025	2046	11-14
11Bi	11-159	11-158	340	36	NSVI	27.25	15.98	24.94	30.64	30.64	33.42	6.17	2023	2042	11-21
11Biii	11-156	11-151A	2220	42	NSVI	29.07	15.98	24.94	30.64	30.64	33.42	4.35	2027	2052	11-22
11C	11-151A	11-145	3784	42	NSVI	29.07	16.3	25.49	31.2	31.2	34.13	5.06	2025	2050	11-3
11Di	11-145A	11-141	1558	36	NSVI	37.81	19.89	29.28	35.01	35.01	37.88	0.07	2059		11-41
11Dii	11-141	11-137	1648	30	NSVI	35.75	19.89	29.28	35.01	35.01	37.88	2.13	2040		11-42
11Diii	11-137	11-129	3995	33	NSVI	31.31	19.89	29.28	35.01	35.01	37.88	6.57	2022	2040	11-43
11Div	11-129	11-127	733	36	NSVI	35	19.89	29.28	35.01	35.01	37.88	2.88	2030	2060	11-44
11Dv	11-127	11-116A	4855	54	NSVI	31.12	19.89	29.28	35.01	35.01	37.88	6.76	2021	2039	11-45
11Fi	11-116A	11-111A	2788	54	NSVI	31.12	20.6	29.94	35.74	35.74	38.6	7.48	2020	2035	11-51
11Fii	11-111A	11-106A	2716	54	NSVI	31.12	20.65	31.28	37.5	37.5	42.55	11.43	2019	2030	11-52
PS11					Pumping Station 11	25.5	22.04	32.45	39.21	39.21	43.76	18.26	2005	2010	11-6
PS 11 FM	PS 11	BD11-01008	4173	36	Pumping Station 11 FM	36.5	22.04	32.6	39.27	39.27	44.4	7.26	2025	2050	11-7
12Hi	12-110	12-101	3484	48	NSVI to PS 12	22.73	14.05	23.12	28.75	28.75	31.26	8.53	2018	2029	12-11
12Hii	12-101	PS 12	38	48	NSVI to PS 12	22.73	14.2	23.34	29.04	29.04	31.85	9.12	2017	2028	12-12
PS 12					Pumping Station 12	16.6	14.17	23.33	28.89	28.89	32.1	15.5	2005	2008	12-2
15C	05-113	05-106	2943	24	WI-West Extension	5.85	4.75	4.88	5.05	5.05	6.2	0.35	2049		15-1
PS 15					PS 15	5.8	5.44	5.66	5.82	5.82	6.96	1.16	2029	2060	15-2
16Aii	05-315	05-310	1002	18	WI	6.18	2.85	5.18	6.74	6.74	6.74	0.56	2049		16-11
16Aiv	05-306	05-236	1771	24	WI	6.03	2.85	5.18	6.74	6.74	6.74	0.71	2025	2046	16-12
PS 17					Pumping Station 17	4.6	2.72	7.83	11.26	11.26	13.58	8.98	2007	2011	17-1
PS 17 FM	PS17	17-14450	13357	16	Pumping Station 17 FM	7.2	2.69	8.12	11.59	11.59	14.52	7.32	2015	2025	17-2
PS 17 FM to MH 12-110	17-14550	12-110	3071	3071	Pumping Station 17 FM to MH 1	11.3	2.69	8.12	11.59	11.59	14.52	3.22	2029	2058	17-3
NSWTP					Nine Springs WTP	57.00	41.33	49.57	59.84	59.84	69.72	12.72	2025	2051	

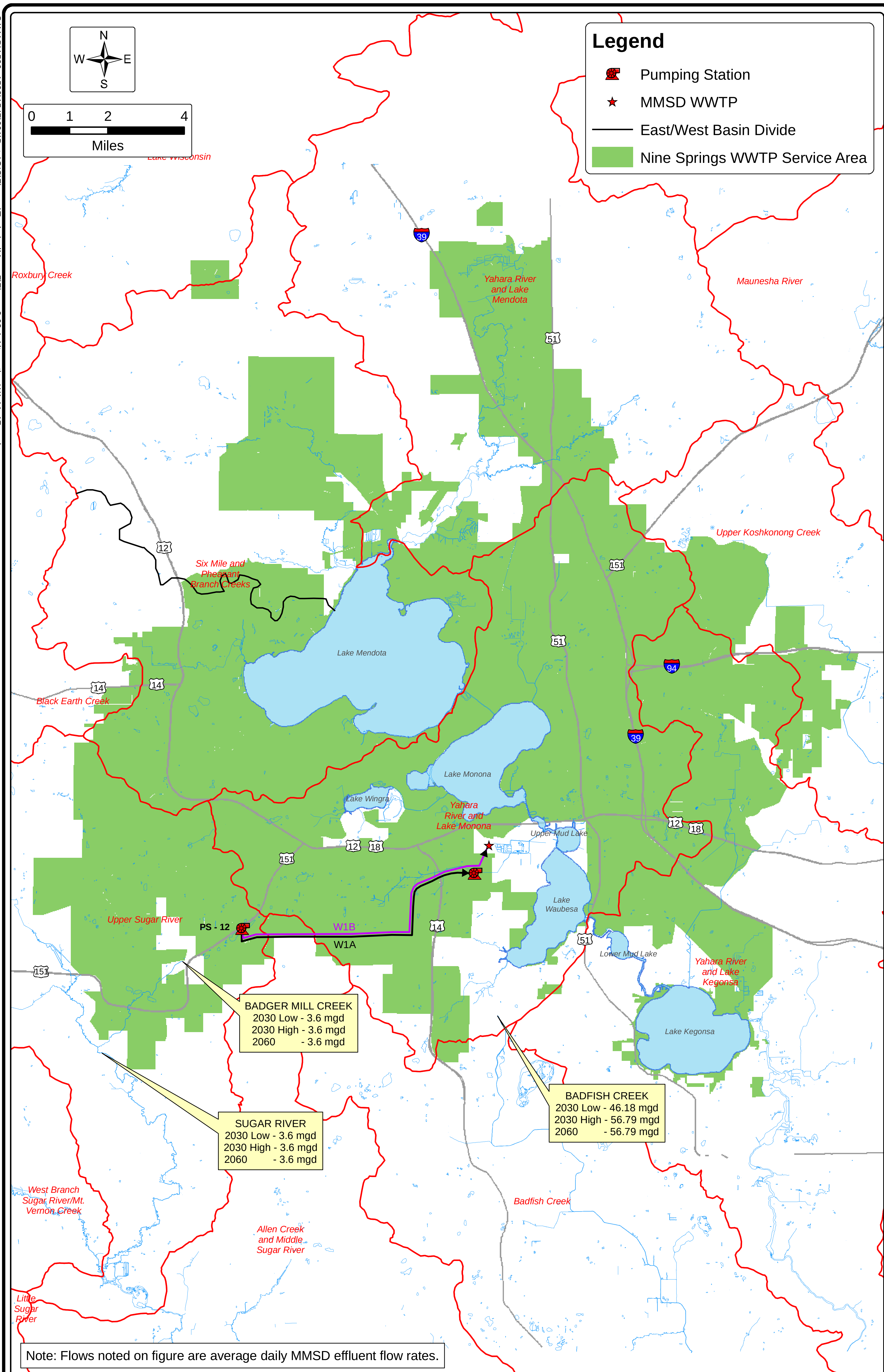


FIGURE 2.02-1
1547.001

ALTERNATIVE PROJECTS W1A/W1B
PS 12 RELIEF FORCE MAIN

TECHNICAL MEMO 7
ALTERNATIVES DEVELOPMENT
MADISON METROPOLITAN SEWERAGE DISTRICT

Construction of Project W1C would eliminate the need to provide additional capacity for the existing PS 11 force main and would extend the time until PS 11 capacity would be reached by 35 years under high flow projections, from the scheduled upgrade in 2015 until 2050. Construction to address equipment condition issues at PS 11 would still be required by 2015; however, capacity expansion, which would be part of this project, would not be necessary until 2050.

Project W4 would require additional capacity of about 4.4 mgd by the year 2030 and 5.8 mgd by the year 2060 at PS 11 and in the PS 11 force main for a future Village of Oregon connection to the MMSD system. If Project W1C is implemented in conjunction with Project W4, the resulting increase in the peak flow capacity required at PS 11 would need to be addressed 24 years earlier under high flow projections, in 2026, rather than in 2050.

Under any of the W1 projects, 3.6 mgd of flow would be returned to Badger Mill Creek in the Sugar River watershed through the Badger Mill Creek force main. Figure 2.02-1 shows the proposed routing for the relief force mains and the potential MMSD service area for the year 2060. Effluent flows in 2060 to Badger Mill Creek, the Sugar River and Badfish Creek are also shown in this figure.

This project also includes the construction of additions to the NSWTP in 2020 and 2030 to address higher levels of phosphorus and nitrogen removal, respectively, and an expansion of the biosolids facilities' capacity in 2030.

B. Project W2–Sugar River WWTP

This project would redirect all flow tributary to PS 17 to a new Sugar River WWTP with a discharge to the Sugar River downstream of its confluence with Badger Mill Creek (Project W2A), or alternatively, via force main to the Sugar River at CTH PD (Project W2B). Figure 2.02-2 shows the proposed service area for the Sugar River WWTP and the year 2060 effluent flows for the Sugar River WWTP, the Sugar River, Badger Mill Creek, and Badfish Creek.

A very high quality effluent will be required for a Sugar River discharge because of its exceptional resource water classification. To achieve this quality the use of membrane filtration and ultra violet disinfection are expected to be necessary. Since the design peaking factor for these unit processes is 2, the average day design flow for these processes is calculated by dividing the PHF, calculated from the MDC, by 2. Based on the flow projections prepared by CARPC, this treatment plant would have the following design flows and loadings.

Population Projection	ADF (mgd)	PHF (mgd)	Biological and Disinfection Design Flow (mgd)	BOD Loading (lbs/day)
2030 Low	2.22	7.83	3.91	5,554
2030 High	3.42	11.26	5.63	8,557
2060 High	4.27	13.58	6.79	10,684

A proposed process schematic for this high quality effluent plant is provided in Section 2.05 of this Technical Memo.

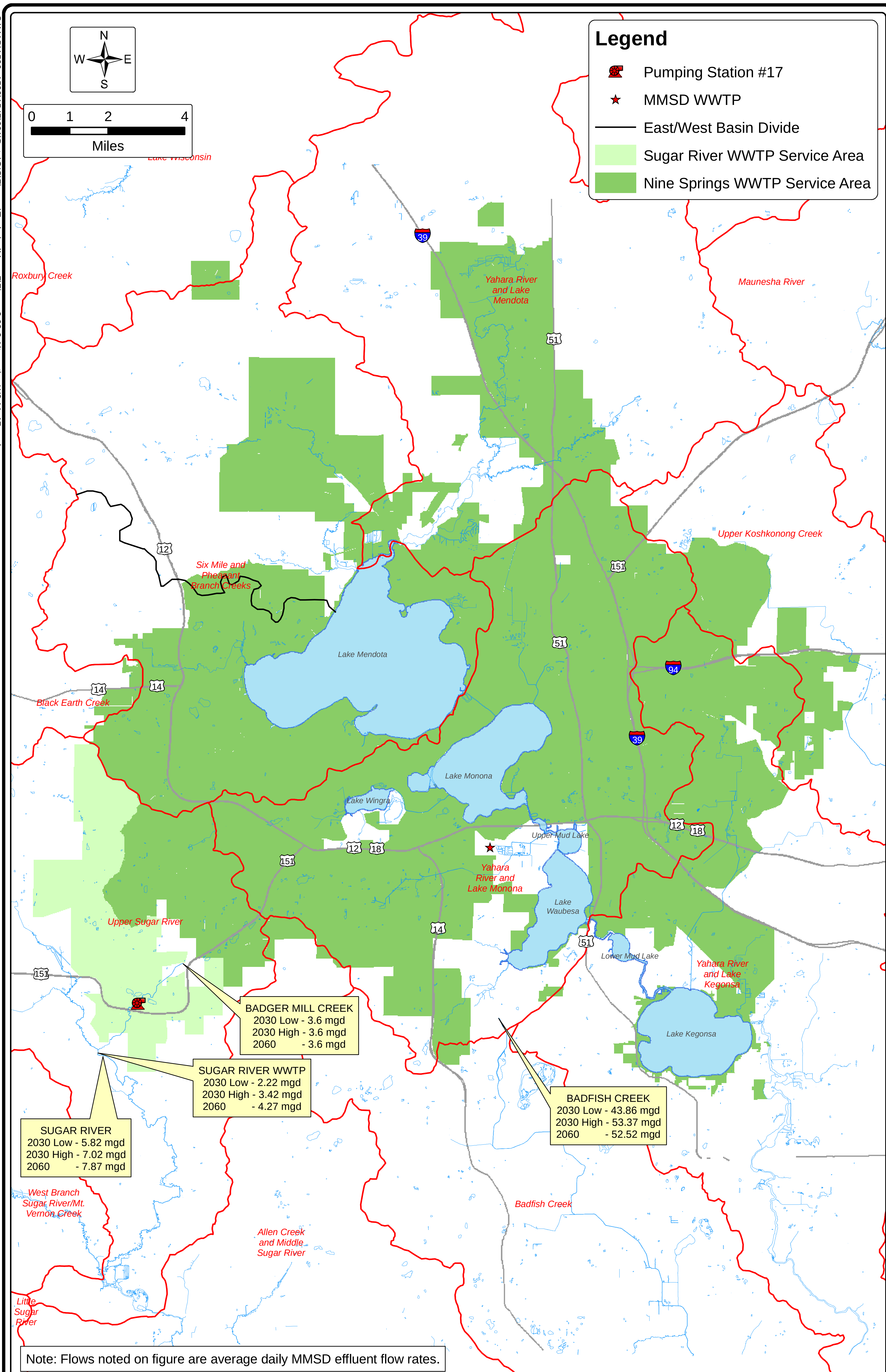


FIGURE 2.02-2
1547.001

STRAND ASSOCIATES, INC. ENGINEERS

MALCOLM PIRNIE

ALTERNATIVE PROJECT W2
SUGAR RIVER WWTP

TECHNICAL MEMO 7
ALTERNATIVES DEVELOPMENT
MADISON METROPOLITAN SEWERAGE DISTRICT

Construction of this facility would eliminate the need for all capacity improvements for the NSVI and the PS 12 and PS 11 force mains, provided it could be sited and constructed prior to approximately 2020. Construction to address equipment condition issues at PS 11 and PS 12 would still be required by 2015. Construction of the Sugar River WTP would delay the need to address the firm pumping capacity at PS 11 by 30 years, from its scheduled upgrade in 2015 until 2045. The capacities of the 2020 and 2030 NSWTP additions to address phosphorus and nitrogen would be reduced if this project was implemented, and the need to expand the biosolids facilities at the NSWTP would be delayed.

If this project were built in conjunction with Project W4, which provides reserve capacity for the Village of Oregon, the resulting increase in the peak flow capacity required at PS 11 would need to be addressed 15 years earlier under high flow projections, in 2030, rather than in 2045.

The average day effluent flows to the Sugar River, including the 3.6 mgd Badger Mill Creek force main discharge, are shown in the table below. The effluent volumes very closely match the projected groundwater withdrawals in the Sugar River watershed.

Population Projection	MMSD Effluent to Sugar River (mgd)	Service Area Flows Located in Sugar River Watershed
2030 Low	5.82	5.03
2030 High	7.02	6.86
2060 High	7.87	8.09

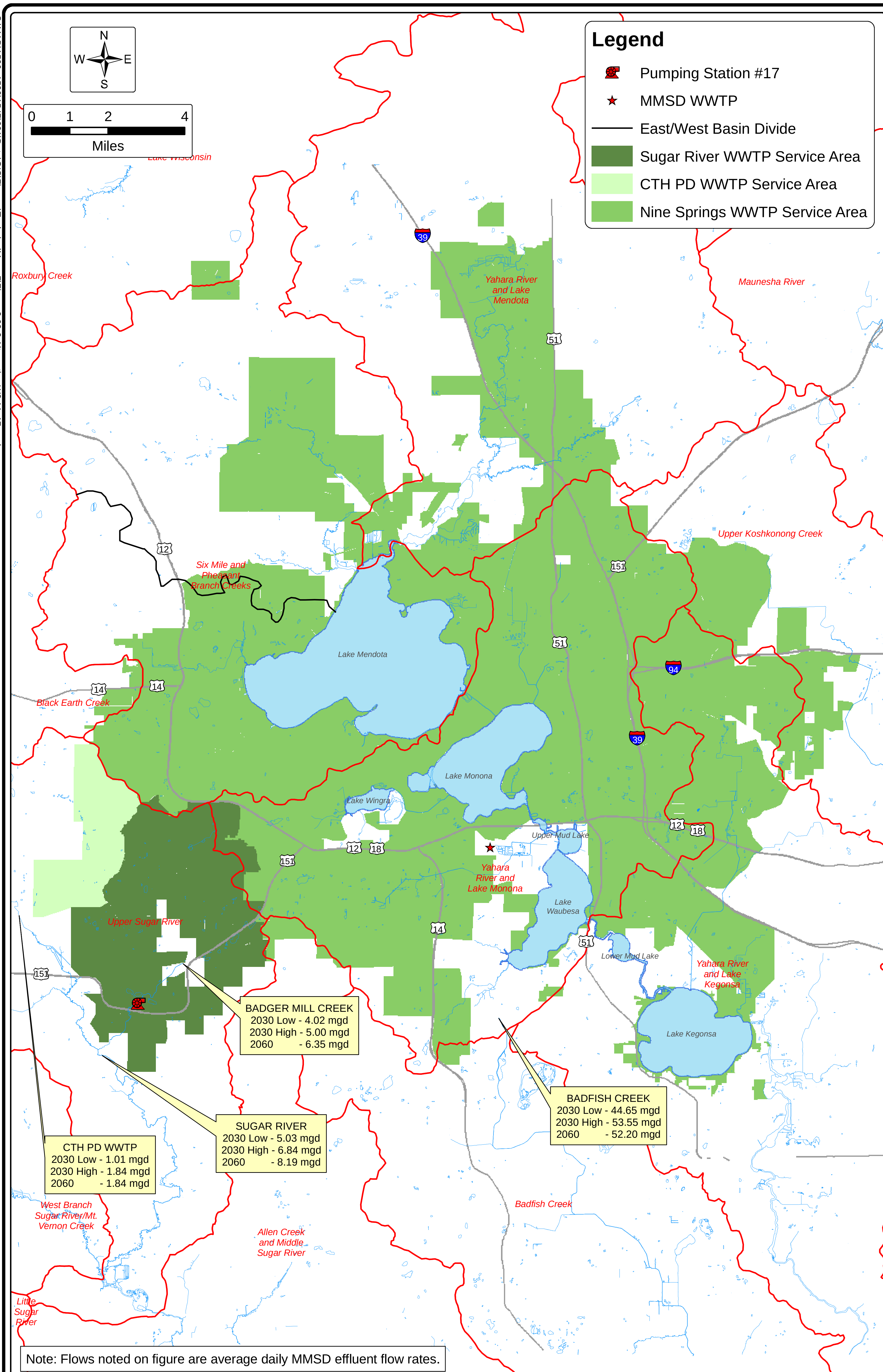
C. <u>Project W3–Dual Sugar River Satellite Plants</u>

The two proposed treatment plants for this project would return water to the Sugar River upstream of the confluence with Badger Mill Creek (CTH PD WWTP) and to the City of Madison storm water pond on Nesbitt Road (Nesbitt Road WWTP), which are the headwaters for continuous flow on Badger Mill Creek.

The CTH PD WWTP would be sized to serve PS 17 subservice areas B and C, which are located north of CTH PD. Figure 2.02-3 shows the proposed service area for the CTH PD WWTP, the effluent flows from this plant, and the effluent flows to Badger Mill Creek, the Sugar River and Badfish Creek.

A very high quality effluent will be required for a Sugar River discharge because of its exceptional resource water classification. To achieve this quality the use of membrane filtration and ultra-violet disinfection are expected to be necessary. Since the design peaking factor for these unit processes is 2, the average day design flow for these processes is calculated by dividing the PHF, calculated from the MDC, by 2. Based on the flow projections prepared by CARPC, this treatment plant would have the following design flows and loadings.

Population Projection	ADF (mgd)	PHF (mgd)	Biological and Disinfection Design Flow (mgd)	BOD Loading (lbs/day)
2030 Low	1.01	4.03	2.02	2,527
2030 High	1.84	6.68	3.34	4,604
2060 High	1.84	6.68	3.34	4,604



S:\MAD\1500--1599\1547\001\Data\GIS\Figures\TechnicalMemo7\Figure 2.02-3 Alternative W3 11x17.mxd

FIGURE 2.02-3
1547.001

STRAND ASSOCIATES, INC. ENGINEERS

MALCOLM PIRNIE

ALTERNATIVE PROJECT W3
SUGAR RIVER SATELLITE WWTPS

TECHNICIAL MEMO 7
ALTERNATIVES DEVELOPMENT
MADISON METROPOLITAN SEWERAGE DISTRICT

The Nesbitt Road WWTP would be sized to serve the PS 17 subservice areas A and D and PS 12 subservice areas 12B through 12H. Since Badger Mill Creek is not an exceptional resource water and there is an existing permitted 3.6 mgd discharge to this creek upstream of the Sugar River, the treatment requirements may be less stringent than for the CTH PD WWTP or the Sugar River WWTP described in Project W2. However, for purposes of the preliminary design, this facility will be configured in a similar fashion to the CTH PD WWTP and the Sugar River WWTP. Based on the flow projections prepared by CARPC, the treatment plant would have the following design flows and loadings.

Population Projection	ADF (mgd)	PHF (mgd)	Biological and Disinfection Design Flow (mgd)	BOD Loading (lbs/day)
2030 Low	4.02	12.91	6.45	10,058
2030 High	5.00	15.51	7.75	12,560
2060 High	6.35	18.97	9.49	15,638

Construction of both facilities would eliminate all required capacity improvements to the NSVI, provided the facilities could be sited and constructed prior to about 2020. Construction of the Nesbitt Road WWTP would also provide for a future reserve for the Village of Oregon flows at PS 11 (Project W4) without any hydraulic expansion of either PS 11 or its force main required before 2060. Construction to address equipment conditions at PS 11 and PS 12 would still be required by 2015. The capacities of the 2020 and 2030 NSWTP additions to address phosphorus and nitrogen would be reduced if this project was implemented, and the need to expand the biosolids facilities at the NSWTP would be delayed.

If both treatment plants are constructed, all of the flow generated in the Sugar River watershed would be retained in the Sugar River Watershed. Effluent from these facilities would either provide stream flow directly to the Sugar River and Badger Mill Creek or effluent for reuse or infiltration in the Sugar River watershed. The total flow that would remain in the Sugar River watershed would be 8.19 mgd. Under this project, the Badger Mill Creek Force Main would not be used to return 3.6 mgd of treated effluent from the NSWTP, but it would be available to route effluent from the NSWTP for reuse or infiltration in the south central portions of MMSD. The flow available to return to the Sugar River and Badger Mill Creek would be as follows.

Population Projection	Upper Sugar River Flow (mgd)	Badger Mill Creek (mgd)	Sugar River Flow (mgd)
2030 Low	1.01	4.02	5.03
2030 High	1.84	5.02	6.86
2060 High	1.84	6.25	8.09

One possible discharge option for the CTH PD WWTP would be to route the effluent through the wetlands that border the Sugar River in the vicinity of CTH PD. Figure 2.02-3 shows the potential location for the terminal point of the interceptors serving areas 17B and 17C and the potential area for discharge to the Sugar River. Depending on available soils, infiltration in or around these wetlands may also serve as a water source to maintain the wetlands. Effluent reuse options near the CTH PD WWTP would include golf course irrigation and possible industrial reuse should an industrial park be cited in the vicinity, which would provide a location for a wet industry with significant water use potential.

The proposed discharge location for the Nesbitt Road WWTP is also shown on Figure 2.02-3. This would provide continuous flow to the entire reach of Badger Mill Creek downstream of the City of Madison stormwater pond located south of Nesbitt Road. An effluent cascade similar to the one at the terminus of the Badger Mill Creek Force Main could be provided to aerate the wastewater prior to entering the stormwater pond. This discharge option would also be available for Alternative W2 if the Badger Mill Creek Force Main were rerouted to terminate at the City of Madison stormwater pond rather than at its current location.

An alternative to construction of a high quality effluent plant near Nesbitt Road would be to continue to pump treated effluent from the NSWTP. Construction of only the CTH PD WWTP would eliminate all capacity improvements for NSVI segments 11Aiv, 11B, 11C, and 11Di, 11Dii, and 11Div. Need for capacity improvements to other sections of the NSVI would be delayed as shown in the following table.

	2060 High Baseline Deficit	**Original Date for Reaching Capacity**	**Revised Date for Reaching Capacity**
11Aii	7.98	2021	2048
11Aiii	8.79	2019	2041
11Aiv	7.94	2021	2048
11Diii	7.01	2022	2057
11Dv	7.20	2021	2055
11Fi	7.91	2021	2049
11Fii	11.86	2019	2032
11Fiii	12.48	2018	2023
11Fiv	12.48	2016	2023
PS11-Firm	19.3	2010	2017
PS 11 Force Main	8.32	2025	2051
12Hi	8.14	2018	2040
12Hii	9.57	2017	2034
PS 12-Firm	15.5	2010	2010

D. Project W4–Village of Oregon Discharge to PS 11

Based on the development plans for the City of Fitchburg, a significant expansion of the Village of Oregon's WWTP to serve current or future MMSD service areas is unlikely. However, a potential exists for the Village of Oregon to join the District. Flow from the Village of Oregon would be directed ultimately to the PS 11 service area to be pumped to the NSWTP. Figure 2.02-4 shows the proposed addition of the Village of Oregon to the MMSD service area and the resulting effluent flows to Badger Mill Creek, the Sugar River and Badfish Creek. The flows in Badfish Creek downstream of the existing Village of Oregon's WWTP discharge would be unchanged from the current operating configuration if this project were implemented. The projected flow for the Village of Oregon based on CARPC population projections, a 100 gallons per capita per day (gcd) flow contribution, and the MDC peaking factor would be as follows.

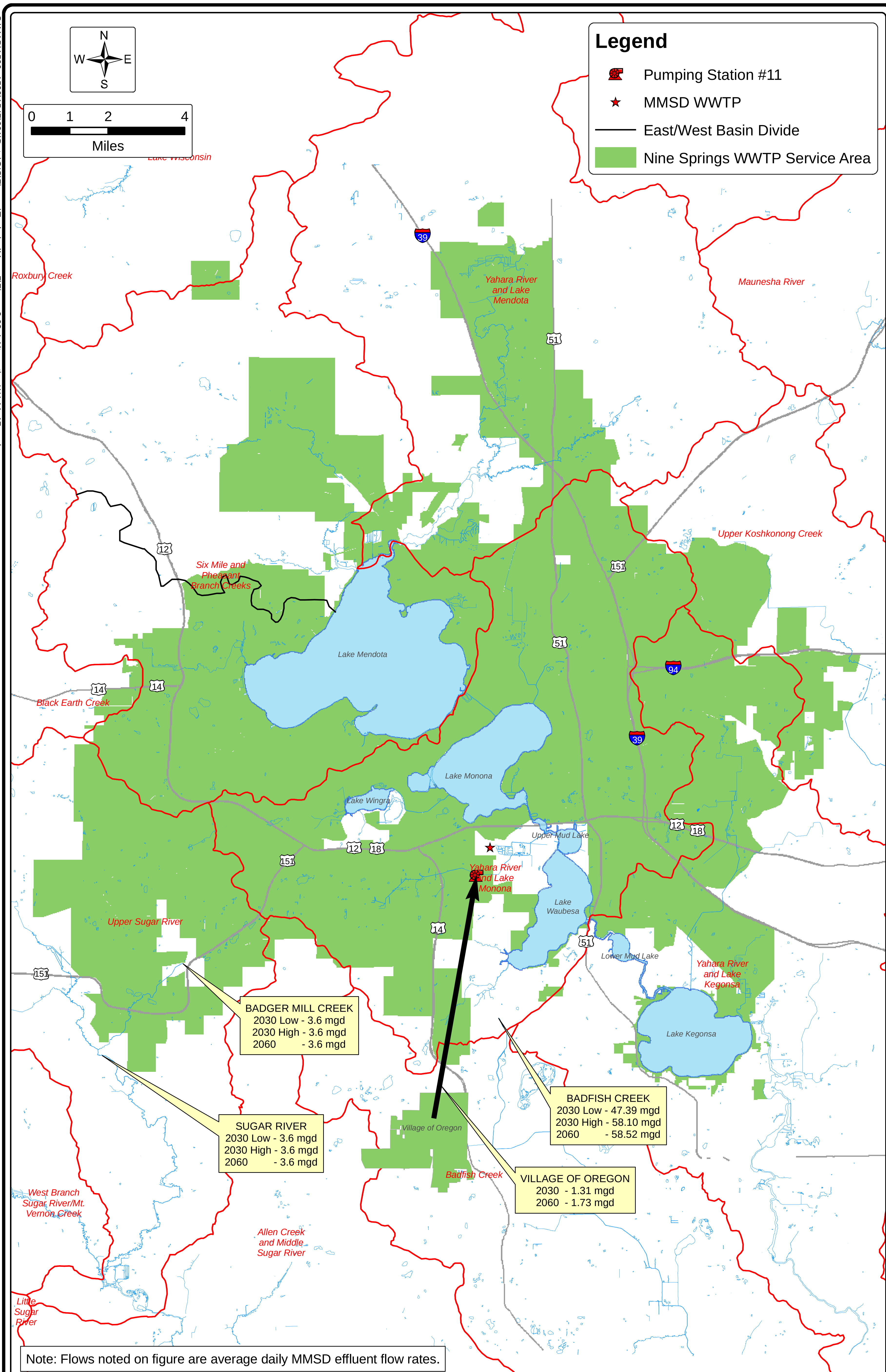


FIGURE 2.02-4
1547.001

STRAND ASSOCIATES, INC. ENGINEERS

MALCOLM PIRNIE

MMSD

ALTERNATIVE W4
VILLAGE OF OREGON DISCHARGE TO PS 11

TECHNICAL MEMO 7
ALTERNATIVES DEVELOPMENT
MADISON METROPOLITAN SEWERAGE DISTRICT

Year	Population	ADF (mgd)	PHF (mgd)
2000	7,514	0.75	3.01
2030	13,106	1.31	5.02
2060	17,275	1.73	6.34

All flows from the Village of Oregon are generated in the Badfish Creek watershed and would be returned to the Badfish Creek watershed through the Badfish Creek effluent force main. Since the maximum effluent pumping rate from the NSWTP to Badfish Creek is 75 mgd, if this project were implemented, the frequency of exceeding the effluent storage lagoons capacity at the NSWTP would increase slightly, resulting in more effluent discharges to Nine Springs Creek and Lake Waubesa. The same increase will occur when an equivalent population conveys its wastewater to the NSWTP, regardless of where that population is located...

The District should meet with the Village of Oregon as part of the planning effort for future capacity expansions at PS 11 to assess the likelihood of the need for the additional capacity required under this project. If none of the west side decentralization projects are pursued, planning for capacity expansion at PS 11 needs to begin immediately.

If flows from the Village of Oregon were rerouted to the NSWTP, the additions to address phosphorus and nitrogen in 2020 and 2030, respectively, would increase in size under this project, and the biosolids facilities would need to be expanded earlier.

2.03 EAST SIDE PROJECTS DEVELOPMENT

Table 2.03-1 summarizes all the capacity-related projects for the conveyance system on the east side of the MMSD service area. The east side includes the service areas for PSs 6, 7, 9, 10, 13, and 14.

Based on the projected capacity needs for the interceptors and pumping stations on the east side of the MMSD service area and potential expansion of the NSWTP, the following alternative projects to "baseline" operation were identified.

- Project E1–Mendota WWTP
- Project E2–Starkweather Creek WWTP
- Project E3–PS 13 and PS 14 Service Area WWTP
- Project E4–Stoughton WWTP Expansion
- Project E5–Centralized High Quality Effluent Treatment Facilities
- Project E6–Sun Prairie WWTP Expansion

These projects are described in more detail, including project options, in the following paragraphs.

A. Project E1–Mendota Wastewater Treatment Plant

In the late 1990s, the MMSD identified a potential option to discharge highly treated effluent to the Yahara River upstream of Lake Mendota. As a result, MMSD purchased land north of Lake Mendota near the Yahara River to maintain the option of constructing a WWTP that would serve the District service area in the Yahara River watershed north of Lake Mendota. Figure 2.03-1 shows the proposed service area for this facility and the effluent flows associated with this project for the Mendota WTP, the Yahara River, Badfish Creek, Badger Mill Creek, and the Sugar River. This project would redirect all flow from the service area for PS 14 to this new facility. Effluent from this facility could provide additional base flow to the Yahara River upstream of Lake Mendota, provide effluent for wetland restoration, recharge the groundwater by infiltration, or serve as reuse water or water for turf irrigation. The design flows and BOD_5 loadings for this facility would be as follows.

Population Projection	**ADF (mgd)**	**PHF (mgd)**	**Design Flow (mgd)**	**BOD_5 Loading (lbs/day)**
2030 Low	4.66	14.62	7.31	11,659
2030 High	5.27	16.21	8.11	13,186
2060 High	6.83	20.17	10.08	17,089

TABLE 2.03-1

MMSD 2010-2060 EAST SIDE PROJECTS

				Existing			Cumulative Flows (mgd)									Project
	From MH	to MH	Length	Diameter	Description	Capacity								Earliest	Latest	
Pump Station Pump Basin			(feet)	(Inches)		(mgd)	2000	2030 TAZ	2030 UF	2060 Low	2060 High	Deficit				
7Ai	07-955	07-954	95	48	NEI-PS 10 to SEI at Buckeye Road	40.45	25.09	31.30	37.44	37.44	40.88	0.43		2056		7-1
7Ci	07-734	07-728	2917	21	FEI-Door Creek Extension	4.36	0.18	2.62	7.14	7.14	12.23	7.87		2018	2042	7-31
7Cii	07-728	07-723	2496	21	FEI-Door Creek Extension	5.41	0.18	2.62	7.14	7.14	12.23	6.82		2023	2049	7-32
7Ciii	07-723	07-707	7899	24	FEI-Door Creek Extension	5.98	0.18	2.62	7.14	7.14	12.23	6.25		2025	2052	7-33
7Civ	07-707	07-426	3474	24	FEI-Door Creek Extension	7.12	0.18	3.82	8.49	8.49	15.36	8.24		2025	2050	7-34
7Di	07-426	07-425	153	36	FEI-East of Interstate 90	12.19	1.68	6.41	11.11	11.11	17.31	5.12		2035		7-41
7Dii	07-425	07-416	3861	30	FEI-East of Interstate 90	7.49	1.68	6.41	11.11	11.11	17.31	9.82		2018	2037	7-42
7Diii	07-416	07-415	355	42	FEI-East of Interstate 90	15.92	1.68	6.41	11.11	11.11	17.31	1.39		2053		7-43
7E	07-415	07-932	8067	42	FEI-West of Interstate 90	15.92	1.96	6.71	11.44	11.44	17.60	1.68		2052		7-5
7Fi	07-932	07-313	14	42	NEI-Downstream of FEI	15.92	26.75	35.94	45.50	45.50	53.68	37.76		2000	2000	7-61
7Fii	07-313	07-215	5591	48	NEI-Downstream of FEI	32.14	26.75	35.94	45.50	45.50	53.68	21.54		2009	2018	7-62
7Ji	07-249	07-242	2794	18	SEI-Blooming Grove Extension	2.25	0.37	1.21	5.21	5.21	6.36	4.11		2012	2038	7-71
7Jii	07-242	07-231	4974	24	SEI-Blooming Grove Extension	3.87	0.37	1.21	5.21	5.21	6.36	2.49		2022	2050	7-72
7Jiii	07-231	07-228	1347	24	SEI-Blooming Grove Extension	5.06	0.37	1.21	5.21	5.21	6.36	1.30		2029	2059	7-73
7Ki	07-228	07-224	2001	30	SEI-Blooming Grove Ext. Downstream of Mc Farland Relief Sewer	10.26	3.84	6.18	9.98	9.98	12.28	2.02		2034		7-81
7Kii	07-224	07-222	650	30	SEI-Blooming Grove Ext. Downstream of Mc Farland Relief Sewer	10.26	4.21	6.54	10.42	10.42	12.70	2.44		2029	2058	7-82
7Kiii	07-222	07-218	1647	36	SEI-Blooming Grove Ext. Downstream of Mc Farland Relief Sewer	10.55	4.21	6.54	10.42	10.42	12.70	2.15		2032		7-83
7Kiv	07-218	07-215	1606	36	SEI-Downstream of Blooming Grove Extension	11.40	4.51	6.82	10.71	10.71	12.99	1.59		2039		7-84
7Mi	07-215	07-211	2468	60	SEI-Downstream of NEI	37.62	29.44	40.10	52.28	52.28	64.74	27.12		2011	2023	7-91
7Mii	07-211	PS 7	5342	60	SEI-Downstream of NEI	37.62	30.09	40.74	53.01	53.01	65.62	28.00		2010	2021	7-92
PS7 MDC					Pumping Station 7-MDC-Firm Capacity	39.00	35.13	45.90	59.86	59.86	72.27	33.27		2005	2011	7-10
					Pumping Station 7-MDC-Maximum Capacity	45.00	35.13	45.90	59.86	59.86	72.27	27.27		2012	2027	7-10
PS 7 FM	PS 7	TEO7A-1520	6996	36	Pumping Station 7 Force Main	27.50	17.57	22.95	29.93	29.93	36.15	8.64		2024	2050	7-111
	PS 7	TEO7A-1520	6996	36	Pumping Station 7 Force Main	27.50	17.57	22.95	29.93	29.93	36.15	8.64		2024	2050	7-111
	TEO7A-1520	NSWTP	1665	48	Pumping Station 7 Force Main	65.00	35.13	45.90	59.86	59.86	72.30	7.30		2042		7-112

TABLE 2.03-1

MMSD 2010-2060 EAST SIDE PROJECTS

	From MH	to MH	Length	Existing Diameter	Description	Capacity	Flows						Earliest	Latest	Project
Pump Station Pump Basin			(feet)	(Inches)		(mgd)	2000	2030 TAZ	2030 UF	2060 Low	2060 High	Deficit			
9A	09-108	09-104	1678	24	SEI-Upstream of PS 9	4.13	2.05	2.92	3.67	3.67	5.25	1.12	2039		9-1
9Bi	09-104	09-101	1373	27	SEI Upstream of MH 09-101	5.66	3.22	4.24	4.93	4.93	6.39	0.73	2045		9-21
9Bii	09-101	PS 9	285	24		4.62	3.22	4.24	4.93	4.93	6.39	1.77	2025	2047	9-22
PS 9					Pumping Station 9	4.50	3.22	4.24	4.93	4.93	6.39	1.89	2022	2041	9-3
PS 9 FM	PS9	TE09-20598	40	14	Pumping Station 9 FM	5.50	3.22	4.24	4.93	4.93	6.39	0.89	2048		9-4
10A	10-145	10-121	10973	48	NEI PS 13 to PS 10	24.55	19.09	24.11	28.47	28.47	32.08	7.53	2017	2033	10-1
10Bi	10-121	10-118	874	36	NEI PS 13 to PS 10	21.54	20.06	25.10	29.54	29.54	33.13	11.59	2005	2009	10-21
10Bii	10-118	10-201	1597	42	NEI PS 13 to PS 10	18.38	20.06	25.10	29.54	29.54	33.13	14.75	2000	2000	10-22
10Ei	10-201	10-115	140	42	NEI PS 13 to PS 10-Downstream of Lien Extension	18.38	20.85	27.04	33.02	33.02	36.54	18.16	2000	2000	10-31
10Eii	10-115	10-104A	4412	48	NEI PS 13 to PS 10-Downstream of Lien Extension	20.75	21.26	27.44	33.44	33.44	36.95	16.20	2000	2000	10-32
10Eiii	10-104A	10-102A	1110	48	NEI PS 13 to PS 10-Downstream of Lien Extension	20.75	21.72	27.87	33.87	33.87	37.38	16.63	2000	2000	10-33
10G	10-102A	10-101	959	48	NEI-Downstream of Hwy 30 Extension	20.75	21.74	27.90	33.91	33.91	37.42	16.67	2000	2000	10-4
10H	10-101	PS10	108	48	NEI-Downstream of Hwy 30 Extension	20.75	23.13	29.25	35.26	35.26	38.74	17.99	2000	2000	10-5
PS 10FM	PS 10	MH 07-955	11109	36	Pumping Station 10 Force Main	36.50	23.13	29.25	35.26	35.26	38.74	2.24	2041		10-6
PS 6 to PS 10 FM	PS 10	PS 6			Pumping Station 6/10 Force Main Interconnection										10-7
13G	13-132	13-122A	4397	48	NEI PS 14 to PS 13	20.75	12.01	15.71	17.31	17.31	21.24	0.49	2056		13-1
13A-Ei	13-122A	13-116H	153	48	NEI PS 14 to PS 13	20.75	16.94	20.57	22.52	22.52	26.28	5.53	2020	2033	13-2
13Hi	13-105A	13-105	125	46.5	NEI PS 14 to PS 13	26.70	17.00	21.56	25.77	25.77	29.44	2.74	2038		13-3
13Hii	13-105	PS 13	1758	48	NEI PS 14 to PS 13	24.55	17.00	21.56	25.77	25.77	29.44	4.89	2026	2051	13-4
PS 13					Pumping Station 13	20.00	17.00	21.56	25.77	25.77	29.44	9.44	2010	2020	13-5

TABLE 2.03-1

MMSD 2010-2060 EAST SIDE PROJECTS

				Existing			Flows								Project
	From MH	to MH	Length	Diameter	Description	Capacity							Earliest	Latest	
Pump Station Pump Basin			(feet)	(Inches)		(mgd)	2000	2030 TAZ	2030 UF	2060 Low	2060 High	Deficit			
14B	14-196	14-193	1203	21	NEI-Deforest Extension	3.39	2.69	3.16	3.61	3.61	5.19	1.80	2023	2045	14-1
14D	14-182	14-171	5724	21	NEI-Deforest Extension	5.51	2.97	3.87	4.32	4.32	5.80	0.29	2054		14-2
14E	14-171	14-166	2351	21	NEI-Deforest Extension	5.51	3.13	4.02	4.45	4.45	5.92	0.41	2052		14-3
14Fi	14-166	14-165	488	21	NEI-Deforest Extension	5.51	3.76	4.90	5.35	5.35	7.27	1.76	2033		14-41
14Fii	14-165	14-162	1401	24	NEI-Deforest Extension	7.01	3.76	4.90	5.35	5.35	7.27	0.26	2056		14-42
14G	14-162	14-156	2687	24	NEI-Deforest Extension	7.01	3.81	5.23	5.72	5.72	7.62	0.61	2050		14-5
14Jii	14-415	14-411	1619	15	NEI-Hwy 19 Extension	2.21	0.81	1.42	2.08	2.08	2.50	0.29	2039		14-61
14Jv	14-407	14-134	3059	18	NEI-Hwy 19 Extension	2.35	0.81	1.42	2.08	2.08	2.50	0.15	2049		14-62
14K	14-134	14-102	16679	36	NEI:DeForest Extension after HWY 19 Extension	9.63	5.57	7.45	8.58	8.58	10.74	1.11	2045		14-7
14Li	14-362	14-358	775	10	NEI-Waunakee Extension	1.54	1.34	1.52	1.58	1.58	1.80	0.26	2025	2040	14-81
14Lii	14-358	14-356	674	24	NEI-Waunakee Extension	5.47	3.45	4.42	4.69	4.69	5.71	0.24	2053		14-82
14Mi	14-356	14-345	4659	24	NEI-Waunakee Extension	5.85	4.45	6.42	7.03	7.03	9.00	3.15	2016	2021	14-91
14Mii	14-345	14-338	2859	21	NEI-Waunakee Extension	6.31	4.45	6.42	7.03	7.03	9.00	2.69	2022	2028	14-92
14Miii	14-338	14-333	2110	21	NEI-Waunakee Extension	7.99	4.45	6.42	7.03	7.03	9.00	1.01	2045		14-93
14Miv	14-333	14-323	4889	30	NEI-Waunakee Extension	7.01	4.45	6.42	7.03	7.03	9.00	1.99	2030	2059	14-94
14N	14-323	14-315	4055	30	NEI-Waunakee Extension	7.01	4.86	7.02	7.65	7.65	9.75	2.74	2023	2030	14-10
14Oi	14-315	14-301	5251	30	NEI-Waunakee Extension	9.18	5.46	7.63	8.28	8.28	10.61	1.43	2042		14-11
PS 14					Pumping Station 14	15.00	11.00	14.58	16.18	16.18	20.16	5.16	2023	2038	14-12

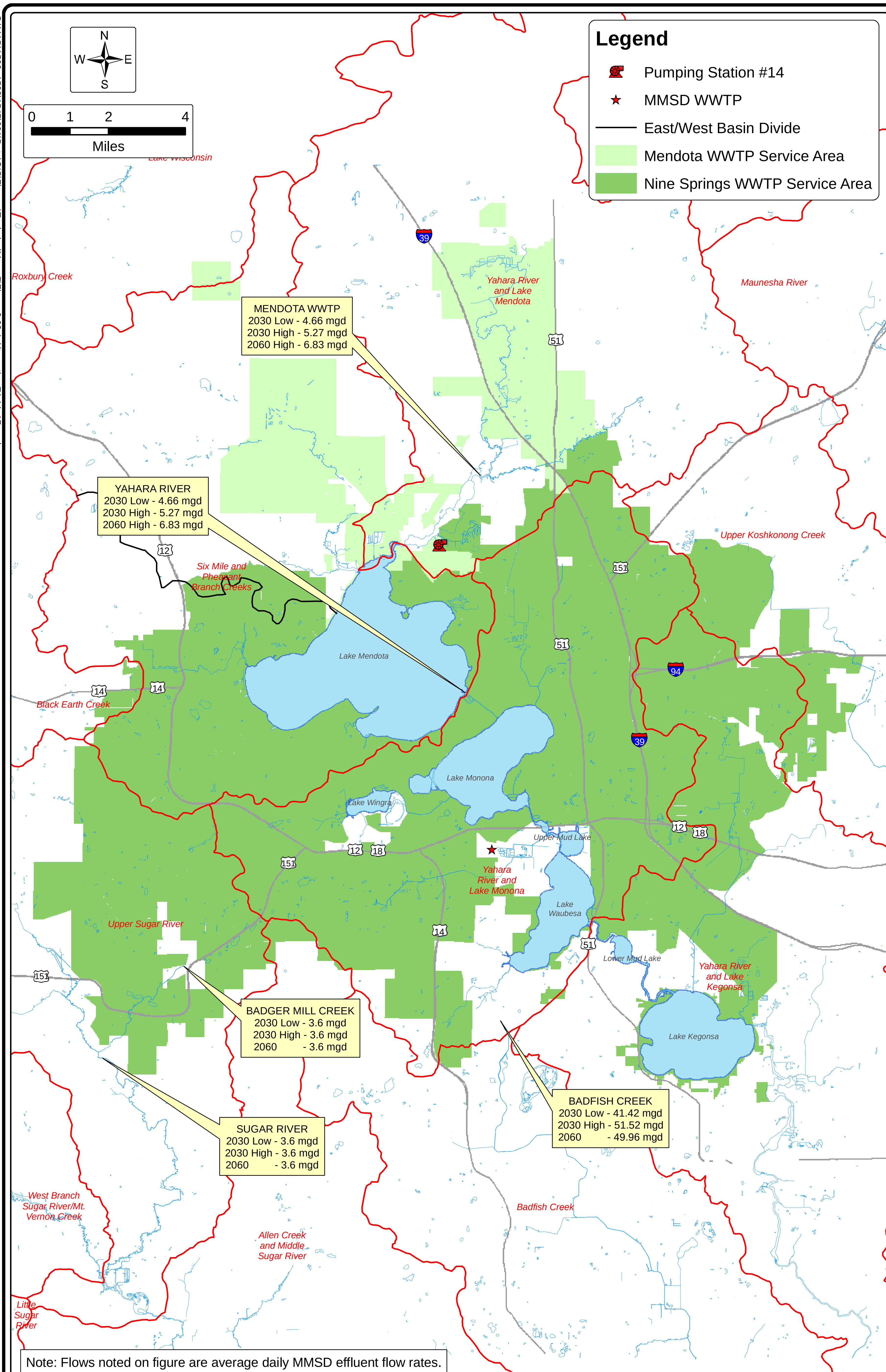


FIGURE 2.03-1
1547.001

STRAND ASSOCIATES, INC. ENGINEERS

MALCOLM PIRNIE

MMSD

ALTERNATIVE PROJECT E1
MENDOTA WWTP

TECHNICAL MEMO 7
ALTERNATIVES DEVELOPMENT
MADISON METROPOLITAN SEWERAGE DISTRICT

Because of current legislative and regulatory constraints and the time necessary to gain public acceptance for a treatment plant and effluent discharge at this location, the ability to site and operate a WWTP on the north side of Lake Mendota is not expected to be possible before 2025. There are several components of the conveyance system downstream of the Mendota WTP site that will reach capacity before 2025. The District has plans in place to address these components, including the construction of PS 18 that will provide backup capacity for PS 7 and relief for the SEI downstream of its juncture with the NEI, as shown in the following table.

Project Description	Interceptor Segments	Project Start Date	Project Cost
NEI Relief Upstream of PS 10	10 B to 10 H	2008	$10,200,000
PS 18		2010	$8,500,000
PS 18 Force Main		2010	$8,500,000
NEI–from FEI to PS 18	7Fi, 7Fii	2012	$5,260,000
PS 7 Improvements		2013	$1,110,000
NEI Relief Upstream of PS 10	10A	2016	$10,000,000
Total			**$43,570,000**

The segments of the NEI where future capacity expansions could be delayed if the Mendota WTP were constructed by 2025 are listed in the following table along with the earliest date expansion would be required without the Mendota WTP and the expected date expansion would be required it the Mendota WTP were built.

NEI Segment	Current Capacity (mgd)	Earliest Year Capacity Expansion Needed	Year Capacity Expansion Needed with Mendota WTP
PS 10 FM	36.5	2040	Beyond 2060
13Hii	24.55	2026	Beyond 2060
13Hi	26.70	2037	Beyond 2060
13A-Ei	24.00	2041	Beyond 2060
13G	20.75	2056	Beyond 2060

Implementation of this project would have no impact on the NSWTP phosphorus-related additions in 2020; however, the 2030 NSWTP addition to address nitrogen would require a smaller capacity if the Mendota WTP was implemented by then, and the need to expand the biosolids facilities at the NSWTP would be delayed.

B. Project E2–Starkweather Creek WWTP

This project would redirect the gravity flow tributary to PS 13 to a Starkweather Creek WWTP located northeast of the Dane County Regional Airport. Effluent from this facility could provide stream flow augmentation to Starkweather Creek, provide effluent for wetland restoration at Cherokee Marsh, recharge the groundwater by infiltration, or serve as reuse water or water for turf irrigation. Figure 2.03-2 shows the potential sewer service area for this facility and the effluent flows associated with this project for the Starkweather WTP, the Yahara River, Badfish Creek, Badger Mill Creek, and the Sugar River. The design flows and BOD_5 loadings for this facility are as follows.

Population Projection	ADF (mgd)	PHF (mgd)	Design Flow (mgd)	BOD_5 Loading (lbs/day)
2030 Low	2.74	9.35	4.67	6,855
2030 High	3.89	12.55	6.28	9,733
2060 High	3.89	12.55	6.28	9,733

Because of current legislative and regulatory constraints and the time necessary to gain public acceptance for a treatment plant and effluent discharge at this location, the ability to site and operate a WWTP near the Dane County Regional Airport is not expected to be possible before 2025. There are several components of the conveyance system downstream of this site that will reach capacity before 2025. The District has plans in place to address these components, including the construction of PS 18 that will provide backup capacity for PS 7 and relief for the SEI downstream of its juncture with the NEI, as shown in the following table.

Project Description	Interceptor Segments	Project Start Date	Project Cost
NEI Relief Upstream of PS 10	10 B to 10 H	2008	$10,200,000
PS 18		2010	$8,500,000
PS 18 Force Main		2010	$8,500,000
NEI–from FEI to PS 18	7Fi, 7Fii	2012	$5,260,000
PS 7 Improvements		2013	$1,110,000
NEI Relief Upstream of PS 10	10A	2016	$10,000,000
Total			**$43,570,000**

Capacity expansion of the PS 10 force main can be delayed if the Starkweather WTP is constructed by 2025. If this plant is not built, the PS 10 force main will require capacity expansion as early as 2040. If the Starkweather WTP is built, this expansion will not be required until sometime after 2060.

Implementation of this project would have no impact on the NSWTP phosphorus-related additions in 2020; however, the 2030 NSWTP addition to address nitrogen would require a smaller capacity if this project was implemented by then, and the need to expand the biosolids facilities at the NSWTP would be delayed.

S:\MAD\1500--1599\1547\001\Data\GIS\Figures\TechnicalMemo7\Figure 2.03-2 Alternative E2 11x17.mxd

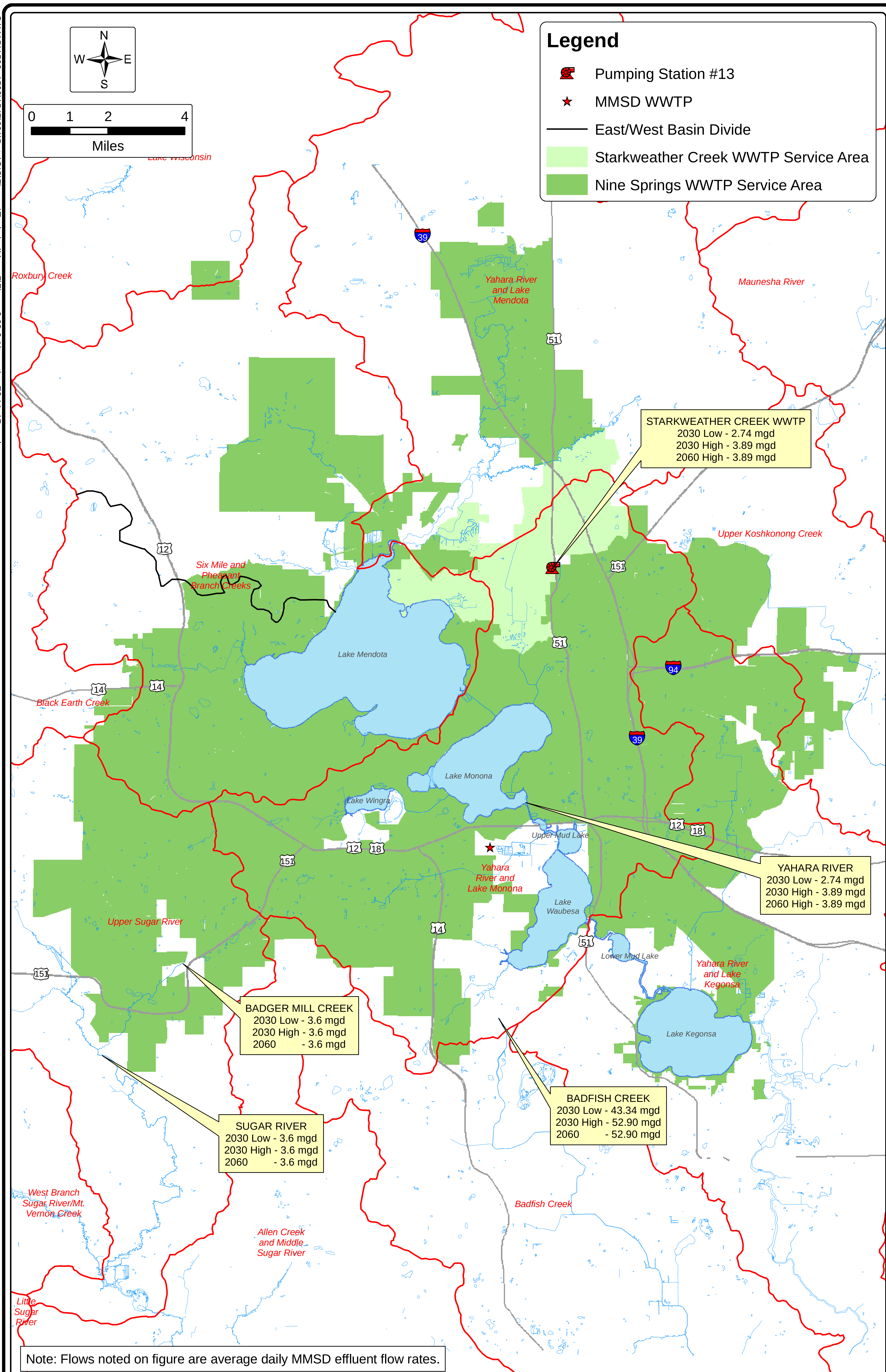


FIGURE 2.03-2
1547.001

STRAND ASSOCIATES, INC.® ENGINEERS

MALCOLM PIRNIE

MMSD

ALTERNATIVE PROJECT E2
STARKWEATHER CREEK WWTP

TECHNICAL MEMO 7
ALTERNATIVES DEVELOPMENT
MADISON METROPOLITAN SEWERAGE DISTRICT

C. Project E3–PS 13 and PS 14 Service Area WWTP

This project would redirect the flow tributary to PS 13 and PS 14 to a Combined PS 13 and PS 14 WWTP located northeast of the Dane County Regional Airport. Effluent from this facility could provide stream flow augmentation to Starkweather Creek, provide effluent for wetland restoration at Cherokee Marsh, recharge the groundwater by infiltration, or serve as reuse water or water for turf irrigation. Figure 2.03-3 shows the potential sewer service area for this facility and the effluent flows associated with this project for the PS 13 and PS 14 Service Area WTP, the Yahara River, Badfish Creek, Badger Mill Creek, and the Sugar River. The design flows and BOD_5 loadings for this facility are as follows.

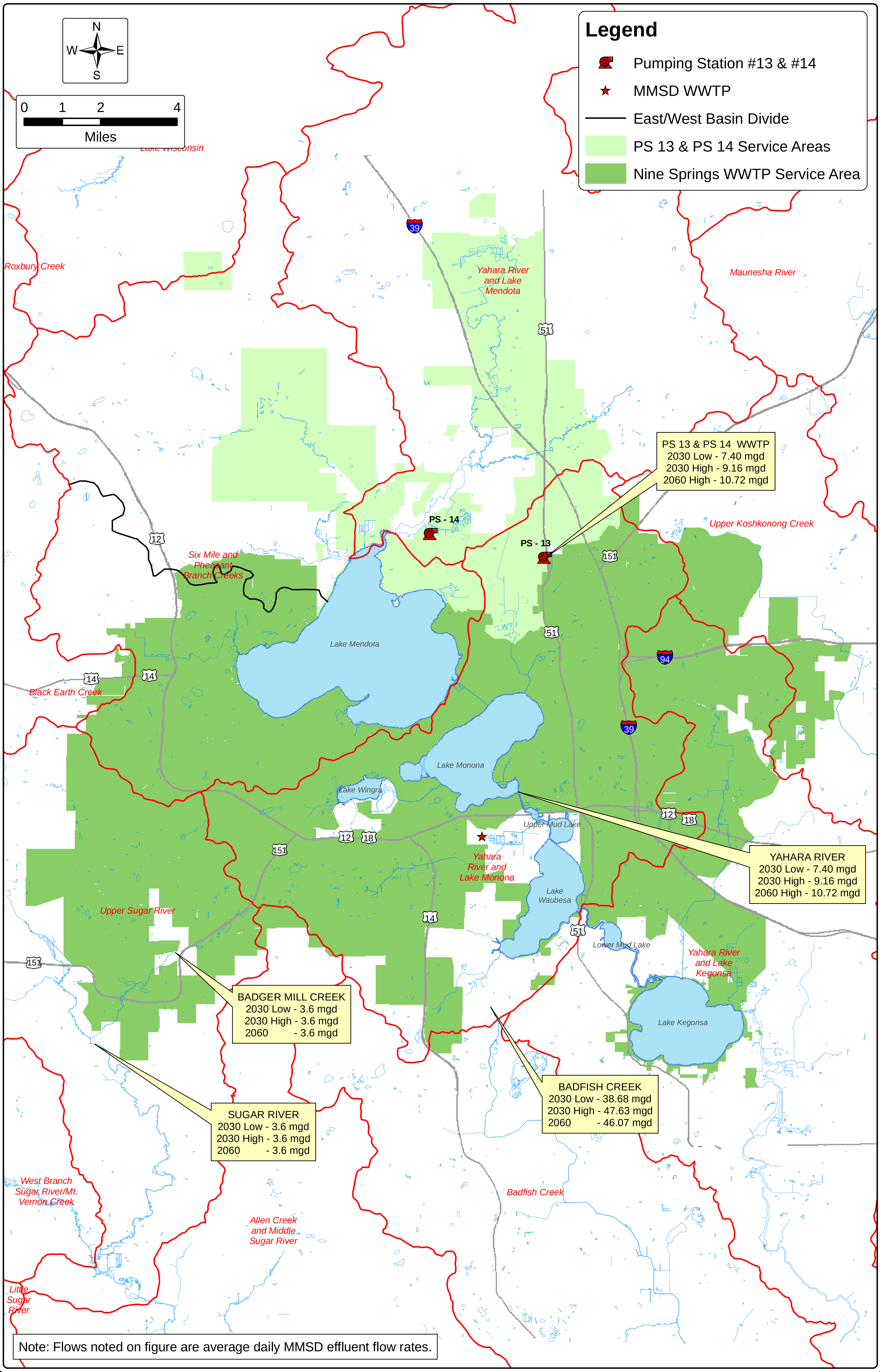
Population Project	ADF (mgd)	PHF (mgd)	Design Flow (mgd)	BOD Loading (lbs/day)
2030 Low	7.40	21.58	10.79	18,515
2030 High	9.16	25.82	12.91	22,918
2060 High	10.72	29.48	14.74	26,821

Because of current legislative and regulatory constraints and the time necessary to gain public acceptance for a treatment plant and effluent discharge at this location, the ability to site and operate a WWTP near the Dane County Regional Airport is not expected to be possible before 2025. There are several components of the conveyance system downstream of this site that will reach capacity before 2025. The District has plans in place to address these components, including the construction of PS 18 that will provide backup capacity for PS 7 and relief for the SEI downstream of its juncture with the NEI, as shown in the following table.

Project Description	Interceptor Segments	Project Start Date	Project Cost
NEI Relief Upstream of PS 10	10 B to 10 H	2008	$10,200,000
PS 18		2010	$8,500,000
PS 18 Force Main		2010	$8,500,000
NEI–from FEI to PS 18	7Fi, 7Fii	2012	$5,260,000
PS 7 Improvements		2013	$1,110,000
NEI Relief Upstream of PS 10	10A	2016	$10,000,000
Total			**$43,570,000**

Capacity expansion of the PS 10 force main can be delayed if the PS 13 and PS 14 Service Area WTP is constructed by 2025. If this plant is not built, the PS 10 force main will require capacity expansion as early as 2040. If the PS 13 and PS 14 Service Area WTP is built, this expansion will not be required until sometime after 2060.

Implementation of this project would have no impact on the NSWTP phosphorus-related additions in 2020; however, the 2030 NSWTP additions to address nitrogen would require a smaller capacity if this project was implemented by then, and the need to expand the biosolids facilities at the NSWTP would be delayed.



S:\MAD\1500--1599\1547\001\Data\GIS\Figures\TechnicalMemo7\Figure 2.03-3 Alternative E3 11x17.mxd

FIGURE 2.03-3
1547.001

STRAND ASSOCIATES, INC.® ENGINEERS

MALCOLM PIRNIE

ALTERNATIVE PROJECT E3
PS 13 & PS 14 SERVICE AREAS WWTP

TECHNICAL MEMO 7
ALTERNATIVES DEVELOPMENT
MADISON METROPOLITAN SEWERAGE DISTRICT

D. Project E4–Stoughton WWTP Expansion

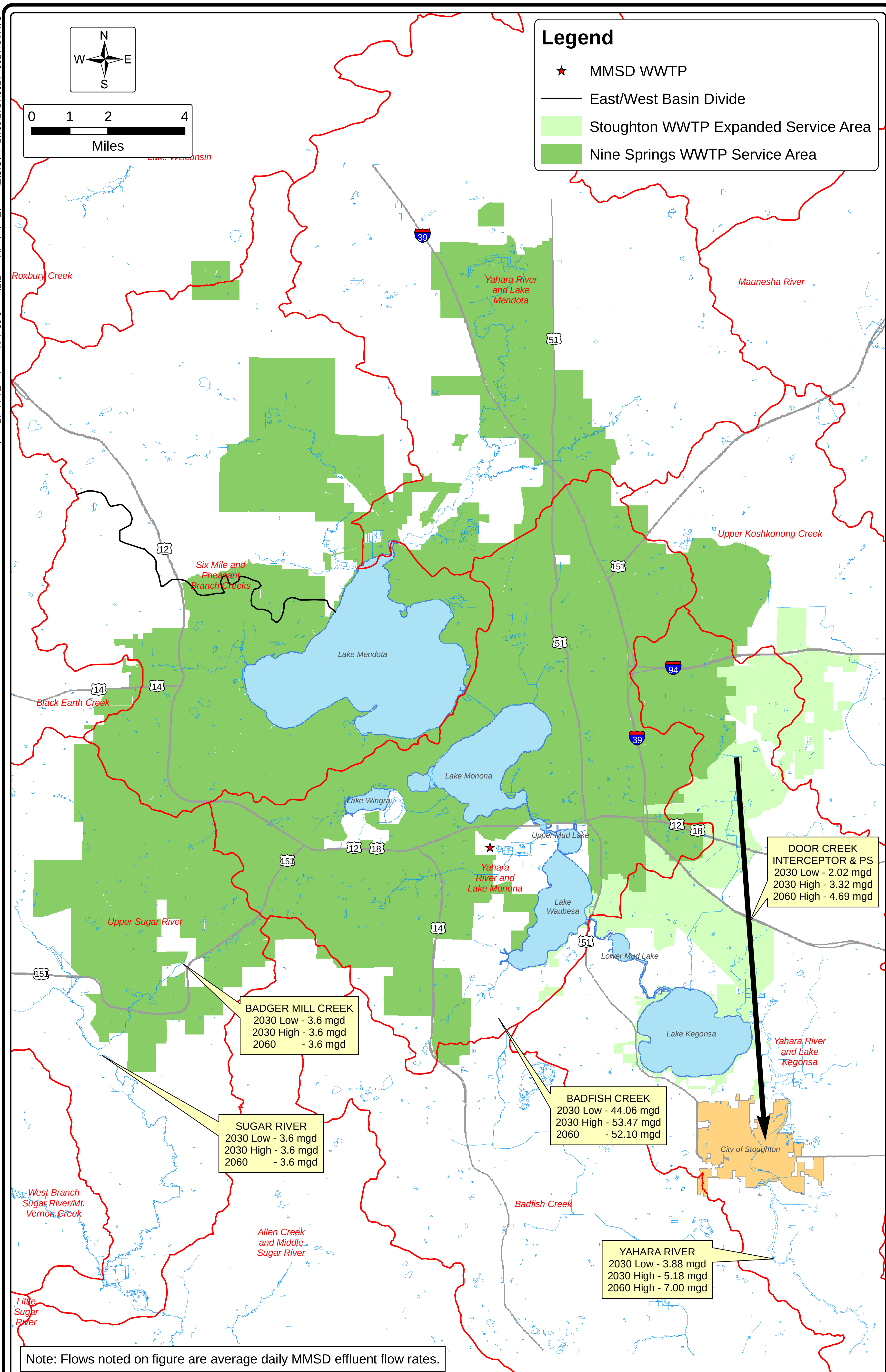
This project would redirect flow from the Door Creek watershed in the PS 7 and PS 9 service areas to an expanded City of Stoughton WWTP. Based on the population projections for Stoughton prepared by CARPC, the existing Stoughton WTP would need to meet the following design conditions to serve only the Stoughton area.

Population Projection	Population	ADF (mgd)	PHF (mgd)	BOD_5 Loading (lbs/day)
2000	12,671	1.27	4.28	3,178
2030	18,609	1.86	6.26	4,653
2060	23,064	2.31	7.78	5,779

Implementation of this project includes the construction of a parallel treatment plant to treat the wastewater diverted from the MMSD system. Biosolids treatment would be provided by expanding the existing biosolids treatment train at the Stoughton WTP.

The proposed service area for this expanded Stoughton plant is shown in Figure 2.03-4. Figure 2.03-4 also shows the average daily wastewater flows diverted to the Stoughton WTP and the average daily effluent flows to the Yahara River, Badfish Creek, Badger Mill Creek and the Sugar River. This project would include a relief force main and interceptor from PS 9, a pumping station located on the northeast side of Lake Kegonsa, a force main from the pumping station to the Stoughton WWTP, and a hydraulic and biosolids expansion at the Stoughton WWTP. In addition, the interceptor paralleling Door Creek would need to be increased in size from a 30-inch pipe to a 36-inch pipe and extended to the site of the new pump station on the northeast side of Lake Kegonsa. The additional proposed design flows and BOD_5 loadings from MMSD that would be diverted to the Stoughton WWTP are shown in the following table.

Service Area	2030 Low	2030 High	2060 High
7B	0.98	1.02	1.57
7H-2030	0.1	.01	.01
7J-2030 and 2060	.21	1.28	1.64
9A	0.73	0.92	1.38
Total ADF (mgd)	2.02	3.32	4.69
PHF (mgd)	7.23	10.99	14.70
BOD Loading (lbs/day)	5,054	8,307	11,734



S:\MAD\1500--1599\1547\001\Data\GIS\Figures\TechnicalMemo7\Figure 2.03-4 Alternative E4 11x17.mxd

FIGURE 2.03-4
1547.001

STRAND ASSOCIATES, INC.® ENGINEERS

MALCOLM PIRNIE

ALTERNATIVE PROJECT E4
STOUGHTON WWTP EXPANSION

TECHNICAL MEMO 7
ALTERNATIVES DEVELOPMENT
MADISON METROPOLITAN SEWERAGE DISTRICT

The resulting design flows and BOD_5 loadings for the combined MMSD and Stoughton service area would be as follows.

Population Projection	ADF (mgd)	PHF (mgd)	BOD_5 Loadings (lbs/day)
2030 Low	3.88	12.53	8,232
2030 High	5.18	15.98	12,960
2060 High	7.00	20.58	17,513

The Stoughton WTP has a current hydraulic capacity of 2.35 mgd and a current average daily flow of 1.6 mgd. The City of Stoughton is considering adding additional treatment capacity to address organic loadings and is waiting to receive their new WPDES permit before making a final decision on when this additional capacity may be required. A more detailed investigation of the potential benefits of a regional solution involving the City of Stoughton and MMSD could be undertaken as part of planning for the next addition to the Stoughton WTP. Based on discussions with the City of Stoughton, they are uncertain that the city would realize sufficient benefits from regionalization to justify participation in a regional project at this time. To adequately investigate a regional alternative and gain the City of Stoughton's acceptance would take time. As a result, the earliest a regional project involving the City of Stoughton could be initiated is 2015.

The conveyance system components impacted by this project that will have reached their design capacity by 2015 are shown in the following table. The District has plans in place to address these components, including the construction of PS 18 that will provide backup capacity for PS 7 and relief for the SEI downstream of its juncture with the NEI, as shown in the following table.

Project Description	Interceptor Segments	Project Start Date	Project Cost
PS 18		2010	$8,500,000
PS 18 Force Main		2010	$8,500,000
NEI–from FEI to PS 18	7Fi, 7Fii	2012	$5,260,000
PS 7 Improvements		2013	$1,110,000
Total			**$23,370,000**

The Village of Cottage Grove has constructed a new pumping station and force main. The new force main will convey flow to the Far East Interceptor–Cottage Grove Extension (Segment 7B) up to the 2.71 mgd capacity of this gravity sewer. Flows in excess of 2.71 mgd will be conveyed to the upstream end of Segment 7Civ on the Door Creek Extension of the Far East Interceptor. Rerouting flows from areas 7B and 7J (the Village of Cottage Grove and the area in the Door Creek watershed south and west of the Village of Cottage Grove) delays the need to provide capacity expansions for approximately 12,400 feet of relief sewers for the Far East Interceptor as shown in the following table.

FEI Interceptor Segment	Current Capacity (mgd)	Earliest Year Capacity Expansion Needed	Year Capacity Expansion Needed with Stoughton WTP
7E	15.92	2052	Beyond 2060
7Diii	15.92	2053	Beyond 2060
7Dii	7.49	2018	2033
7Di	12.19	2035	Beyond 2060
7Civ	7.12	2025	Beyond 2060

If wastewater flows from Area J are to be treated at the NSWTP, Area J will be served by an interceptor that would parallel Door Creek and terminate at a pumping station near the point where Door Creek crosses Interstate Highway 90, east of the Village of McFarland. The pumping station at this location would discharge through a force main to the SEI Blooming Grove Extension near the Highway 12 and 18 and Interstate 90 interchange. If flows from the Village of Cottage Grove and Area J are to be diverted to the Stoughton WTP as proposed in this project, the 35,000 feet of interceptor paralleling Door Creek from the Village of Cottage Grove to the northeast side of Lake Kegonsa will need to be a 36-inch pipe, rather than a 30-inch pipe if only Area J is served. The pumping station and force main for discharge to the SEI Blooming Grove Extension will not be required under this project.

Rerouting flows from areas 7H and 9A (the Village of McFarland and the area east of the village in the Door Creek watershed) delays the need to provide capacity expansions for approximately 15,000 feet of relief sewers for the Southeast Interceptor as shown in the following table. A new PS 9 force main and interceptor will be needed as part of this project to convey wastewater 20,000 feet to the northeast side of Lake Kegonsa.

SEI Interceptor Segment	Current Capacity (mgd)	Earliest Year Capacity Expansion Needed	Year Capacity Expansion Needed with Stoughton WTP
7Kiv	11.4	2039	Beyond 2060
7Kiii	10.55	2032	Beyond 2060
7Kii	10.26	2029	Beyond 2060
7Ki	10.26	2034	Beyond 2060
7Jiii	5.06	2029	Beyond 2060
7Jii	3.87	2022	Beyond 2060
7Ji	2.25	2012	Beyond 2060

The capacities of the 2020 and 2030 NSWTP additions to address phosphorus and nitrogen would be reduced if this project was implemented by 2020, and the need to expand the biosolids facilities would be delayed.

E. Project E5–Centralized High Quality Effluent Treatment Facilities

This project includes the construction of a high quality effluent treatment plant on the NSWTP property. The year of implementation and the capacity of this facility would depend on the District's policies, economics, and the need for a high quality effluent. Three possible scenarios that could lead to the implementation of this project include:

1. The construction of a demonstration facility that incorporates different treatment technologies. Effluent from such a facility could be discharged to Nine Springs Creek, or used for stream flow augmentation, industrial reuse, turf irrigation, or infiltration. This would allow the study of various treatment processes at a production scale and the impacts of different quality effluents on the receiving environment in advance of implementing larger scale projects. This project could be implemented as early as the District desired, subject to regulatory approval.

2. The average daily flow received at the NSWTP will reach its rated design capacity of 57 mgd no earlier than 2050. If there is a defined need for a high quality effluent before then, this project could be implemented at that time. This plant would receive the current NSWTP effluent and provide additional treatment to meet the required characteristics for its intended use. This option, coupled with the appropriate effluent return facilities, should be compared to each satellite treatment plant option for costs, environmental impacts, public acceptance, and operational considerations.

3. When the frequency of effluent discharges from the effluent equalization facilities results in unacceptable environmental impacts, this project could be implemented to avoid any expansion of the current Badfish Creek effluent pumping capacity at the NSWTP. Flows to this facility would receive preliminary treatment and primary treatment at the existing NSWTP. Biosolids would be processed through the existing anaerobic digestion facilities at the NSWTP. Effluent from this facility would be of a quality suitable for return to Lake Waubesa or for reuse. The biological, filtration and disinfection portions of this facility would be sized at a 2 to 1 peaking factor consistent with the requirements of the technology required to produce a higher than advanced secondary quality water.

Construction of the high quality effluent treatment facilities at the NSWTP will not relieve any conveyance capacity needs. It could negate or delay the need to expand the advanced secondary treatment facilities at the NSWTP. All wastewater would continue to be pumped to the NSWTP.

F. Project E6–Sun Prairie WTP

This project provides sewer service for the portion of the District's future service area in the Koshkonong Creek watershed (Service Area 7C) by directing flow from this area to the City of Sun Prairie WWTP. Figure 2.03-5 shows this area, the average wastewater flow from this area to the Sun Prairie WTP, and the resulting effluent flows to Badger Mill Creek, the Sugar River and Badfish Creek.

The Sun Prairie WTP has an average day hydraulic capacity of 4.4 mgd with a peak month capacity of 6.2 mgd. The current average daily flow is 3.2 mgd. The City of Sun Prairie completed an upgrade of its WTP in 2007. The next capacity expansion is expected to be required in 2015 to 2020. A more detailed investigation of the potential benefits of a regional solution involving the City of Sun Prairie and MMSD could be undertaken as part of planning for the next addition to the Sun Prairie WTP. This timing would work well with the projected development timeline for area 7C.

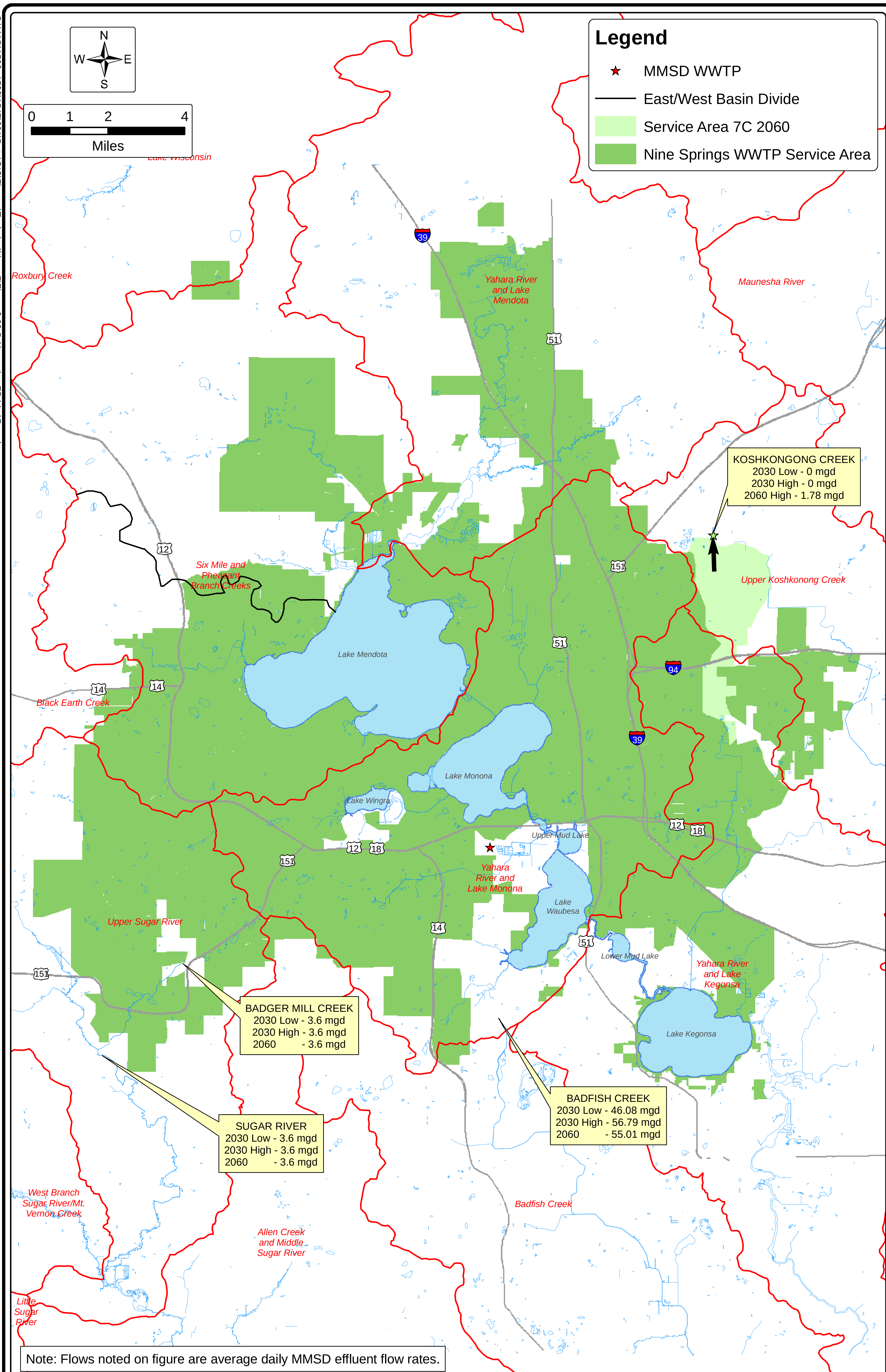


FIGURE 2.03-5
1547.001

STRAND ASSOCIATES, INC. ENGINEERS

MALCOLM PIRNIE

MMSD

ALTERNATIVE PROJECT E5
SUN PRAIRIE WWTP EXPANSION

TECHNICAL MEMO 7
ALTERNATIVES DEVELOPMENT
MADISON METROPOLITAN SEWERAGE DISTRICT

Development in the part of area 7C in the Koshkonong Creek watershed is not expected to begin until 2030. The projected growth in area 7C between 2030 and 2060 will generate an average day wastewater flow of 1.78 mgd with a corresponding peak hourly flow of 6.50 mgd. If this project is implemented at the time development begins in this area, it will eliminate the need for capacity expansion in Segment 7Civ of the FEI Door Creek Extension. Three segments of the FEI Door Creek Extension upstream of Segment 7Civ could require capacity expansion prior to 2030 as shown in the following table. If an additional 0.47 mgd of wastewater from areas north of Interstate Highway 94 in the Door Creek watershed were also directed to the Sun Prairie WTP, rather than to the FEI Door Creek Extension, the need to increase the capacity in any of the 7C interceptor segments could be eliminated.

FEI Door Creek Extension Segment	Current Capacity (mgd)	Earliest Year Capacity Expansion Needed
7Ciii	5.98	2025
7Cii	5.41	2022
7Ci	4.36	2018

Implementation of this project would have no impact on the NSWTP phosphorus-related additions in 2020; however, the 2030 NSWTP additions to address nitrogen would require a smaller capacity if the Sun Prairie WTP project was implemented by then, and the need to expand the biosolids facilities at the NSWTP would be delayed.

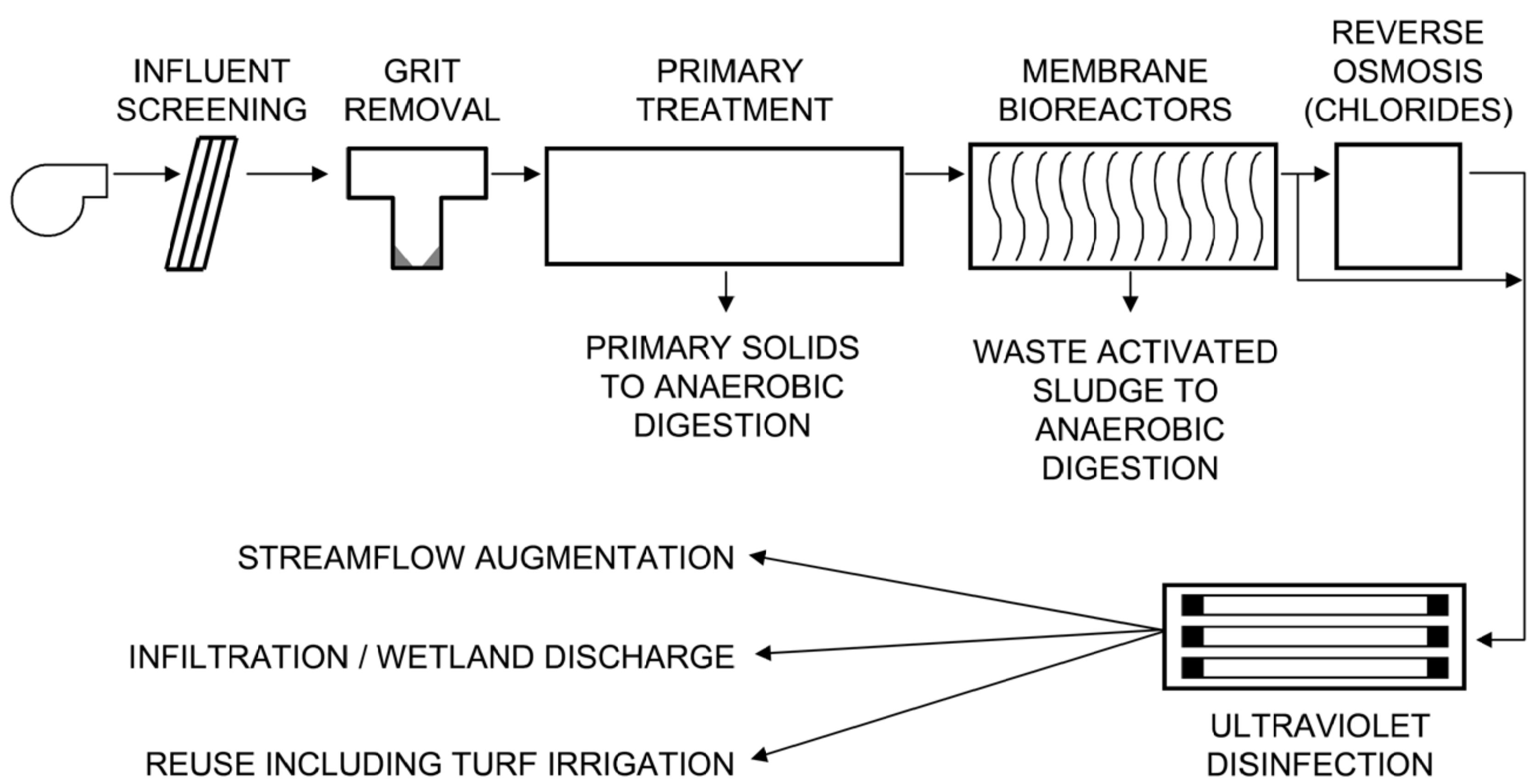
2.04 SATELLITE WASTEWATER TREATMENT PLANTS–FORWARD FLOW TREATMENT PROCESSES

The proposed forward flow wastewater treatment processes for all high quality satellite treatment plants is shown schematically in Figure 2.05-1. These facilities would be designed to meet the most stringent applicable limits required for discharge to exceptional resource water streams (e.g., the Sugar River), infiltration for groundwater recharge, or effluent reuse for either industrial use or turf irrigation. Figure 2.05-2 shows a proposed process schematic for the NSWTP high quality effluent treatment facilities.

2.05 SATELLITE WASTEWATER TREATMENT PLANTS–BIOSOLIDS PROCESSES

Proposed wastewater treatment facilities with a design ADF less than 4 mgd would have an aerobic biosolids stabilization scheme. Depending on the economics, aerobic stabilization would be provided at the site, or the biosolids would be conveyed to the NSWTP for anaerobic digestion.

Proposed wastewater treatment facilities with a design ADF greater than 4 mgd would have either on-site anaerobic digestion, depending on the site, or biosolids would be conveyed to the NSWTP for anaerobic digestion.



HIGH QUALITY WWTP
PROCESS SCHEMATIC

TECHNICAL MEMO 7
ALTERNATIVES DEVELOPMENT
MADISON METROPOLITAN SEWERAGE DISTRICT

STRAND ASSOCIATES, INC.® ENGINEERS

FIG. 2.05-1
1547.001

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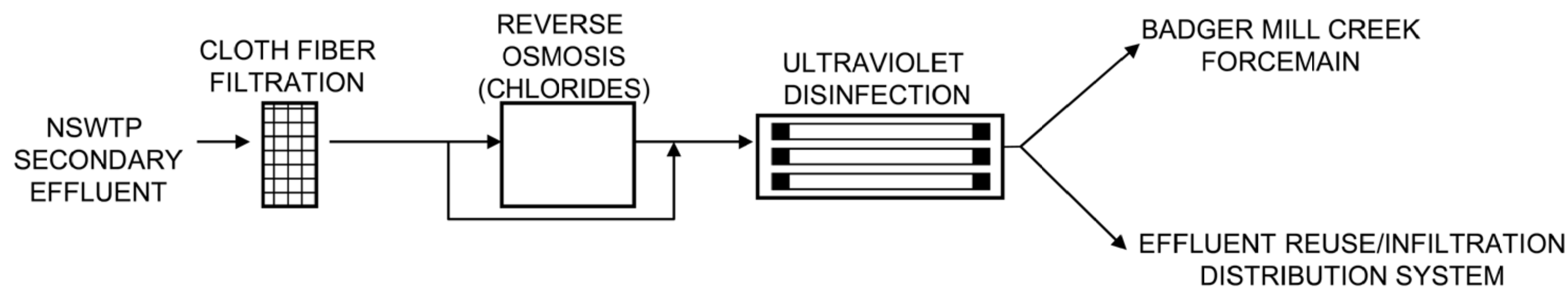

HIGH QUALITY NSWTP
PROCESS SCHEMATIC
50 YEAR MASTER PLAN

MADISON METROPOLITAN SEWERAGE DISTRICT
MADISON, WISCONSIN

STRAND ASSOCIATES, INC.® ENGINEERS

FIG. 2.05-2
1547.001

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SECTION 3
MASTER PLAN ALTERNATIVE DEVELOPMENT

3.01 MASTER PLAN ALTERNATIVES DEVELOPMENT (2010 to 2030)

This section presents the preliminary screening of projects to develop master plan alternatives. Following sections present a detailed description of each alternative that will provide the basis for alternative evaluations to be presented in Technical Memo 9.

Key principles used to incorporate near-term projects (2010 to 2030) into Master Plan alternatives include the following:

1. The proposed alternative project must have an implementation date that would allow sufficient time for the District to site and construct the alternative project prior to the time necessary to alleviate an existing MMSD capacity need.

2. Alternatives will include sufficient capacity so that any future expansion of the current advanced secondary treatment facilities at the NSWTP beyond the 57 mgd capacity will not be required before 2060.

Based on these criteria, the following projects will not be included separately in the development of alternatives. Discussions of the reasons they are not included are noted in the following paragraphs.

1. Projects W1A, W1B, and W1C (NSVI Relief)–These three alternative projects for providing additional capacity in the Nine Springs Valley Interceptor will be incorporated into the baseline alternative since they do not involve adding any new WWTP capacity. Life-cycle costs for these three projects will be compared, and the project with the lowest cost will be included in the baseline alternative.

2. Project W3 (Dual Sugar River Satellite Plants)–This alternative is similar to Project W2. Project W2 at the projected ADF and PHF will eliminate the need for any capacity increases in the NSVI before 2060. Discharge at the locations defined in Project W3 could also be accomplished by pumping wastewater from NSWTP to Badger Mill Creek or via an effluent force main from the proposed Sugar River WWTP. Project W3 could alleviate firm capacity deficits at PS 11 and PS 12, but implementation would likely not be possible prior to the time these expansions would be necessary.

3. Project W4 (Village of Oregon Discharge to PS 11)–This is an operational reserve project for a potential annexation of the Village of Oregon by the District with treatment of the Village's wastewater at the NSWTP. This project does not include additional treatment capacity away from the NSWTP.

4. Project E1 (Mendota WTP)–This project cannot be implemented in a near-term (2110 to 2025) timeframe that would alleviate the need for downstream interceptor, pumping station, or force main capacity expansion. However, it remains a future option (2025 to 2060) to alleviate capacity expansion at the NSWTP or to provide for a more local source of effluent for either infiltration or reuse. The need to provide additional base flow in the Yahara River upstream of Lake Mendota has not been established. Elements of this project will be included in alternative evaluations for the longer term (2030 to 2060) presented in Section 4.

5. Projects E2 (Starkweather Creek WTP) and E3 (PS 13 and PS 14 Service Area WWTP)–These projects cannot be implemented in a near-term timeframe that would alleviate the need for downstream interceptor, pumping station, or force main capacity expansion. However, they remain as future options (2025-2060) to alleviate capacity expansion at the NSWTP or to provide a more local source of water for infiltration, reuse, or potential stream flow augmentation for Starkweather Creek. Elements of these projects will be included in alternative evaluations for the longer term (2030 to 2060) presented in Section 4.

6. Project E4 (Stoughton WTP Expansion)–This project eliminates future interceptor capacity improvements for interceptor segments: 7Civ, 7Di, 7Diii, 7E, 7Ji, 7Jii, 7Jiii, 7Ki, 7Kii, 7Kiii, 7Kiv, 9A, 9Bi, and 9Bii. The need to expand the capacity of segment 7Dii is delayed by 15 years, from 2018 until 2033. This project includes an expansion of the forward flow facilities and anaerobic digestion facilities at the Stoughton WWTP, a new pump station located on the northeast side of Lake Kegonsa with a force main from there to the Stoughton WTP, and a new force main and interceptor from PS 9 to the Lake Kegonsa pumping station. This project also requires the Area J interceptor to be a 36-inch-diameter pipe, rather than a 30-inch-diameter pipe under the base conditions, but eliminates a proposed pumping station and force main to convey flow from the downstream end of the Area J interceptor to the Southeast Interceptor–Blooming Grove Extension.

   The projected costs savings from eliminating future capacity improvements for interceptor segments: 7Civ, 7Di, 7Dii 7Diii, 7E, 7Ji, 7Jii, 7Jiii, 7Ki, 7Kii, 7Kiii, 7Kiv, 9A, 9Bi, and 9Bii is estimated to be about $13,000,000. Costs for the Door Creek Interceptor and related components would for this project be substantially greater than the potential savings of $13,000,000. A capacity expansion for the Stoughton WWTP would likely cost more than the potential savings itself.

   In addition, The City of Stoughton at the present time does not have an interest in providing wastewater service for any of the Madison Metropolitan Sewerage District Sewer Service Area.

7. Project E6 (Sun Prairie WTP Expansion)–This project would not be implemented before 2030. Although it may be a viable project, it is not a near-term option.

The following projects will be incorporated into the 50-year Master Plan near-term (2010 to 2030) detailed alternative evaluations.

1. Project W2 (Sugar River WTP)–This alternative at the projected ADF and PHF will alleviate the needs for any expansion of the NSVI before 2060 and reduce the capacity expansion needs for PS 11 and PS 12. Coupled with a return of 3.6 mgd from the NSWTP, this project would return the equivalent volume of water generated in the Sugar River watershed to the Sugar River watershed.

2. Project E5 (Centralized High Quality Effluent Treatment Facilities)–This project is an extension of the base-line project if none of the other projects were implemented before the capacity needs of the NSWTP exceed 57 mgd. This could also become an option for providing high quality effluent water throughout the district from a centralized location.

The following master planning alternatives, which are combinations of the projects described above, will be compared to the baseline alternative of expansion for all increases in flow and biosolids at the existing NSWTP:

3.02 ALTERNATIVE MP1–BASELINE-WESTSIDE CONVEYANCE SYSTEM EXPANSION

Under this alternative all wastewater will continue to be conveyed to the NSWTP. Costs for this alternative include all conveyance system items included with the NSVI, PS 11, and PS 12 that would be either eliminated, reduced in capacity, or delayed with construction of Project W2–Sugar River plant. This alternative is directly comparable to Alternatives MP2A and MP2B detailed in Section 3.03. This alternative also includes costs for a high quality effluent treatment plant at the NSWTP. The high quality effluent from the NSWTP would be pumped to the Sugar River basin.

3.03 ALTERNATIVE MP2-SUGAR RIVER WWTP

This alternative includes the costs associated with construction of a 4.27 mgd high quality effluent WWTP in 2020 with discharge to either the main branch of the Sugar River downstream of the confluence with Badger Mill Creek (Alternative MP2A) or discharge to the Sugar River in the vicinity of CTH PD (Alternative MP2B).

Construction of the Sugar River WWTP would eliminate the need to provide additional capacity in the NSVI and at PS 12. The need to address capacity expansion at PS 11 would be delayed by 35 years, until 2050.

The Sugar River WTP, for purposes of alternative evaluation, would have an average daily flow of 4.27 mgd with a peak hourly flow of 13.6 mgd. The biological, filtration and disinfection portions of this facility would be sized for an equivalent 2 to 1 peaking factor to provide for better overall operation. This facility is of sufficient size at the 2060 design flows to support anaerobic digestion. This facility would be designed to produce effluent that would be suitable for discharge to the Sugar River, infiltration, effluent reuse, wetlands discharge, and turf irrigation. Alternative MP2B includes all the costs for Alternative MP2A plus the costs associated with providing a pump station and force main from the new plant to the Sugar River near CTH PD.

With the return of 3.6 mgd to Badger Mill Creek from the NSWTP, the amount of water generated in the District service area in the Sugar River basin would equal the amount of water returned to the Sugar River basin.

This alternative is directly comparable to Alternative MP1 described in Section 3.02.

3.04 ALTERNATIVE EVALUATION SUMMARY

Based on the screening of alternative projects presented in this Technical Memo, detailed evaluations of near-term (2010 to 2030) alternatives to be included in Technical Memo 9 are as follows.

1. Alternative Evaluation 1–In this evaluation, Alternative MP1 (Baseline) would be compared against Alternative MP2A and MP2B (Sugar River WWTP). Alternative MP1 costs will include the costs associated with construction of additional phosphorus treatment facilities at the NSWTP in 2020, denitrification facilities and an expansion of the biosolids treatment facilities at the NSWTP in 2030, a relief sewer or force main for the NSVI in 2020, and capacity expansions at PS 11 and PS 12 in 2010. Alternative MP2A costs would include the costs of construction a Sugar River WWTP in 2020 with an ADF of 4.27 mgd, additional phosphorus treatment facilities at the NSWTP in 2020, denitrification facilities at the NSWTP in 2030, an expansion of the biosolids treatment facilities at the NSWTP in 2040, and capacity expansions at PS 11 and PS 12 in 2010. Alternative MP2B costs would equal the costs for Alternative MP2A plus the costs for an effluent pumping station at the Sugar River WTP site with a force main to a discharge point on the Sugar River at CTH PD.

SECTION 4
LONG-TERM MASTER PLAN ALTERNATIVES DEVELOPMENT

4.01 LONG-TERM MASTER PLAN ALTERNATIVES DEVELOPMENT (2030 TO 2060)

Several alternative projects presented in Section 2 could not be implemented soon enough to provide near-term capacity relief for the conveyance system. However, they remain potentially viable options beyond the year 2030. Specifically, Projects E1-Mendota WWTP, E2-Starkweather Creek WWTP, and E3-PS 13 and PS 14 Service Area WWTP remain as viable options for providing high quality effluent for various uses at these locations. Project E6–Sun Prairie WWTP Expansion may be a viable project for providing relief in the conveyance system and mitigating inter-basin transfers of water.

For comparison purposes, alternate projects that would align with Projects E1, E2 and E3 would include centralized treatment of all wastewater at the NSWTP, including treatment to a higher quality for a portion of the effluent that would then be pumped back to the Project E1, E2 and E3 sites. Project E5-Centralized High Quality Effluent Treatment Facilities provides the basic description of this type of project.

The need for high quality effluent is not definitively known at this time, but additional demands on available groundwater supplies coupled with the long-range goal of stabilizing the groundwater aquifer operating level make this need a distinct possibility in the future, especially if population growth occurs as expected.

Another approach to decentralizing treatment to mitigate interbasin water transfers would be to locate a satellite plant at the site of a defined water reuse user. Such plants would be sized based on the reuse volume requirements, and as such, might not provide meaningful conveyance system relief. Such an approach might work well on the east side of the District since most of the conveyance capacity expansions will be in place before any type of satellite plant could be constructed in this area. This may result in a larger number of smaller satellite treatment and reuse facilities. This approach could potentially save on the costs of constructing an effluent reuse distribution system and reduce the amount of energy required to operate such as system. Since the conveyance capacity for peak flows will be constructed in advance of the timing for installation of local high quality effluent facilities, the need to treat peak flows in these smaller facilities would not be required. It is also possible that the facilities could be run only when the effluent reuse water is required (e.g., summer operation for turf irrigation).

The following projects will not be included separately in the development of long-term alternatives. Discussions of the reasons they are not included are noted in the following paragraphs.

1. Project E6 (Sun Prairie WTP Expansion)–Although this may be a viable project, the rate of growth in Sun Prairie and on the east side of the District is uncertain. The District should evaluate this option in the future, but there are too many unknowns to provide a meaningful evaluation of this project at this time.

2. Projects involving small satellite plants sized to address a specific reuse demand will not be included due to the many unknowns associated with such a plant, including location, timing, and demand volume.

The long-term (2030 to 2060) alternative projects that will be included in the evaluations in Technical Memo 9 follow:

1. MP 3-Project E5–Centralized High Quality Effluent Treatment and Distribution Facilities. This project includes the costs associated with construction of facilities at the NSWTP for providing effluent of a quality for reuse for various options including stream flow augmentation in Starkweather Creek, infiltration, industrial reuse, and turf irrigation. Such a facility would be constructed in 2030 with a capacity of 4 mgd (MP3A) or 10 mgd (MP3B). Project MP3 is directly comparable to Project MP4A, and Project MP3B is directly comparable to Project MP4B. Project MP3A and MP3B will include the costs associated with providing additional facilities at the NSWTP to produce a high quality effluent as well as the costs associated with pumping facilities to return the water to the PS 13 site.

2. MP 4–Decentralized High Quality Effluent Treatment Facilities (Projects E2 and E3). This project includes the costs associated with providing the decentralized treatment facilities defined in Project E2 (MP4A) and Project E3 (MP4B). These facilities would include the necessary pumping, screening, biological treatment (likely membrane bioreactors), disinfection and additional treatment as necessary to meet the effluent reuse needs and would be constructed in 2030.

4.02 LONG-TERM ALTERNATIVE EVALUATIONS (2030 TO 2060)

The following long-term (2030 to 2060) alternative evaluation will be included in Technical Memo 9:

Alternative Evaluation 2–In this evaluation, Alternatives MP3A and MP3B (Centralized High Quality Effluent Treatment and Distribution) will be compared to Alternatives MP4A and MP4B, respectively (Decentralized High Quality Effluent Treatment). Each cost analysis will compare the life-cycle costs of a high quality effluent treatment facility and pumping system at the NSWTP with a force main to the PS 13 site to the life-cycle costs of a decentralized high quality effluent treatment facility at the site of PS 13.

Technical Memorandum No.8

Date:	October 27, 2009
To:	Madison Metropolitan Sewerage District
From:	Steve McGowan, P.E., BCEE
	Project Manager, Malcolm Pirnie, Inc.
	Eric Wang, P.E.
	Project Engineer, Malcolm Pirnie, Inc.
Subject:	50-Year Master Plan
	TM-8: Planning Alternative Evaluation Criteria (Final)
Project No.:	MMSD No. 8425001
	MPI No. 6100-001

1. Purpose

This technical memorandum has been developed as part of the Madison MSD 50-Year Master Plan. The objectives of this memorandum are:

- To identify the applicable evaluation criteria to be used for master planning alternative evaluation.
- To determine appropriate level of importance for all evaluation criteria to be used in the planning alternative evaluation process.

2. Background

A list of alternative projects was developed in Technical Memorandum No. 7 – Development of Alternatives (TM-7) to meet the District's needs for wastewater conveyance, wastewater treatment, and biosolids management during the 50 year planning period. As part of TM-7, a rational screening process was conducted to generate a list of four master planning alternatives. The evaluation criteria developed in this technical memorandum will be used to evaluate and rank these identified alternatives on a common basis, and to determine the most cost-effective alternative(s) to be implemented to achieve the District's planning goals during the planning period.

Several meetings and workshops have been conducted with the MMSD and Technical Advisory Committee (TAC) to identify applicable evaluation criteria and determine their levels of importance. A survey has also been conducted to solicit opinions on evaluation criteria from TAC members and MMSD staff. The documents used in the survey and a MMSD prepared memo summarizing the survey results are attached to this technical memorandum in Appendix A.

The identified planning criteria are categorized into the following 4 groups:

- Economic criteria

  The impacts the planning alternatives have on the economic conditions of the District's stakeholders and on the District's own financial performance.

- Technical criteria

  The impacts the planning alternatives have on the technical aspects of the District operation, such as the ease of maintenance, system reliability, system flexibility, etc.

- Social criteria

  The impacts the planning alternatives have on the social systems within which the District operates, including public acceptance, staffing requirements, etc.

- Environmental criteria

  The impacts the planning alternatives have on natural systems, including ecosystems, land, air and water.

These evaluation criteria incorporate the major elements of typical sustainability evaluations of water and wastewater utilities. Adoption of these criteria in the evaluation process will allow evaluating and ranking planning alternatives from a multiple dimension perspectives.

3. Evaluation Criteria Description and Level of Importance

In this section, all the identified evaluation criteria are described and discussed. Levels of importance (Low, Medium and High) are then assigned to each of 10 criteria based on the combined efforts of the TAC, MMSD and the consultant.

3.1 Economic Criteria

3.1.1 Life Cycle Cost

A District mandate is to provide cost-effective wastewater conveyance, treatment and biosolids management services. Life cycle cost is used as a basis for making economic comparisons between alternatives. The life cycle costs is the total discounted dollar cost of owning, operating, maintaining, and disposing of the planning alternatives over the 50 year planning period. The life cycle cost includes the components listed below:

- Initial Capital Costs

  Initial capital costs include the purchase of land, buildings, equipment, and construction activities to bring all the component projects associated with a planning alternative to a fully operable status. Initial costs do not include labor costs except for the labor used for construction.

- 50-Year Replacement Cost

  All of the costs associated with the replacement of the structures, equipment, and other major components of the facilities included in a planning alternative to maintain the proper operation efficiency and physical conditions of the facilities during the 50 year planning period.

- Annual Operation/Maintenance Costs

  The annual operation/maintenance costs are composed of all the expenses including labor, materials, and other expenses for maintaining day-to-day facility functions and preserving the operating efficiency and physical condition of the facilities included in a planning alternative.

3.2 Technical Criteria

3.2.1 Regulatory Constraints

Alternatives must meet all regulatory requirements. However, the regulatory requirements associated with any given planning alternative may be easier or more difficult to meet, depending on a number of factors. For example, the regulatory requirements associated with an effluent discharge to an Exceptional Resource Water (ERW) or to a lake would be more stringent than those associated with discharge to a warm water stream.

3.2.2 Proven Effectiveness

The selected alternative(s) must be able to provide reliable service during the planning period. This criterion is used to evaluate planning alternatives for their reliability in providing required service. For example, fifteen years ago, biological phosphorous removal was not as proven a technology for removing phosphorous as chemical addition. As such, it would not have been considered to be as well "proven" as chemical addition. The proven ability of an alternative to meet the regulatory goals will need to be considered.

3.2.3 Flexibility, Expandability, and Compatibility

The selected alternative(s) must have the ability to be phased into connection with the existing system. This allows for ease of construction and financial burden to the District. The selected alternative(s) must be compatible with the existing collection system and treatment facilities, and maximize continued use of the existing facilities. The selected alternative(s) must also be compatible with other planning goals of Dane County and the City of Madison. This criterion is used to rank alternatives for their potentials to meet the following requirements:

- Can the alternative be readily modified to meet potential future needs such as re-routing wastewater, meeting more stringent future permit limits and regulations, etc?
- Can it be readily expanded to meet future flows and loadings?
- Is the alternative compatible with the existing collection system and treatment facilities?
- Does it maximize continued use of existing facilities?
- Can it be phased into connection with the existing system?
- Is it compatible with other planning goals of Dane County and the City of Madison?

3.2.4 Ease of Operation

Some alternatives may be more difficult or challenging to operate. For example, operation of a facility utilizing membrane filtration facility may be more difficult than operating the District's current facility. The selected alternative(s) must consider the level of complexity involved in operating the facilities included in the planning alternatives. This criterion will be used to rank all planning alternatives for efforts involved in the facility operation.

3.3 Social Criteria

3.3.1 Public Acceptance

Public acceptance has significant impacts on the implementation of planning alternatives. The selected planning alternative(s) must have the support of the public or a plan must be developed to gain this support. This criterion ranks all the planning alternatives for the likelihood of being accepted or resisted by the public.

3.3.2 Staffing Implications

Alternatives may have different staffing implications, both in terms of staffing level and required skills. For example, operation of multiple plants may be more labor intensive than operation of a centralized system. In addition, operating an advanced treatment (tertiary) system may require a more skilled workforce than operating a secondary treatment system. This criterion will be used to rank all planning alternatives for these staffing requirements.

3.4 Environmental Criteria

3.4.1 Maintains Watershed Balance

Stream flow augmentation and water balancing within the watershed are issues to address in the Master Plan. The volumes and locations at which the District discharges its effluent based on recommendations by the Master Plan will have significant impacts on sustaining water level in streams and aquifers, and maintaining watershed balancing throughout the watersheds. This criterion will be used to rank all the planning alternatives for their potential in augmenting low flow streams and alleviating imbalanced inter-watershed water transfer.

3.4.2 Opportunities for Effluent Reuse

One of the potential outcomes of the Master Plan is to maximize the use of treated effluent as a resource. Effective effluent reuse could reduce the need for groundwater withdrawals from the Madison area aquifer and improve the sustainability in water resource utilization in the Madison and the Dane County areas. The available effluent reuse options include:

- Turf irrigation
- Groundwater recharge
- Industrial water use
- Other uses

Some alternatives may present greater opportunity to beneficially reuse effluent because of location of facilities, level or treatment, etc. This criterion will be used to rank all the planning alternatives for their potential in treated effluent utilizations.

3.4.3 Carbon Footprint

Carbon footprint is a measure of the impact that the planning alternatives have on the environment in terms of the amount of the greenhouse gases produced. It will be evaluated for the utilization of electricity, natural gas, gasoline, etc. Some alternatives may have larger carbon footprint then the others. This criterion will be used to rank all the planning alternatives for their magnitude of carbon footprints.

4. Planning Alternative Level of Importance

The levels of importance for all planning alternatives were determined based on independent rankings by the TAC, MMSD and the consultant. The ranking scores from three sources were then averaged to calculate the final scores for all planning criteria. Evaluation criteria receiving scores higher than 10 are classified as “High” level of importance; those with scores between 6 and 10 are classified as “Medium” level of importance; while those with scores lower than 6 are classified as “Low” level of importance. The ranking results of all evaluation criteria are shown in Table 1.

Table 1. Planning Alternative Evaluation Criteria

No.	Evaluation Criteria	TAC Ranking Score	MMSD Ranking Score	Consultant Ranking Score	Average	Level of Importance
1	Life Cycle Cost	15	33	30	26	High
2	Public Acceptance	10	14	15	13	High
3	Watershed Balance	12	10	10	11	High
4	Flexibility/Expandability/ Compatibility	12	9	7	9	Medium
5	Effluent Reuse	13	7	8	9	Medium
6	Regulatory Constraints	8	9	10	9	Medium
7	Proven Effectiveness	10	7	8	8	Medium
8	Carbon Footprint	9	3	3	5	Low
9	Ease of Operation	6	5	4	5	Low
10	Staffing Implications	5	4	5	5	Low

Appendix A
Planning Alternative Ranking Criteria Survey

Madison Metropolitan Sewerage District 50-Year Master Plan

TAC Planning Alternative Ranking Criteria Survey

INTRODUCTION

As discussed at the March 17, 2009 Master Plan meeting, there are a number of criteria that have been discussed to assist in ranking alternatives for the Master Plan. Many of these alternatives have been discussed with the TAC either at a Planning Variables Workshop or a Scenario Planning Workshop. The District would now like feedback from the TAC on the relative importance of the criteria.

A Ranking Criteria Survey has been developed and is attached to this document. Ten alternative ranking criteria have been categorized into 4 groups: economic criteria, technical criteria, social criteria and environmental criteria. A detailed description for all these ranking criteria is provided below. The survey respondents should assign a weighting score ranging from 1 to 50 to each of the 10 ranking criteria in the spreadsheet provided according to their relative importance. The more important a ranking criterion is, the higher score is should be assigned to. However, to "force "a differentiation among the criteria, we ask that the total sum of the weighting scores for all 10 ranking criteria be equal to 100. For example, if someone thinks that all ten criteria are equally important, then they would each receive a score of 10. Or, if someone thinks that Life Cycle Costs is by far the most important criteria, it could receive a maximum score of 50 and the remaining 50 points could be spread among the remaining criteria as the evaluator sees fit. Evaluators may also add comments, in the appropriate section, if they wish.

Please send your comments to Dave Taylor at davet@madsewer.org. If you have any questions, you may call Dave at 608-222-1201 x-276. We would appreciate a response by April 3, 2009.

RANKING CRITERIA DESCRIPTION

Economic Criteria

Life Cycle Cost

A District mandate is to provide cost-effective wastewater conveyance, treatment and biosolids management services. Life cycle cost is used as a basis for making economic comparisons between alternatives. The life cycle costs is the total discounted dollar cost of owning, operating, maintaining, and disposing of the planning alternatives over the 50 year planning period. The life cycle cost includes the components listed below:

- Initial Capital Costs

  Initial capital costs include the purchase of land, buildings, equipment, and construction activities to bring all the component projects associated with a planning alternative to a fully operable status. Initial costs do not include labor costs except for the labor used for construction.

- 50-Year Replacement Cost

  All of the costs associated with the replacement of the structures, equipment, and other major components of the facilities included in a planning alternative to maintain the proper operation efficiency and physical conditions of the facilities during the 50 year planning period.

- Annual Operation/Maintenance Costs

  The annual operation/maintenance costs are composed of all the expenses including labor, materials, and other expenses for maintaining day-to-day facility functions and preserving the operating efficiency and physical condition of the facilities included in a planning alternative.

Technical Criteria

Regulatory Constraints

Alternatives must meet all regulatory requirements. However, the regulatory requirements associated with any given option may be easier or more difficult to meet, depending on a number of factors. For example, the regulatory requirements associated with an effluent discharge to an Exceptional Resource Water (ERW) or to a lake would be more stringent than those associated with discharge to a warm water stream.

Proven Effectiveness

The selected alternative(s) must be able to provide reliable service during the planning period. This criterion is used to evaluate planning alternatives for their reliability in providing required service. For example, fifteen years ago, biological phosphorous removal was not as proven a technology for removing phosphorous as chemical addition. As such, it would not have been considered to be as well "proven" as chemical addition. The proven ability of an alternative to meet the regulatory goals will need to be considered.

Flexibility/Expandability/Compatibility

This criterion is used to rank alternatives for their potentials to meet the following requirements:

- Can the alternative be readily modified to meet potential future needs such as re-routing wastewater, meeting more stringent future permit limits and regulations, etc?
- Can it be readily expanded to meet future flows and loadings?

- Is the alternative compatible with the existing collection system and treatment facilities?
- Does it maximize continued use of existing facilities?
- Can it be phased into connection with the existing system?
- Is it compatible with other planning goals of Dane County and the City of Madison?

Ease of Operation

Some alternatives may be more difficult or challenging to operate. For example, operation of a facility utilizing membrane filtration facility may be more difficult than operating the District's current facility.

Social Criteria

Public Acceptance

Public acceptance will significantly impact the ability to implement an alternative. This criterion assesses the likelihood that an alternative will be accepted or resisted by the public.

Staffing Implications

Alternatives may have different staffing implications, both in terms of staffing level and required skills. For example, operation of multiple plants may be more labor intensive than operation of a centralized system. In addition, operating an advanced treatment (tertiary) system may require a more skilled workforce than operating a secondary treatment system.

Environmental Criteria

Carbon footprint

Carbon footprint is a measure of the impact that the alternative will have on the environment in terms of the amount of the greenhouse gases produced. This carbon footprint of each alternative will be considered.

Opportunities for Effluent Reuse

Effective effluent reuse could reduce the need for groundwater withdrawals from the Madison area aquifer and improve the sustainability in water resource utilization in the Madison and the Dane County areas. Some alternatives may present greater opportunity to beneficially reuse effluent because of location of facilities, level or treatment, etc. The available effluent reuse options include:

- Turf irrigation
- Groundwater recharge
- Industrial water use
- Other uses

Maintains Watershed Balance
The volumes and locations at which the District discharges its effluent will have significant impacts on sustaining water level in streams and aquifers, and maintaining watershed balancing throughout the watersheds.

Madison-MSD 50 Year Master Plan
Planning Criteria Survey

Survey Respondent:
Date:

1	2	3	4	5
Ranking Criteria Category	**Ranking Criteria**	**Total Category Weighting Score**	**Ranking Criteria Weighting Score (1-50)**	**Comments**
Economic Criteria	Life Cycle Cost	0		
Technical Criteria	Regulatory Constraints	0		
	Proven Effectiveness			
	Flexibility/Expandability/Compatibility			
	Ease of Operation			
Social Criteria	Public Acceptance	0		
	Staffing Implications			
Environmental Criteria	Carbon Footprint	0		
	Effluent Reuse			
	Watershed Balance			

Total Score: **0**

Note:
1. Survey respondent should only input in column Nos. 4 and 5 (the areas in yellow color). All other columns were locked for protecting the spreadsheet format and formulas.
2. The respondent should assign a weighting score ranging from 1 to 50 for each criterion in column 4 based on their relative importance. The more important a criterioan is, the higher score it should receive.
3. The sum of the weighing scores in column 4 should be equal to 100.

A Brief Summary of Ranking Criteria Weighting Results
(04/09/09-prepared by Dave Taylor)

Ten criteria were identified for possible use in assisting with the ranking of Master Planning alternatives. TAC members and District Directors were each given 100 points and were asked to assign a score to each criterion according to their opinion of its relative importance. A maximum of 50 points could be assigned to any one criterion, with criterion deemed more important being assigned a higher score. Respondents were also given the opportunity to provide comments. Comments were recorded on the individual scoring worksheets which are being sent to you via email.

Responses were received from all but one TAC member and all District Directors. Raw data is given in the spreadsheet that accompanies this memo. Figure 1 compares the TAC and Director scores (average, min and max) for each criterion. The following table lists the criterion from highest to lowest average score for both the TAC and Directors. Use of median scores would not have changed the order for the TAC and would have resulted in minor differences for the Directors.

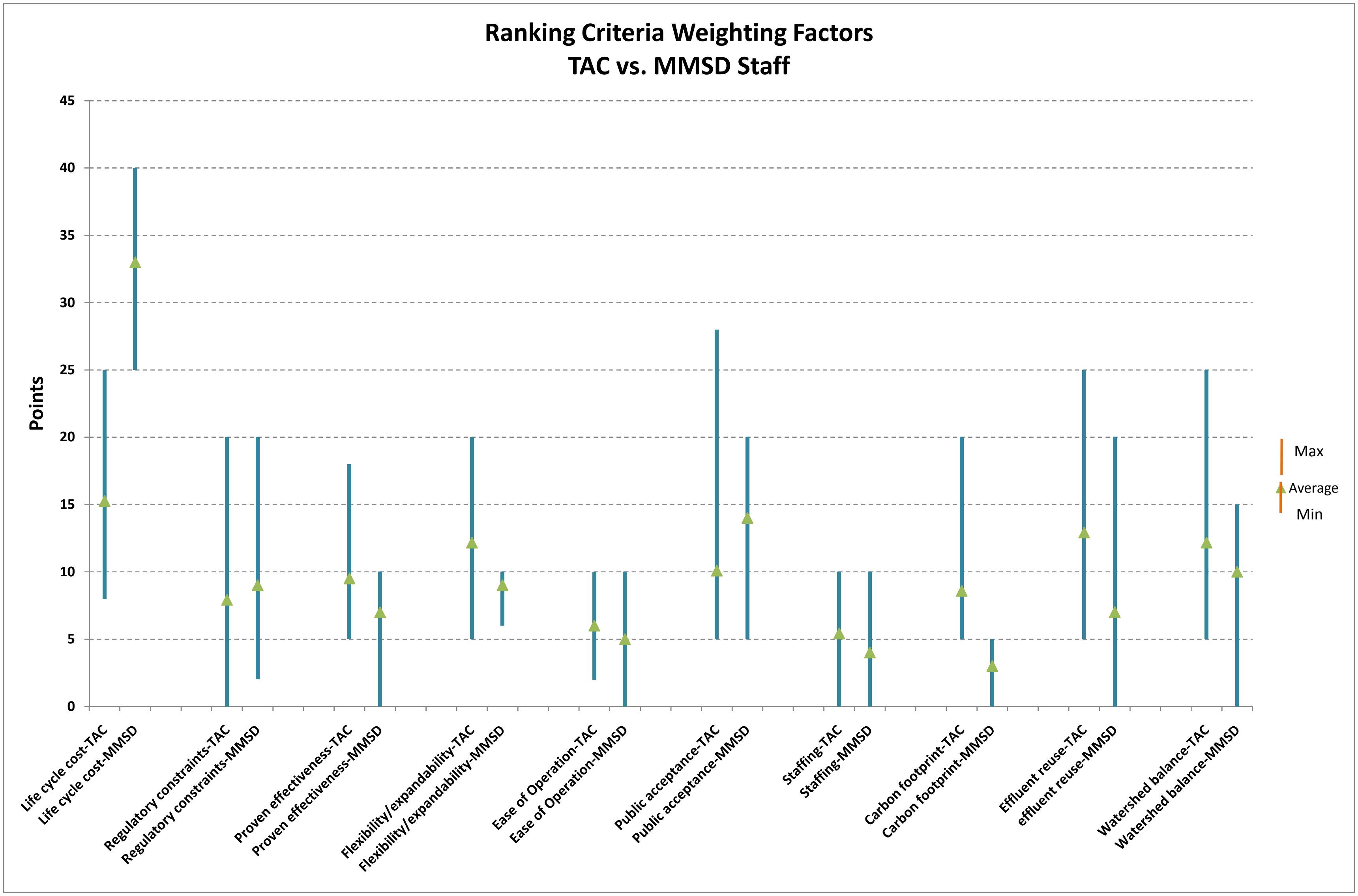
TAC	**Average score**	**Directors**	**Average score**
Life cycle cost	15	Life cycle cost	33
Effluent reuse	13	Public acceptance	14
Watershed balance	12	Watershed balance	10
Flexibility	12	Regulatory constraints	9
Public acceptance	10	Flexibility	9
Proven effectiveness	10	Effluent reuse	7
Carbon footprint	9	Proven effectiveness	7
Regulatory constraints	8	Ease of operation	5
Ease of operation	6	Staffing	4
Staffing	5	Carbon footprint	3

Drawing conclusions from the data is challenging because it is difficult to know the thought process that respondents used when assigning scores. For example, some respondents may have thought that while carbon footprint is important, it basically comes down to energy consumption, which was accounted for in the life cycle cost. Therefore, they may have assigned more points to life cycle costs and fewer points to carbon footprint. That said, some initial observations are given below:

- In general, the TAC scores were grouped tighter than the Director’s scores.

- Both groups assigned the highest score to life cycle cost. There was a small gap between life cycle cost and the next highest ranked criterion for the TAC and a substantial gap for the Directors.
- Public acceptance and regulatory constraints were ranked relatively high by Directors-the TAC ranked these categories lower.
- Watershed balance received a relatively high score from both groups, but the TAC assigned a higher score to effluent reuse than the Directors.
- Ease of operation and staffing received relatively low scores from both groups.

We will need to get together with Malcolm Pirnie to discuss the scoring information and determine how to best use this information moving forward.



Ranking Criteria Weighting Factors
TAC vs. MMSD Staff
Points
45
40
35
30
25
20
15
10
5
0
Life cycle cost-TAC
Life cycle cost-MMSD
Regulatory constraints-TAC
Regulatory constraints-MMSD
Proven effectiveness-TAC
Proven effectiveness-MMSD
Flexibility/expandability-TAC
Flexibility/expandability-MMSD
Ease of Operation-TAC
Ease of Operation-MMSD
Public acceptance-TAC
Public acceptance-MMSD
Staffing-TAC
Staffing-MMSD
Carbon footprint-TAC
Carbon footprint-MMSD
Effluent reuse-TAC
effluent reuse-MMSD
Watershed balance-TAC
Watershed balance-MMSD
Max
Average
Min

Technical Memorandum No.9

Date:	November 23, 2009
To:	Madison Metropolitan Sewerage District
From:	Steve McGowan, P.E., BCEE
	Project Manager, Malcolm Pirnie, Inc.
	Eric Wang, P.E.
	Project Engineer, Malcolm Pirnie, Inc.
Subject:	50-Year Master Plan
	TM-9: Planning Alternative Ranking and Evaluation (Final)
Project No.:	MMSD No. 8425001
	MPI No. 6100-001

1.01 Purpose

This technical memorandum is the last of the nine technical memoranda developed as part of the Madison Metropolitan Sewerage District (MMSD) 50-Year Master Plan. The objectives of this memorandum are as follows:

- Refine the master planning alternatives recommended in TM-7: Development of Planning Alternatives, and develop specific implementation requirements for each selected planning alternative.
- Determine the life cycle costs for the short-listed master planning alternatives recommended by TM-7.
- Evaluate and rank the short-listed master planning alternatives using the criteria and methods previously developed in TM-8: Planning Alternative Ranking Criteria.
- Identify the most favorable near-term (2010 to 2030) planning alternative based on the evaluation conducted in this memorandum.
- Evaluate the identified long-term (2030 to 2060) planning alternatives and provide general guidance regarding potential implementation. Due to the longer planning horizon and less certain nature of these long-term alternatives, the evaluation will be less specific than for short-term alternatives.

1.02 Background

The following eight Tech Memos have been prepared for this project and have been used in developing this tech memo:

- **Technical Memo 1 – Review of Existing Treatment Facilities –** An analysis of the capacities of the existing facilities at the Nine Springs Wastewater Treatment Plant (NSWTP).
- **Technical Memo 2 – Flow and Loading Projections –** Flow and loading projections based on information prepared by the Capital Area Regional Planning Commission (CARPC) and contained in the 2008 MMSD Collection System Evaluation, and an analysis of the projected capacity needs based on treating all wastewater generated in the District service area at the NSWTP.
- **Technical Memo 3 – Conveyance Facilities Analysis (CFA) –** An analysis of existing conveyance facilities to convey wastewater from MMSD users to the NSWTP.
- **Technical Memo 4 – Planning Variables –** A summary of identified major planning variables that will govern or impact MMSD's available options for continuing to provide high quality services over the planning period.
- **Technical Memo 5 – Regulatory Review and Analyses** – Review of the regulations that may impact the existing operation and planning alternatives generated in Master Plan.
- **Technical Memo 6 – Scenario Planning Workshops** – Documents the scenario planning process used in identifying factors and uncertainties that could potentially impact MMSD during the planning period.
- **Technical Memo 7 – Development of Planning Alternatives** – Defines Master Plan projects and groups them into potential planning alternatives that provide different approaches to meet the needs of MMSD in the next 50 years.
- **Technical Memo 8 – Planning Alternative Evaluation Criteria** – Presents the development of the evaluation criteria and methods for evaluating planning alternatives for the Master Plan.

A long list of 10 master planning projects was developed and evaluated in TM-7. Preliminary screening of these planning projects has been conducted to select master planning alternatives for further evaluation. Except for Alternative MP-1 (base planning alternative), the following two key principles were incorporated in making the selection of near-term planning alternatives:

- The proposed alternative project must have an implementation date that allows sufficient time for the District to site and construct the alternative project prior to the time necessary to alleviate an existing MMSD capacity need.

- Alternatives must provide sufficient capacity so that any future expansion of the current advanced secondary treatment facilities at the NSWTP beyond the existing 57 mgd capacity will not be required before 2060.

Based on these criteria, two near-term master planning alternatives from TM-7 have been selected for further evaluation in this technical memo. Implementation of either of these alternatives between 2010 and 2030 will address the wastewater treatment and conveyance system capacity needs in the MMSD service area:

- **Alternative MP-1 – Westside Conveyance System Expansion**: This alternative expands the existing conveyance system and continues the current model of centralized treatment at the NSWTP. This alternative includes four variations to pump treated effluent to different receiving water bodies. Detailed descriptions of these alternative variations are provided in Section 2.02.

- **Alternative MP-2 – Sugar River WWTP**: This alternative will construct a new high quality effluent treatment plant in the Sugar River watershed to treat wastewater generated in the PS 17 and PS 12 service areas, and discharge effluent to the Sugar River. This alternative includes two variations to discharge treated effluent to different locations of the Sugar River. Detailed descriptions of these alternative variations are provided in Section 2.03.

Several alternative projects evaluated in TM-7 could not be implemented soon enough to provide near-term capacity relief for the conveyance system. However they remain potentially viable options beyond the year 2030 for providing relief in the conveyance system, mitigating inter-basin transfers of water, or providing high quality effluent for reuse options. These alternatives are categorized as long-term planning alternatives. The following two long-term planning alternatives were selected for further evaluation in this technical memo:

- **Alternative MP-3 – Centralized High Quality Effluent Treatment & Distribution**: This alternative includes construction of facilities at the NSWTP that would produce a high quality effluent for reuse in various applications, including stream flow augmentation in Starkweather Creek, groundwater infiltration, industrial reuse, turf irrigation, etc. Detailed descriptions of these alternative variations are provided in Section 3.02.

- **Alternative MP-4 – Decentralized High Quality Effluent Treatment Facilities**:

  This alternative includes construction of facilities northeast of the Dane County Regional Airport. The new treatment plant will receive wastewater flows tributary to PS13 or both PS13 and PS14. Effluent from this facility could be used for stream flow augmentation to Starkweather Creek, wetland restoration at Cherokee Marsh, groundwater infiltration, industrial reuse water or turf irrigation. Detailed descriptions of these alternative variations are provided in Section 3.03.

1.03 Evaluation Method

Ten criteria were identified in TM-8 for use in evaluating the master planning alternatives and are presented in Table 1.03.1. The level of importance associated with each of these criteria was determined jointly by MMSD staff, the Technical Advisory Committee (TAC) and the consultant team. Each group independently assigned a numeric score designating the level of importance for each criterion, using the procedure identified in TM-8. The scores are shown in Table 1.03.2, with the average score used when comparing alternatives in this technical memorandum. Alternatives were also assigned a relative ranking of 1 to 10 for each criterion by the consultant team, consistent with the approach identified in Table 1.03.2.

All planning alternatives were evaluated based on the ranking score for each criterion, multiplied by the level of importance for that criterion. For example, if the life cycle cost criterion (with a level of importance of 26) received a ranking score of 10, the score for that criterion is 26 x 10 = 260. Total scores for each alternative were then calculated by adding the weighted score for each criterion for that alternative. Planning alternatives with higher total scores represent more favorable alternatives than those with lower scores.

Table 1.03.1 Planning Alternative Evaluation Criteria Definitions

Evaluation Criteria	Definition
Life Cycle Cost	Life cycle cost is the total discounted dollar cost of owning, operating, and maintaining the planning alternatives. A ranking score of “10” for this criterion represents the lowest life cycle cost.
Public Acceptance	Public acceptance is the support level of the public to a planning alternative. A ranking score of “10” for this criterion represents the highest level of public acceptance.
Watershed Balance	Watershed balance is the potential of a planning alternative to mitigate imbalanced inter-basin water transfer. A ranking score of “10” for this criterion represents the highest potential to mitigate imbalanced inter-basin water transfer.
Flexibility Expandability Compatibility	This criterion is used to gauge alternatives for their potential to be readily modified or expanded to meet future needs, and their compatibility with the existing system. A ranking score of “10” for this criterion represents the highest level of flexibility, expandability and compatibility.
Effluent Reuse	This criterion is used to evaluate all planning alternatives for their potential to provide treated effluent utilization. A ranking score of “10” for this criterion represents the highest potential for effluent reuse.

Evaluation Criteria	Definition
Regulatory Constraints	This criterion is used to evaluate the potential regulatory constraints that may affect the implementation of the planning alternative. A ranking score of "10" for this criterion represents the lowest level of regulatory constraints.
Proven Effectiveness	This criterion is used to evaluate planning alternatives for their proven reliability in providing the required level of conveyance and treatment. A ranking score of "10" for this criterion represents the highest level of effectiveness.
Carbon Footprint	This criterion is used to evaluate planning alternatives for their impacts on the environment in terms of the amount of the greenhouse gases produced. A ranking score of "10" for this criterion represents the lowest carbon footprint.
Ease of Operation	This criterion is used to rank the efforts involved in the facility operation. A ranking score of "10" for this criterion represents the relatively easiest operation.
Staffing Implications	This criterion is used to rank all planning alternatives for the staffing requirements in terms of staffing level and required skills. A ranking score of "10" for this criterion represents staffing level and skill requirements most similar to or less than the current requirements.

Table 1.03.2 Level of Importance of Ranking Criteria

No.	Evaluation Criteria	TAC Ranking Score	MMSD Ranking Score	Consultant Ranking Score	Average Score	Level of Importance
1	Life Cycle Cost	15	33	30	26	High
2	Public Acceptance	10	14	15	13	High
3	Watershed Balance	12	10	10	11	High
4	Flexibility/Expandability /Compatibility	12	9	7	9	Medium
5	Effluent Reuse	13	7	8	9	Medium
6	Regulatory Constraints	8	9	10	9	Medium
7	Proven Effectiveness	10	7	8	8	Medium
8	Carbon Footprint	9	3	3	5	Low
9	Ease of Operation	6	5	4	5	Low
10	Staffing Implications	5	4	5	5	Low

2.01 Near-Term Master Planning Alternatives (2010-2030)

In this section, all four variations of the Alternative MP-1 will be compared to the two variations of the Alternative MP-2 to determine the planning alternative to be implemented between 2010 and 2030.

2.02. Alternative MP-1: Westside Conveyance System Expansion

This alternative is based on continued conveyance to and centralized treatment at the NSWTP, with expansion of the existing conveyance and treatment systems as needed to support the treatment of the projected wastewater flows and loads in the MMSD system. Based on the effluent discharge locations, the following four variations are included under this planning alternative:

- **Alternative MP-1A –** This alternative includes returning 3.6 mgd of treated effluent from the NSWTP to Badger Mill Creek through the existing outfall in Badger Prairie Park, and returning the rest of treated effluent to Badfish Creek. This alternative represents the current operation by MMSD. It services as the base alternative to be compared to other alternatives.

- **Alternative MP-1B –** This alternative includes returning a total of 7.9 mgd of treated effluent (3.6 mgd of regular effluent and 4.3 mgd of high quality effluent) from the NSWTP to Badger Mill Creek through the existing outfall in Badger Prairie Park. The rest of the treated effluent will be returned to Badfish Creek. The regular effluent and the high quality effluent will be blended and pumped to the Badger Mill Creek outfall location through the existing effluent force main. This alternative represents a centralized effluent reuse and watershed balance solution (i.e. it returns water to the watershed from where it was withdrawn). This alternative may have less regulatory constraints since:

    - The increased flow is high quality effluent and therefore the increased TP and TN loadings to the Badger Mill Creek will be less significant.
    - It discharges to the existing outfall in Badger Prairie Park

- **Alternative MP-1C –** This alternative includes returning 7.9 mgd of treated effluent to the Badger Mill Creek outfall location through the existing force main with 3.6 mgd being discharged to Badger Mill Creek directly and 4.3 mgd being discharged downstream of the confluence of Badger Mill Creek and Sugar River through a new pumping station and a new force main at the vicinity of the existing Badger Mill Creek outfall. The rest of the treated effluent by the NSWTP will be returned to Badfish Creek. The regular effluent and the high quality effluent will be blended and pumped to the Badger Mill Creek outfall location through the existing effluent force main. This alternative represents a centralized effluent reuse and watershed balance solution. It could achieve similar effluent reuse and watershed balance benefits as the decentralized alternative (MP-2A) discussed later.

- **Alternative MP-1D –** This alternative includes returning 7.9 mgd of treated effluent to the Badger Mill Creek outfall location through the existing force main with 3.6 mgd being discharged to Badger Mill Creek directly and 4.3 mgd being discharged to the Sugar River at the County Highway PD (CTH PD) through a new pumping station and a new force main in the vicinity of the existing Badger Mill Creek outfall. The rest of the treated effluent will be returned to the Badfish Creek. The regular effluent and the high quality effluent will be blended and pumped to the Badger Mill Creek outfall location through the existing effluent force main. This alternative represents a centralized effluent reuse and watershed balance solution. It could achieve similar effluent reuse and watershed balance benefits as the decentralized alternative (MP-2B) discussed later.

For alternatives MP-1B, 1C, and 1D, 7.9 mgd of treated effluent needs to be pumped to the existing Badger Mill Creek outfall. Preliminary analysis shows that the existing 20” force main has sufficient capacity for the increased flow but new pumps would be needed. The detailed calculations are attached in Appendix C. The layouts of these alternatives are shown in Figures 2.02.1 through 2.02.4. The major component projects included in these alternatives are presented in Table 2. 02.1.

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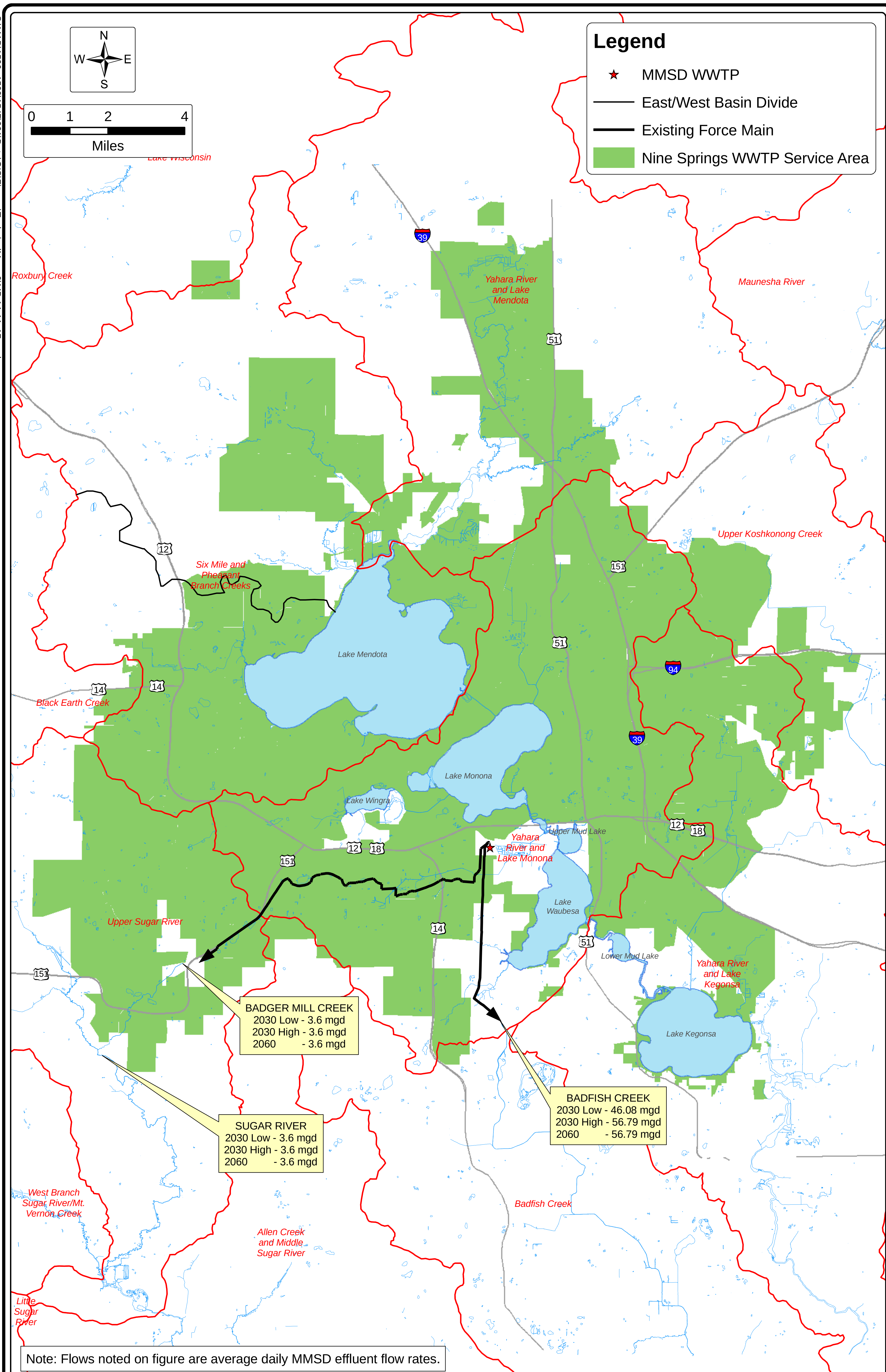

FIGURE 2.02-1
1547.001

STRAND ASSOCIATES, INC. ENGINEERS

MALCOLM PIRNIE

MMSD

ALTERNATIVE MP-1A

TECHNICAL MEMO 9
PLANNING ALTERNATIVE RANKING AND EVALUATION
MADISON METROPOLITAN SEWERAGE DISTRICT

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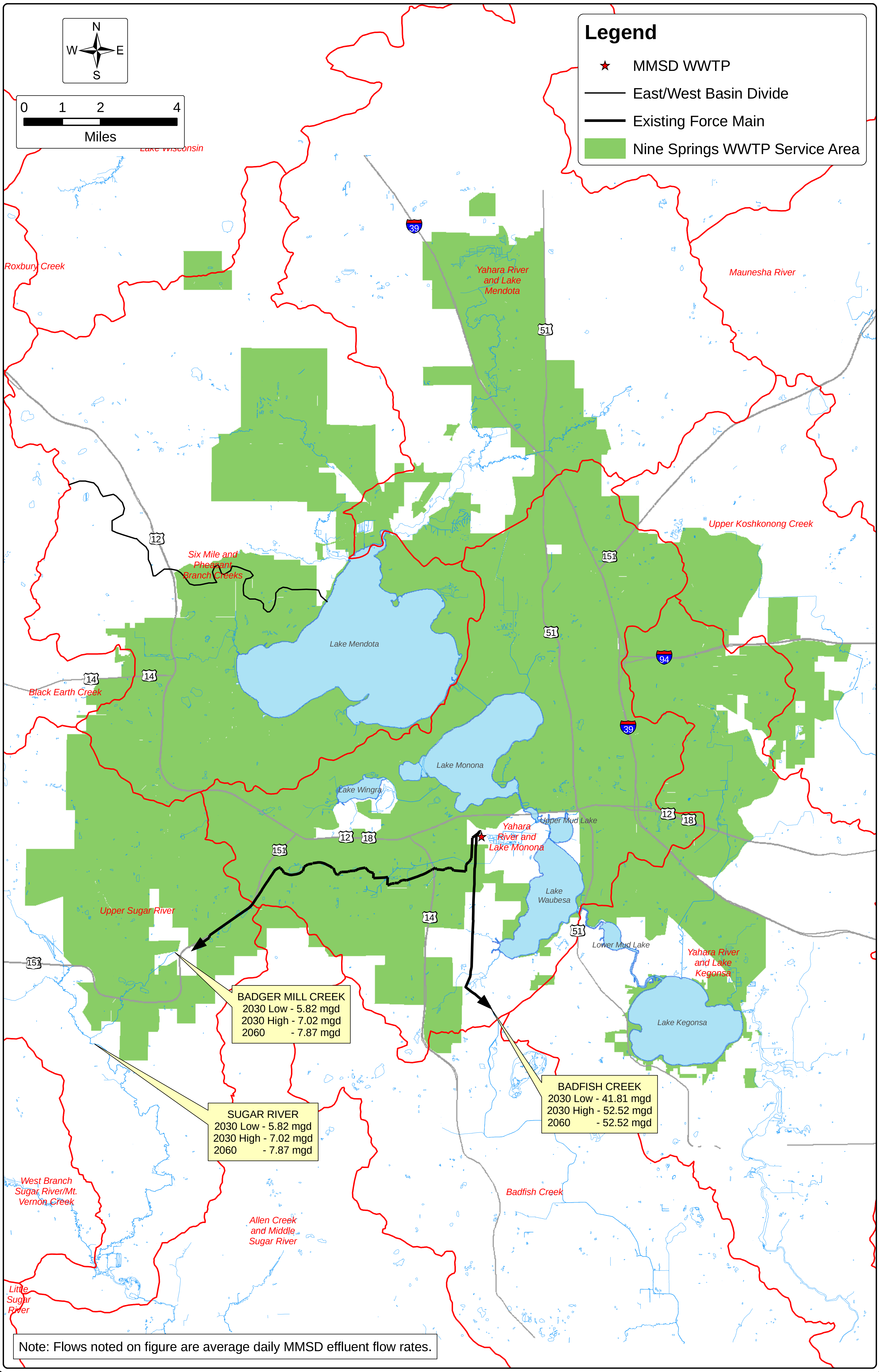

FIGURE 2.02-2
1547.001

STRAND ASSOCIATES, INC. ENGINEERS

MALCOLM PIRNIE

ALTERNATIVE MP-1B

TECHNICAL MEMO 9
PLANNING ALTERNATIVE RANKING AND EVALUATION
MADISON METROPOLITAN SEWERAGE DISTRICT

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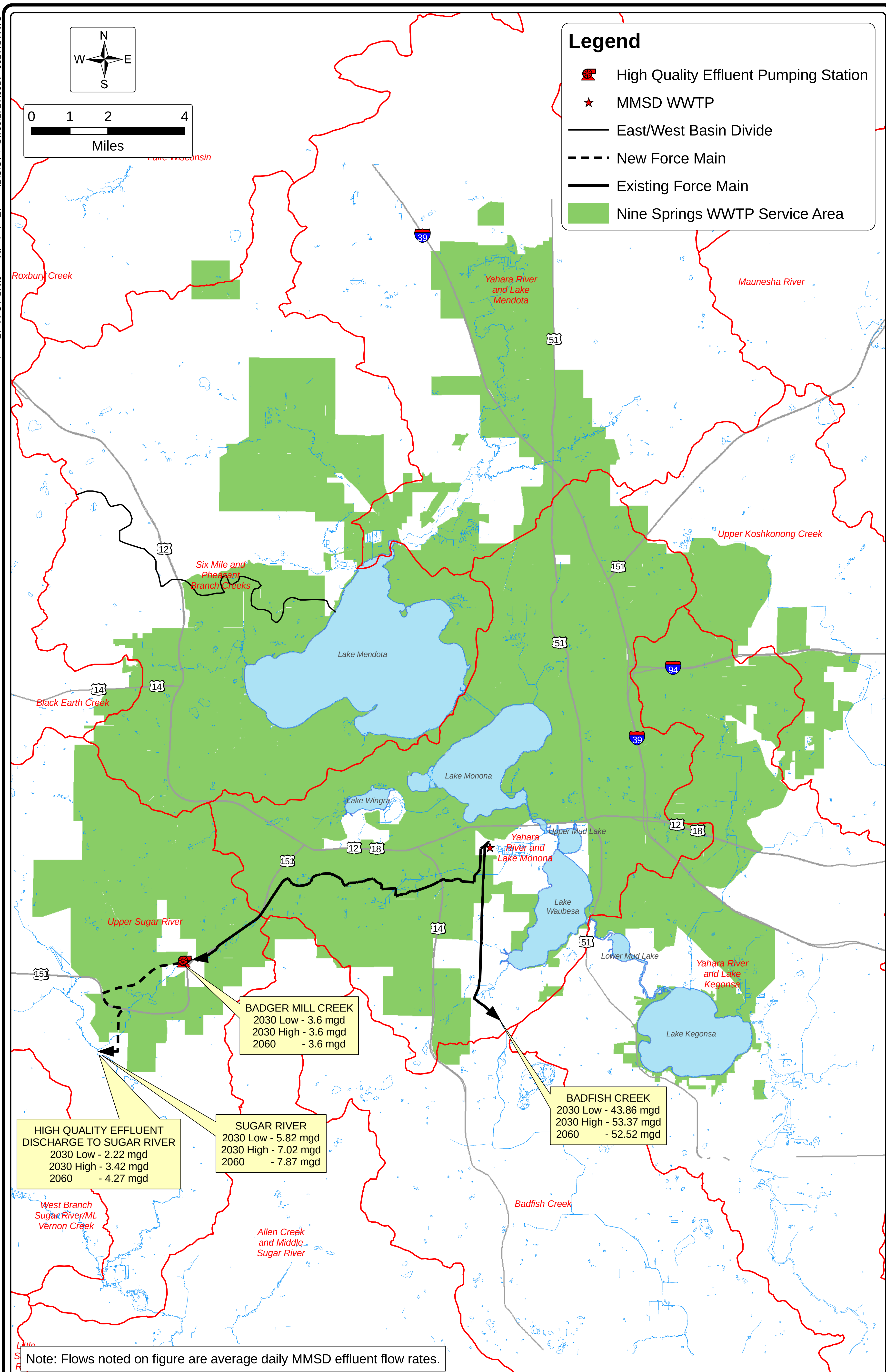

FIGURE 2.02-3
1547.001

STRAND ASSOCIATES, INC. ENGINEERS

MALCOLM PIRNIE

mmsd

ALTERNATIVE MP-1C

TECHNICAL MEMO 9
PLANNING ALTERNATIVE RANKING AND EVALUATION
MADISON METROPOLITAN SEWERAGE DISTRICT

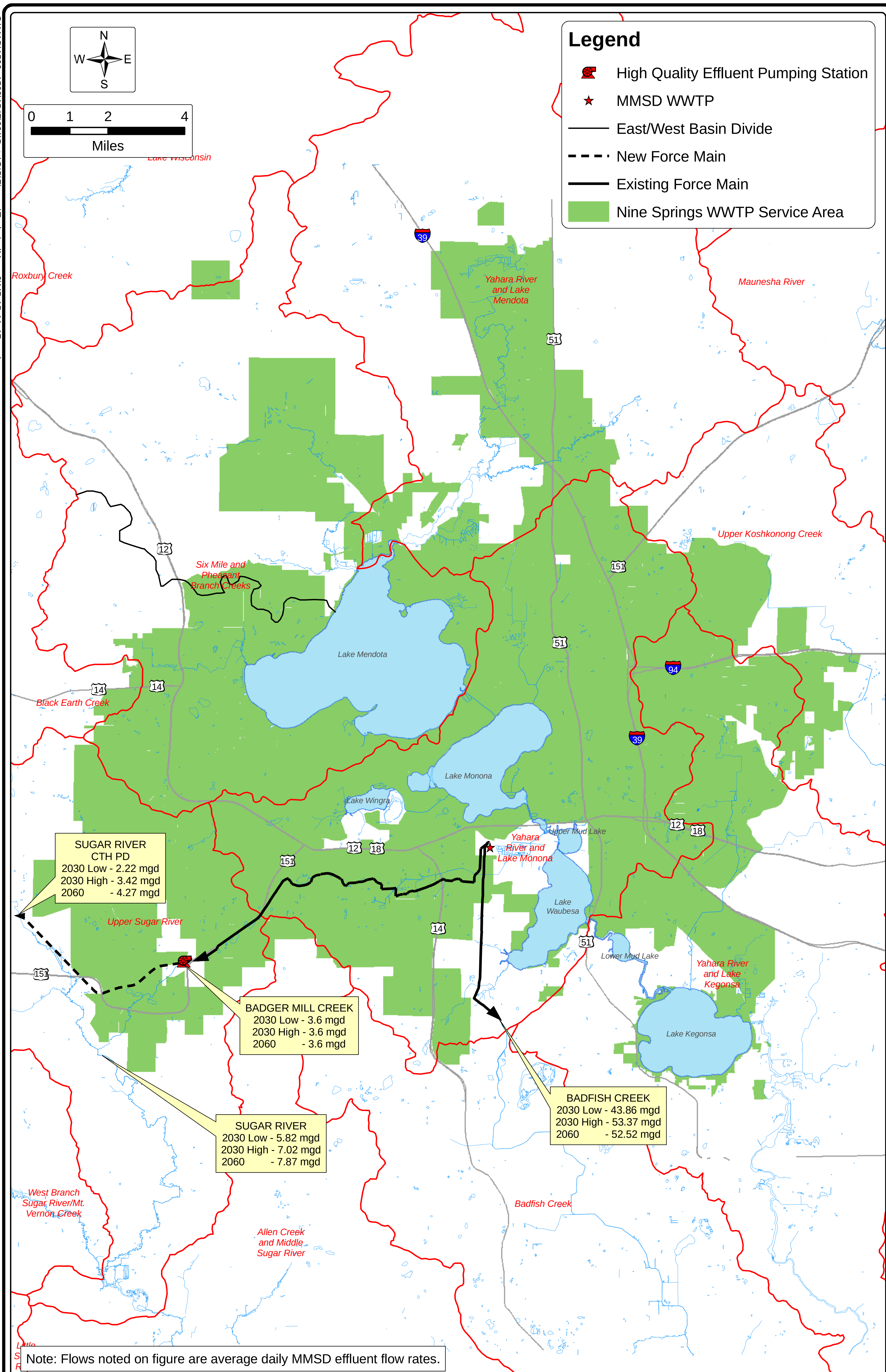

FIGURE 2.02-4
1547.001

STRAND ASSOCIATES, INC. ENGINEERS

MALCOLM PIRNIE

MMSD

ALTERNATIVE MP-1D

TECHNICAL MEMO 9
PLANNING ALTERNATIVE RANKING AND EVALUATION
MADISON METROPOLITAN SEWERAGE DISTRICT

Table 2.02.1 Component Projects of Master Planning Alternative MP-1

Facility Name	Component Project	MP-1A	MP-1B	MP-1C	MP-1D
PS11	Condition improvement and firm pumping capacity expansion. The cost for this improvement is already budgeted and included in the scheduled PS 11 condition improvement project.	Yes	Yes	Yes	Yes
PS11	Install a new 36" diameter force main parallel to the existing force main.	Yes	Yes	Yes	Yes
PS12	Condition improvement and firm pumping capacity expansion. The cost for this improvement is already budgeted and included in the scheduled PS 12 condition improvement project.	Yes	Yes	Yes	Yes
PS17	Firm pumping capacity expansion to average daily flow of 4.37 mgd and peak flow of 13.6 mgd.	Yes	Yes	Yes	Yes
PS17	Force main expansion	Yes	Yes	Yes	Yes
NSVI	Expand capacity of interceptor section from PS11 to PS12.	Yes	Yes	Yes	Yes
NSVI	Expand capacity of section upstream of PS12.	Yes	Yes	Yes	Yes
NSVI	Relining the entire length of the NSVI	Yes	Yes	Yes	Yes
Badger Mill Creek Effluent Pumps	Expand the current average effluent pumping capacity to 7.9 mgd.	No	Yes	Yes	Yes
Sugar River Effluent Pumping Station	Construction of a new pumping station at the vicinity of the Badger Mill Creek outfall with an average capacity of 4.3 mgd.	No	No	Yes	Yes
Sugar River Force Main	Construction of a new force main for the new effluent pumping station to downstream of confluence of Badger Mill Creek and Sugar River.	No	No	Yes	No
Sugar River Headwaters Force Main	Construction of a new force main for the new effluent pumping station to the Sugar River headwaters near CTH PD.	No	No	No	Yes
High Quality Effluent Treatment facility at the NSWTP	Construction of a new high quality effluent treatment facility at the NSWTP with capacities of 4.3 mgd (DAF) and 13.7 mgd (DMF). The facility would include processes for effluent polishing to meet the 5 mg/L limit for BOD_5 and TSS. The facility would also be designed to meet a potential 0.075 mg/L TP limit and a 3 mg/L Total N limit.	No	Yes	Yes	Yes

Alternative MP-1 Evaluation

Life Cycle Cost

Life cycle costs were calculated for these planning alternatives based on the costs of construction, operation and maintenance costs, replacement/rehabilitation costs, and salvage values. Operation costs only include the costs for pumping wastewater to the treatment plants and pumping effluent from the plants to the various discharge locations and high quality effluent operational cost. The results were used for subsequent planning alternative evaluation and ranking.

The results of the total life cycle cost analysis for all four alternatives are summarized in Table 2.02.2. Detailed planning alternative life cycle cost calculations are attached in Appendix A.

Table 2.02.2 Alternative MP-1A Life Cycle Cost Analysis

Item	MP-1A	MP-1B	MP-1C	MP-1D
Initial Capital Costs	$50,881,000	$68,581,000	$75,068,000	$75,068,000
Life Cycle O/M Costs	$18,881,000	$30,843,000	$32,901,000	$33,029,000
Life Cycle Costs for Facility Improvement & Replacement	$2,758,000	$7,313,000	$7,698,000	$7,698,000
Salvage Value	($3,298,000)	($3,702,000)	($4,115,000)	($4,115,000)
50-Year Total Present Value	$69,222,000	$103,034,000	$111,552,000	$111,680,000

Public Acceptance

Alternative MP-1A and MP-1B continue MMSD's current operations on MMSD's property at the NSWTP and expand the conveyance system, which is located primarily on public lands. Therefore, the public acceptance for this alternative will be high except during the construction of the Nine Springs Valley Interceptor (NSVI) capacity expansion.

Since no additional effluent pumping is provided for alternative MP-1A, the frequency of discharges to Lake Waubesa through Nine Springs Creek under high flow conditions may increase. This may raise new public concerns over the water quality in Lake Waubesa. Alternatives MP-1B, MP-1C and MP-1D include increased pumping of treated effluent to the Sugar River watershed, and therefore may reduce the frequency of emergency discharges of treated effluent to Lake Waubesa through Nine Springs Creek under high flows. This may help reduce the public's concern over potential negative impacts on the water quality of Lake Waubesa.

Alternatives MP-1C and MP-1D include discharges of high quality effluent to the Sugar River, which is classified as an Exceptional Resource Water (ERW). The Sugar River also has a downstream impoundment (Lake Belle View) at Belleville. A direct discharge to the Sugar River could generate public concern or resistance because of these factors. A discharge to the Sugar River at CTH PD may raise additional concerns since this section of the river is not currently impacted by wastewater effluents and is closer to the Verona urban area. However, the public has also expressed support for maintaining the water balance within the Sugar River basin.

The rankings of public acceptance for these planning alternatives are:

Alternative MP-1A – 8

Alternative MP-1B – 9

Alternative MP-1C – 6

Alternative MP-1D – 5

Watershed Balance

The 2004 Dane County groundwater model predicted a net base flow loss of 47 mgd at the Yahara River outlet at Lake Waubesa and a base flow loss of 1.47 mgd at the Sugar River watershed in 2030 if the current effluent diversion practices continue. Alternative MP-1A will have no new impact on the water balance between the Sugar River and the Yahara River watershed since it maintains the current practices. Alternative MP-1B, MP-1C and MP-1D increase pumping of treated effluent to the Sugar River watershed. The projected net base flow loss in the Sugar River watershed under Alternative MP-1A are shown in Table 2.02.3.

Table 2.02.3 Water Balance of Alternative MP-1A

Flow Projections	Sugar River Watershed Groundwater Withdrawal (mgd)	Total Flow to Sugar River (mgd)	Watershed Net Loss/Increases (mgd)	Watershed Net Loss/Increases (%)
2030 Low	5.03	3.6	-1.43	-28%
2030 High	6.86	3.6	-3.26	-48%
2060 Low	6.86	3.6	-3.26	-48%
2060 High	8.09	3.6	-4.49	-56%

Alternative MP-1A will result in a significant net loss of water from the Sugar River watershed under all planning flow projections, with losses ranging from 28 percent to 56 percent of the total groundwater withdrawal from the Sugar River Watershed. For alternatives MP-1B, MP-1C and MP-1D, the effluent volume returned to the Sugar River watershed can be controlled to exactly match the volume of water withdrawal, although in slightly different locations. Alternative MP-1C provides additional benefit of flow augmentation to the Sugar River downstream of the confluence of the Sugar River and

Badger Mill Creek. Alternative MP-1D will provide flow augmentation to the Sugar River upstream the confluence of the Sugar River and Badger Mill Creek. The watershed balance rankings for these alternatives are:

Alternative MP-1A – 5

Alternative MP-1B – 8

Alternative MP-1C – 9

Alternative MP-1D – 10

Flexibility/Expandability/Compatibility

Alternatives MP-1A and MP-1B have less flexibility than the other alternatives since effluent discharge locations are limited to Badfish Creek and Badger Mill Creek, and the capacities of the existing force mains are limited. Alternatives MP-1C and MP-1D include a new effluent pumping station and force main to convey treated effluent to the Sugar River. These alternatives will provide the District with enhanced flexibility since there would be watershed options for routing of wastewater and reduction of phosphorus entering the Rock River. All alternatives would allow for potential expansion. Sufficient space exists for expansion of facilities at the NSWTP site and new pumping stations and effluent transmission lines associated with alternatives MP-1C and MP-1D would be sited to allow for expansion if necessary. All alternatives are compatible with current operations at the NSWTP.

Overall the flexibility/expandability/compatibility rankings for these alternatives are:

Alternative MP-1A – 5

Alternative MP-1B – 6

Alternative MP-1C – 8

Alternative MP-1D – 8

Effluent Reuse

The District currently has a program evaluating the use of treated effluent for turf irrigation at the Nine Springs Golf Course, but the demand is seasonal and accounts for a very small fraction of the daily effluent generated at the NSWTP. The NSWTP location provides an excellent location for potential industrial reuse since it is already located in an area zoned for commercial and industrial uses. All alternatives will initially rely on stream flow augmentation as the primary effluent reuse option, absent a defined end user. Under alternative MP-1A, approximately 3.6 mgd of the NSWTP effluent will be pumped to

Badger Mill Creek with the rest of it being pumped to Badfish Creek. Alternative MP-1B, MP-1C and MP-1D have the ability of pumping a maximum of an additional 4.3 mgd to the Sugar River watershed if effluent reuse opportunities emerge in that area.

The rankings of effluent reuse for these alternatives for the southern portion of the MMSD service area are:

Alternative MP-1A – 6

Alternative MP-1B – 7

Alternative MP-1C – 8

Alternative MP-1D – 8

Regulatory Constraints

The Wisconsin Department of Natural Resources is in the process of developing numeric water quality criteria for phosphorus. A value of 0.075 mg/l has been proposed for wadeable streams and would be applicable to all alternatives. Water quality criteria may also be developed for nitrogen in the future, with a limit in the range of 3.0 mg/l being possible. This limit would also be applicable to all alternatives.

Thermal limits have recently been promulgated by WDNR which may make it more difficult to site alternatives involving discharges to streams classified as cold water fisheries or alternatives involving discharges to previously un-impacted stream segments. Alternatives MP-1C and MP-1D both involve discharges to cold water fisheries. Alternative MP-1D involves a discharge to a previously un-impacted stream segment. Both alternatives may require significantly more effort to obtain a WPDES permit. Alternative MP-1C may experience less stringent regulatory constraints since its discharge is to a point on the Sugar River that is already impacted by the current Badger Mill Creek discharge.

The Sugar River is designated as an Exceptional Resource Water (ERW) by WDNR. Discharges to the Sugar River would be subject to the antidegradation requirements in NR 207 of the Wisconsin Administrative Code. Alternatives MP1-C and MP-1D would be impacted and would likely result in a higher level of treatment being required. For example, it is possible that the chloride discharge limits would have to meet the background stream concentration as opposed to meeting a discharge limit calculated using the provisions of NR 102 and NR 106 of the Wisconsin Administrative Code. Alternative MP-1B could also be similarly impacted if it is determined that this represents an increased discharge that results in a significant lowering of water quality in the Sugar River.

Discharge to the Sugar River will likely result in effluent BOD_5 and TSS limitations of 4 mg/L based on a 7Q10 of 14.3 cfs and a river temperature of 18 degrees centigrade, and about 7 mg/L based on a 7Q10 of 14.3 cfs and a river temperature of 5 degrees centigrade. Ammonia limits for the new discharge would likely be easily met since the facility would be designed for nitrification/denitrification.

Based on known or reasonably anticipated regulatory impacts, the overall rankings for these alternatives are:

Alternative MP-1A – 9

Alternative MP-1B – 7

Alternative MP-1C – 4

Alternative MP-1D – 3

Proven Effectiveness

The alternative MP-1A continues the current operations, which has an excellent track record for providing consistent wastewater conveyance and treatment performance. The alternative MP-1B includes high quality effluent treatment facility and increased flow discharge to the Badger Mill Creek and may need to meet more stringent discharge limits. Alternatives MP-1C and MP-1D include direct discharges to an ERW and may be subject to chloride discharge limit. All alternatives may need to meet more stringent phosphorus (0.075 mg/L of TP) and nitrogen (3 mg/L of TN) limits. Currently there are very limited full scale installations and available operation data on technologies that are able to consistently meet these stringent discharge limits. The overall rankings of proven effectiveness for these alternatives are:

Alternative MP-1A – 8

Alternative MP-1B – 6

Alternative MP-1C – 4

Alternative MP-1D – 4

Carbon Footprint

Due to the increased amount of pumping associated with these alternatives, the carbon footprints could be higher than the carbon footprints for the Sugar River WWTP. Preliminary analysis results of the additional electrical consumption and total greenhouse gas (GHG) emissions are presented in Table 2.02.4. Detailed alternative carbon footprint calculations are attached in Appendix B.

Table 2.02.4 Alternative MP-1A Carbon Footprint

Alternative	Electricity Consumption (Mwhr/yr)	Total GHG Emissions (tons CO_2 equivalents/yr)
MP-1A	7,264	3,895
MP-1B	10,261	9,464
MP-1C	11,053	10,195
MP-1D	11,136	10,272

The carbon footprint for each of these alternatives is ranked as follows:

Alternative MP-1A – 8

Alternative MP-1B – 6

Alternative MP-1C – 5

Alternative MP-1D – 5

Ease of Operation

Alternative MP-1A continues the current operations that the District has been doing for a long time, and the District staff is familiar with the operation of the current facilities. The alternative MP-1B is very similar to MP-1A regarding operation, except for the operation of a new high-quality effluent plant. Alternative MP-1C and MP-1D also include operating a new effluent pumping station at the vicinity of the existing Badger Mill Creek outfall and could require more effort in operation. The rankings of ease of operation for these alternatives are:

Alternative MP-1A – 10

Alternative MP-1B – 7

Alternative MP-1C – 6

Alternative MP-1D – 6

Staffing Implications

Alternatives MP-1C and MP-1D may have higher staffing requirements due to the operation of a new effluent pumping station. The rankings of staffing implications for these alternatives are:

Alternative MP-1A – 10

Alternative MP-1B – 9

Alternative MP-1C – 8

Alternative MP-1D – 8

2.03. Alternative MP-2: Sugar River WWTP

This alternative is based on pumping all of the wastewater flows generated within the service area of PS 17 to a new satellite treatment plant in the Sugar River watershed for treatment. This alternative includes the following two variations:

- **Alternative MP-2A –** Construction of a new advanced secondary wastewater treatment plant by 2020 with discharge to the main branch of the Sugar River downstream of the confluence with Badger Mill Creek. Average daily flows in 2060 will be 4.3 mgd.
- **Alternative MP-2B –** Construction of a new advanced secondary wastewater treatment plant by 2020 with discharges to the main branch of the Sugar River downstream of the confluence with Badger Mill Creek and to the headwaters of Sugar River near CTH PD northwest of Verona. Average daily flows in 2060 will be 2.5 mgd and 1.8 mgd, respectively, at these two locations.

For both of these planning alternative variations, 3.6 mgd of treated effluent would continue to be pumped from the NSWTP to Badger Mill Creek. Layouts for each of these alternatives are shown in Figure 2.03.1 and 2.03.2. The major component projects for these two alternatives are listed in Table 2.03.1.

Table 2.03.1 Component Projects of Master Planning Alternative MP-2

Facility Name	Component Project	MP-2A	MP-2B
Sugar River WWTP	Construction of a new Sugar River WWTP with capacities of 4.3 mgd (DAF), and 13.7 mgd (DMF). Facility would include processes for effluent polishing to meet the 5 mg/L limit for BOD_5 and TSS. Facility would also be designed to meet a potential 0.075 mg/L TP limit and a 3 mg/L Total N limit.	Yes	Yes
PS17	Firm pumping capacity expansion to average daily flow of 4.37 mgd and peak flow of 13.6 mgd.	Yes	Yes
PS17	Force main from PS 17 to Sugar River WWTP	Yes	Yes
Effluent Pumping Station	Construction of a effluent pumping station to pump flow to the headwaters of the Sugar River near CTH PD	No	Yes
Effluent Force Main	Construction of an effluent force main to convey flow to the headwaters of the Sugar River near CTH PD	No	Yes

For Alternatives MP-2A and MP-2B, two options are available for biosolids disposal. First, the biosolids can be hauled to the NSWTP for anaerobic digestion and then used for land application or other utilization. Second, onsite anaerobic digestion and 180 days of biosolids

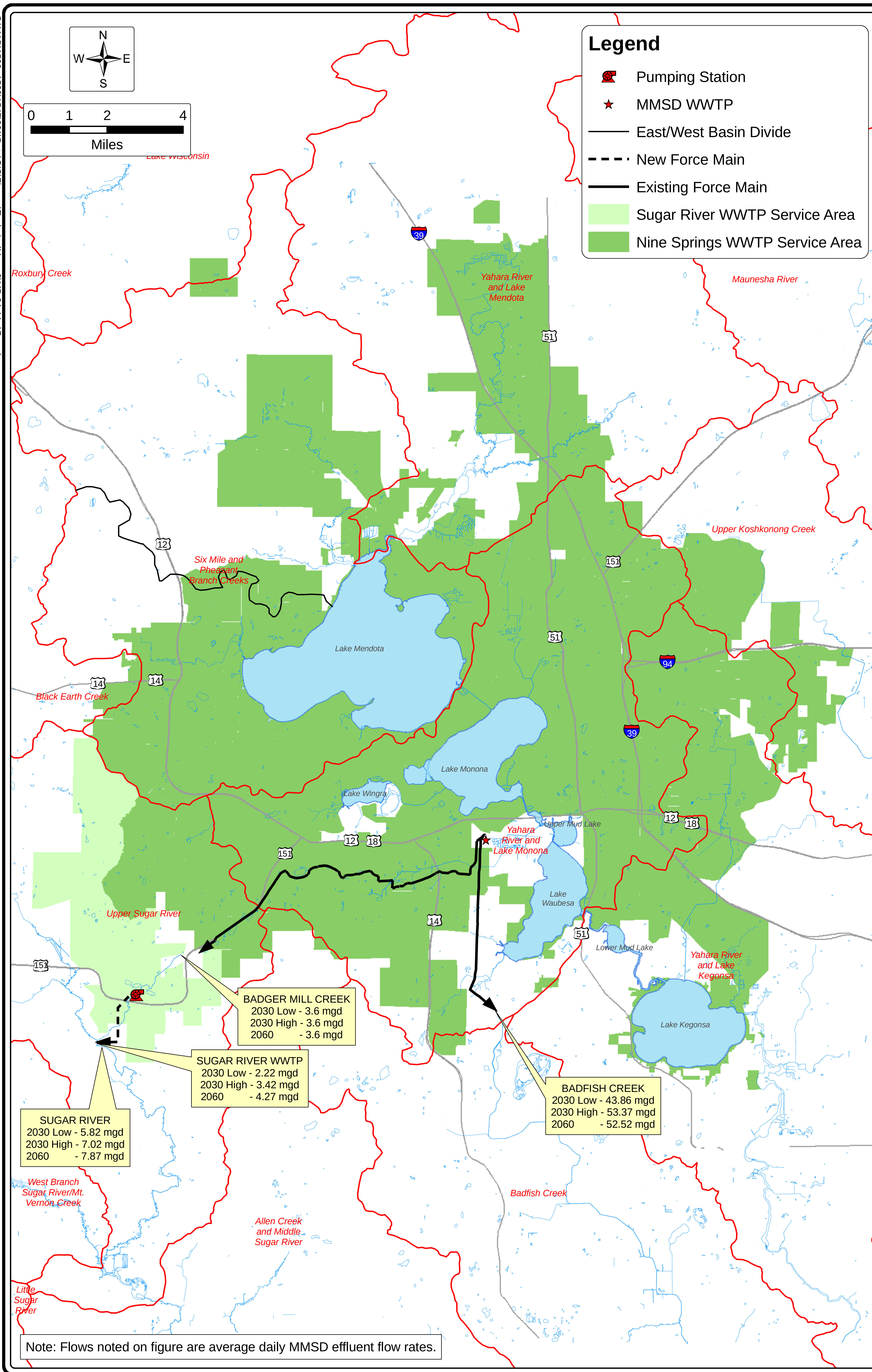

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FIGURE 2.03-1
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ALTERNATIVE MP-2A

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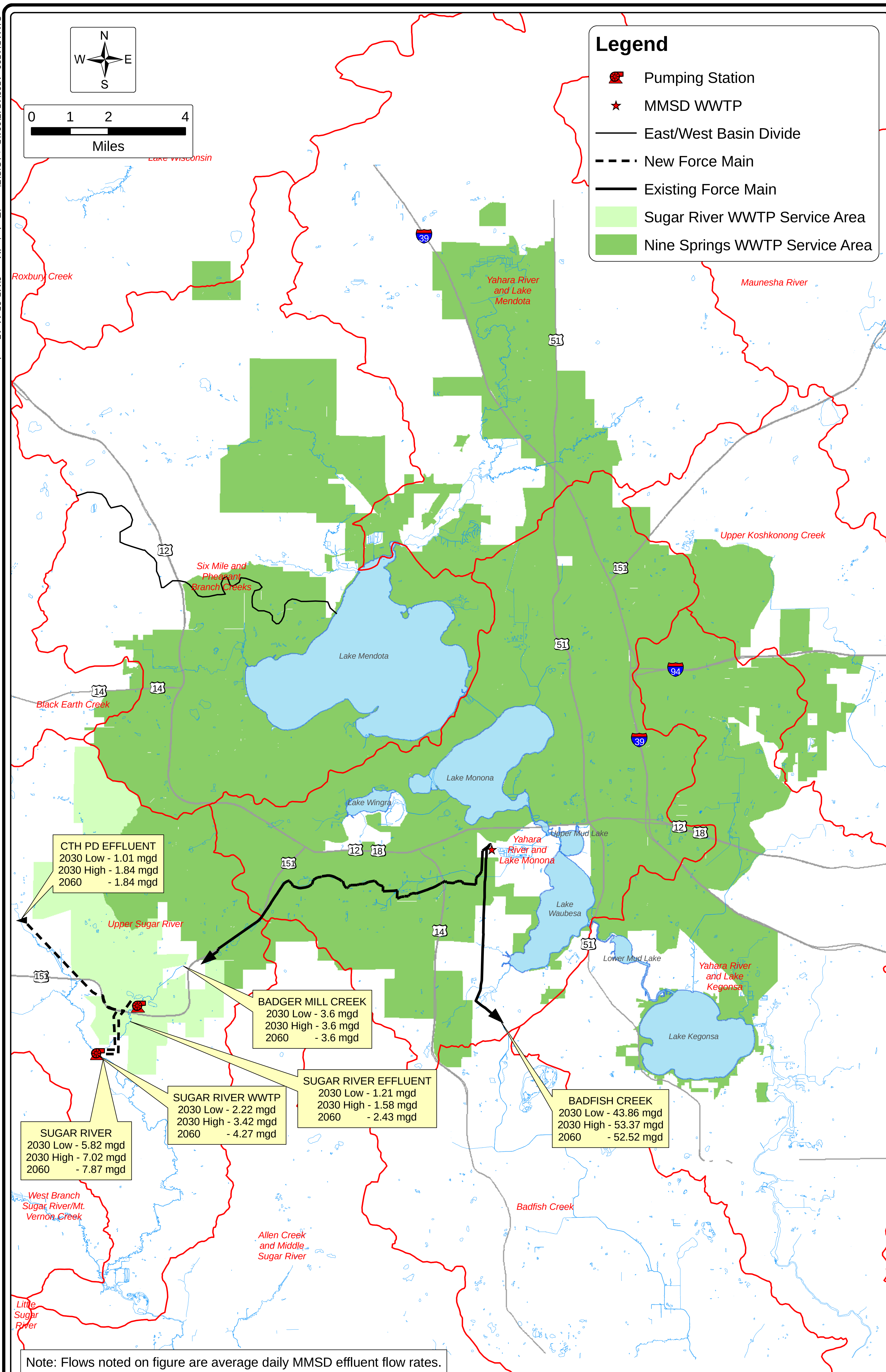

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FIGURE 2.03-2
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ALTERNATIVE MP-2B

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storage can be constructed at the new Sugar River plant. The biosolids can be used for land application in the vicinity of the plant site.

Alternative MP-2 Evaluation

Life Cycle Cost

Life cycle costs were calculated for these planning alternatives based on the costs of construction, operation and maintenance costs, replacement/rehabilitation costs, and salvage values. The results were used for subsequent planning alternative evaluation and ranking.

The results of the total life cycle cost analysis for both alternatives are summarized in Table 2.03.2. Detailed planning alternative life cycle cost calculations are attached in Appendix A.

Table 2.03.2 Alternative MP-2 Life Cycle Cost Analysis

Item	MP-2A	MP-2B
Initial Capital Costs	$67,905,000	$72,305,000
Life Cycle O/M Costs	$32,407,000	$34,036,000
Life Cycle Costs for Facility Improvement & Replacement	$14,028,000	$14,414,000
Salvage Value	($2,261,000)	($2,516,000)
50-Year Total Present Value	$112,079,000	$118,239,000

Public Acceptance

Public acceptance may have a significant impact on implementing these alternatives. Both alternatives involve direct discharges to the Sugar River, which is classified as an ERW and has a downstream impoundment (Lake Belle View). This could generate concerns on the part of the public. However, the public has also expressed support for maintaining the water balance within the Sugar River basin. A new Sugar River plant will be able to achieve this goal. It will be more difficult to gain acceptance for two Sugar River discharge locations as proposed in Alternative MP-2B than the one discharge location in Alternative MP-2A.

Both alternatives should reduce the frequency of emergency discharges of treated NSWTP effluent to Lake Waubesa through Nine Springs Creek under high flows. This should lessen the public's concern of potential negative impacts on the water quality of Lake Waubesa.

The rankings of public acceptance for these alternatives are:

Alternative MP-2A – 4

Alternative MP-2B – 3

Watershed Balance

Alternatives MP-2A and MP-2B will impact the water balance between the Sugar River watershed and the Badfish Creek watershed by retaining wastewater originally generated in the Sugar River watershed in the watershed. Under the current condition, the net base flow loss or increases in the Sugar River watershed at different planning flow projections are shown in Table 2.03.4.

Table 2.03.4 Water Balance of Alternative MP-2A and MP-2B

Flow Projections	Sugar River Watershed Groundwater Withdrawal (mgd)	Total Flow to Sugar River (mgd)	Watershed Net Loss/Increases (mgd)	Watershed Net Loss/Increases (%)
2030 Low	5.03	5.82	0.79	16%
2030 High	6.86	7.02	0.16	2%
2060 Low	6.86	7.02	0.16	2%
2060 High	8.09	7.87	-0.22	-3%

Under all planning flow projections, the water imbalance in the Sugar River watershed ranges from a 3% loss to a 16% increase of the total groundwater withdrawal in the Sugar River Watershed, which is much less significant than the 28% to 56% loss under the alternative MP-1. Alternative MP 2B would have a slightly favored rating over alternative MP2A since it would be returning the water to the Sugar River further up the river and closer to the service areas which generated the wastewater. The watershed balance ranking for these alternatives are:

Alternative MP-2A – 9

Alternative MP-2B – 10

Flexibility/Expandability/Compatibility

Construction of a new wastewater treatment plant in the Sugar River watershed will provide the District with enhanced flexibility since there would be watershed options for routing of wastewater. A second wastewater treatment plant would also provide the District with a reduction of phosphorus entering the Rock River. The flexibility for both alternatives would be high.

The new plant site facility would be expandable to accommodate future growth. The initial facility would likely be sized for the projected 2040 flows with additional modules added depending on actual growth in the PS 17 service area. Therefore expandability for these alternatives would be high. A new WWTP would be a change to current operations and as a result ranks slightly lower in compatibility than alternative MP-1. Overall the flexibility/expandability/compatibility rankings for these alternatives are:

Alternative MP-2A – 8

Alternative MP-2B – 8

Effluent Reuse

Effluent reuse for the Sugar River WWTP would be ranked lower than for the NSWTP due to the lack of potential users for reused effluent unless crop irrigation would be considered a viable option in this area. Overall the ranking of effluent reuse for Alternative MP-2A is low. Alternative 2B includes a new effluent force main which slightly increases its potential for effluent reuse somewhere along or near this force main. The rankings for these two alternatives are:

Alternative MP-2A – 7

Alternative MP-2B – 8

Regulatory Constraints

This alternative provides an increased discharge to an ERW and may require significantly more effort to obtain a WPDES permit. Alternative MP-2A may experience less stringent regulatory constraints since its discharge is to a point on the Sugar River that is already impacted by the current Badger Mill Creek discharge.

Since the new plant discharges to an ERW water body, it is possible that the chloride discharge limit will need to meet the background concentration rather than the concentration calculated based on the provisions of NR 105 and NR 106 of the Wisconsin Administrative Code.

Thermal effluent limits would likely be required for this facility to meet the applicable provisions of NR 106 of the Wisconsin Administrative Code. Additional study of the river may allow for the determination of a site specific limit. The thermal limits for this discharge would potentially be more restrictive than those of NSWTP since it would be a discharge to a cold water fishery. However, based on the available dilution and assuming that 100 % of the dilutional flow is available, thermal limits may not be required.

Alternatively, an option of removing water from the Sugar River and blending this water with the effluent may also mitigate any potential effects that the discharge would have on temperature in the river.

Discharge to the Sugar River will likely result in effluent BOD_5 and TSS limitations of 4 mg/L based on a 7Q10 of 14.3 cfs and a river temperature of 18 degrees centigrade, and about 7 mg/L based on a 7Q10 of 14.3 cfs and a river temperature of 5 degrees centigrade. Ammonia limits for the new discharge would likely be easily met since the facility would be designed for nitrification/denitrification.

The rankings of regulatory constraints for these two alternatives are:

Alternative MP-2A – 4

Alternative MP-2B – 3

Proven Effectiveness

Technologies proposed for this alternative have been proven effective in many operating installations of similar size in the State of Wisconsin. The only exception to this would be the ability of this or any facility to consistently meet a 0.075 mg/L TP limit and a 3 mg/L TN limit. There is limited data available on facilities capable of meeting these limits on a consistent basis. This is a constraint for any of the three discharge locations being evaluated. Based on the effectiveness of the available technologies to meet the limits, the proven effectiveness rating for these two alternatives are:

Alternative MP-2A – 4

Alternative MP-2B – 4

Carbon Footprint

The carbon footprint for this alternative would be lower due to its reduction in the amount of electricity required for pumping. The carbon footprints of these two alternatives are presented in Table 2.03.5. Detailed carbon footprint calculations are included in Appendix B.

Table 2.03.5 Alternative MP-2A Carbon Footprint

Alternative	Electricity Consumption (Mwhr/yr)	Total GHG Emissions (tons CO_2 equivalents/yr)
Alternative 2A	5,885	5,428
Alternative 2B	6,573	6,063

Both of these alternatives have smaller carbon footprints than Alternative MP-1. Alternative MP-2B includes additional pumping of a portion of the plant effluent to the headwaters of the Sugar River, and therefore has higher electricity consumption than Alternative MP-2A. The rankings of carbon footprint for Alternative MP-2A and MP-2B are:

Alternative MP-2A – 10

Alternative MP-2B – 9

Ease of Operation

Since the operation of the new plant would be a different process for the District staff, initially it might be more difficult to operate compared to the existing facilities at the NSWTP. Also, since the plant would serve a smaller population, it would experience greater diurnal flow variations and peak flow variations which would make the plant more susceptible to process upsets.

Lab operations, except for biological examination, would likely continue to occur at the NSWTP. Although this requires sample transport, this is not expected to be a significant issue since the District routinely transports samples from many remote locations throughout its service area as part of its user charge monitoring system.

Biosolids facilities, if provided at this new plant, would likely mimic the District's current liquid operation at the NSWTP and would include biosolids thickening, anaerobic digestion and liquid biosolids storage.

The rankings of ease of operation for these alternatives are:

Alternative MP-2A – 3

Alternative MP-2B – 2

Staffing Implications

These alternatives require more staff than Alternative MP-1 for operation of the new treatment facility. Based on the experience of similar sized facilities, approximately 2 full time equivalent staff would be needed to provide the routine day to day treatment operations and routine maintenance. Most of the lab testing could be conducted at the NSWTP, however samples would have to be transported to the NSWTP. Overall the rankings of staffing implications for these two alternatives are:

Alternative MP-2A – 5

Alternative MP-2B – 5

2.04 Near-Term Planning Alternative Evaluation

The implementation dates and rankings for planning alternatives MP-1 and MP-2 are summarized in Table 2.04.1 and 2.04.2.

Table 2.04.1 Near-Term Master Planning Alternative Evaluation

Project Variable	Sugar River Watershed Service Alternatives					
	1A	1B	1C	1D	2A	2B
Treatment Plant Location	Nine Springs	Nine Springs	Nine Springs	Nine Springs	Sugar River	Sugar River
High Quality Effluent Treatment Design DAF (mgd)	0.0	4.3	4.3	4.3	4.3	4.3
Effluent Discharge Location	Badger Mill Creek	Badger Mill Creek	Sugar River	Headwaters of Sugar River	Sugar River	Headwaters of Sugar River
NSVI Improvements - Year Required						
PS 11 Firm Pumping Capacity	2015	2015	2015	2015	2045	2045
PS 11 Major Condition Upgrade	2015	2015	2015	2015	2015	2015
PS 11 Force Main Capacity	2025	2025	2025	2025	2051	2051
PS 12 Firm Pumping Capacity	2015	2015	2015	2015	2025	2025
PS 12 Major Condition Upgrade	2015	2015	2015	2015	2015	2015
PS 12 Force Main Capacity	>2060	>2060	>2060	>2060	>2060	>2060
PS 17 Firm Capacity	2015	2015	2015	2015	2020	2020
PS 17 Force Main Capacity	2015	2015	2015	2015	2020	2020
NSVI from PS 11 to PS 12	2020	2020	2020	2020	>2060	>2060
NSVI above PS 12	2020	2020	2020	2020	>2060	>2060
Effluent Return Facilities Required						
To Sugar River South of Verona	No	No	Yes	No	No	No
To Sugar River Headwaters	No	No	No	Yes	No	Yes
To Badger Mill Creek	No	Yes	Yes	Yes	No	No
To Badfish Creek	No	No	No	No	No	No

Table 2.04.2 Near-Term Master Planning Alternative Evaluation

	Sugar River Watershed Service Alternatives					
Project Variable	**1A**	**1B**	**1C**	**1D**	**2A**	**2B**
Treatment Plant Location	Nine Springs	Nine Springs	Nine Springs	Nine Springs	Sugar River	Sugar River
Evaluation Criteria						
Life Cycle Cost (in millions)	69.2	103.0	111.6	111.7	112.1	118.2
Relative Life Cycle Cost	1.0	1.5	1.6	1.6	1.6	1.7
Ranking Score	10.0	6.7	6.2	6.2	6.2	5.9
Level of Importance	26					
Weighted Score	**260**	**175**	**161**	**161**	**160**	**152**
Public Acceptance						
Ranking Score	8	9	6	5	4	3
Level of Importance	13					
Weighted Score	**104**	**117**	**78**	**65**	**52**	**39**
Watershed Balance						
Ranking Score	5	8	9	10	9	10
Level of Importance	11					
Weighted Score	**55**	**88**	**99**	**110**	**99**	**110**
Flexibility/Expandability/Compatibility						
Ranking Score	5	6	8	8	8	8
Level of Importance	9					
Weighted Score	**45**	**54**	**72**	**72**	**72**	**72**
Effluent Reuse						
Ranking Score	6	7	8	8	7	8
Level of Importance	9					
Weighted Score	**54**	**63**	**72**	**72**	**63**	**72**
Regulatory Constraints						
Ranking Score	9	7	4	3	4	3
Level of Importance	9					
Weighted Score	**81**	**63**	**36**	**27**	**36**	**27**
Proven Effectiveness						
Ranking Score	8	6	4	4	4	4
Level of Importance	8					

	Sugar River Watershed Service Alternatives					
Project Variable	**1A**	**1B**	**1C**	**1D**	**2A**	**2B**
Treatment Plant Location	Nine Springs	Nine Springs	Nine Springs	Nine Springs	Sugar River	Sugar River
Weighted Score	**64**	**48**	**32**	**32**	**32**	**32**
Carbon Footprint						
Ranking Score	8	6	5	5	10	9
Level of Importance	5					
Weighted Score	**40**	**30**	**25**	**25**	**50**	**45**
Ease of Operation						
Ranking Score	10	7	6	6	3	2
Level of Importance	5					
Weighted Score	**50**	**35**	**30**	**30**	**15**	**10**
Staffing Implications						
Ranking Score	10	9	8	8	5	5
Level of Importance	5					
Weighted Score	**50**	**45**	**40**	**40**	**25**	**25**
Total	**803**	**718**	**645**	**634**	**604**	**584**

3.01 Long-Term Master Plan Alternative (2030-2060)

Long-term alternatives are those planning alternatives that cannot be implemented soon enough to provide relief in the conveyance system; however, they remain potentially viable options beyond the year 2030 for mitigating inter-basin transfers of water, or providing high quality effluent for reuse options. Due to growing demands on available groundwater supplies and the long-term goal of stabilizing the groundwater aquifer operating level in the Dane County area, high quality effluent utilization could be a promising way to solve these issues in the future, especially if population growth occurs as expected. The following two long-term alternatives emphasizing effluent reuse were selected in TM-7 for further evaluation in this technical memo. These two alternatives have potential to be implemented after 2030 and provide high quality effluent to various locations for reuse options and to mitigate inter-basin transfer of water.

- **Alternative MP-3 – Centralized High Quality Effluent Treatment & Distribution:** This alternative includes construction of facilities at the NSWTP for providing effluent of high quality for reuse options including, stream flow augmentation, infiltration, industrial reuse, or turf irrigation.
- **Alternative MP-4 – Decentralized High Quality Effluent Treatment Facilities**:

  This alternative includes construction of facilities northeast of the Dane County Regional Airport. The new treatment plant will receive wastewater flows tributary to PS13 or both PS13 and PS14. Effluent from this facility could be used for stream flow augmentation to Starkweather Creek, wetland restoration at Cherokee Marsh, groundwater infiltration, industrial reuse water or turf irrigation.

Due to the long planning horizon, specific effluent reuse projects cannot be clearly defined at this stage. However, the District would like to take a proactive approach to study the potential economic, technical and environmental factors that may impact implementation of future effluent reuse programs. To facilitate the study, high quality effluent facilities with capacities of 4 mgd and 10 mgd, representing both small and medium sized effluent reuse programs, were chosen for this evaluation. The PS13 service area was selected as the location for the reuse facility. Potential reuse applications could include industrial reuse, stream flow augmentation, turf irrigation, and groundwater infiltration. Layouts for the reuse facilities are shown in Figure 3.01.1 and 3.03.1.

The following three long-term alternatives were also identified in TM-7 but not recommended for further evaluation due to the strict regulatory constraints, high construction and operation costs, lack of proven technical feasibilities, and potential strong public resistance. However these alternatives may become more viable in the future with changes in the political environment, water resource demand, or improvements in wastewater treatment technologies.

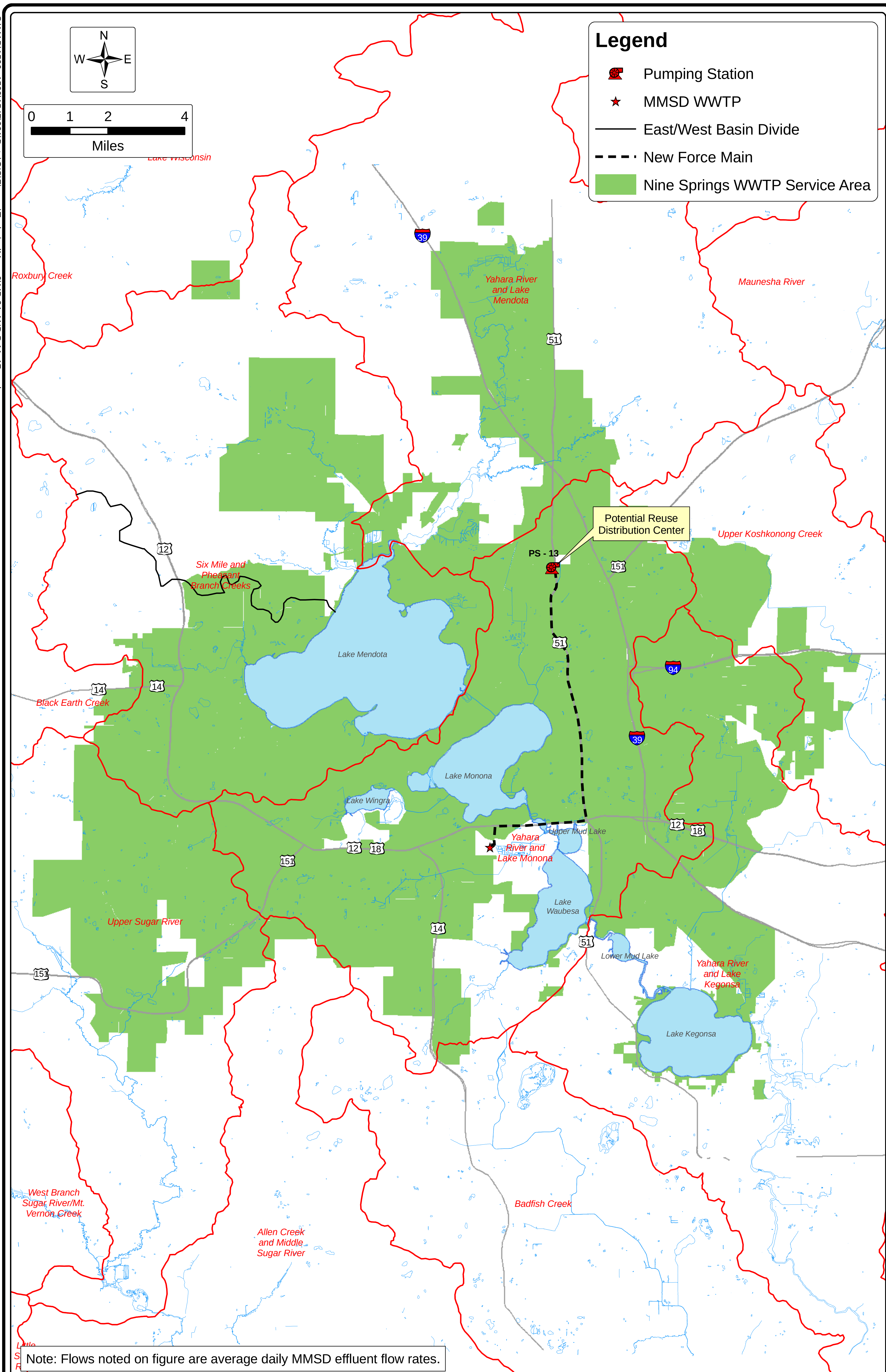

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FIGURE 3.01-1
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ALTERNATIVE MP-3A MP-3B

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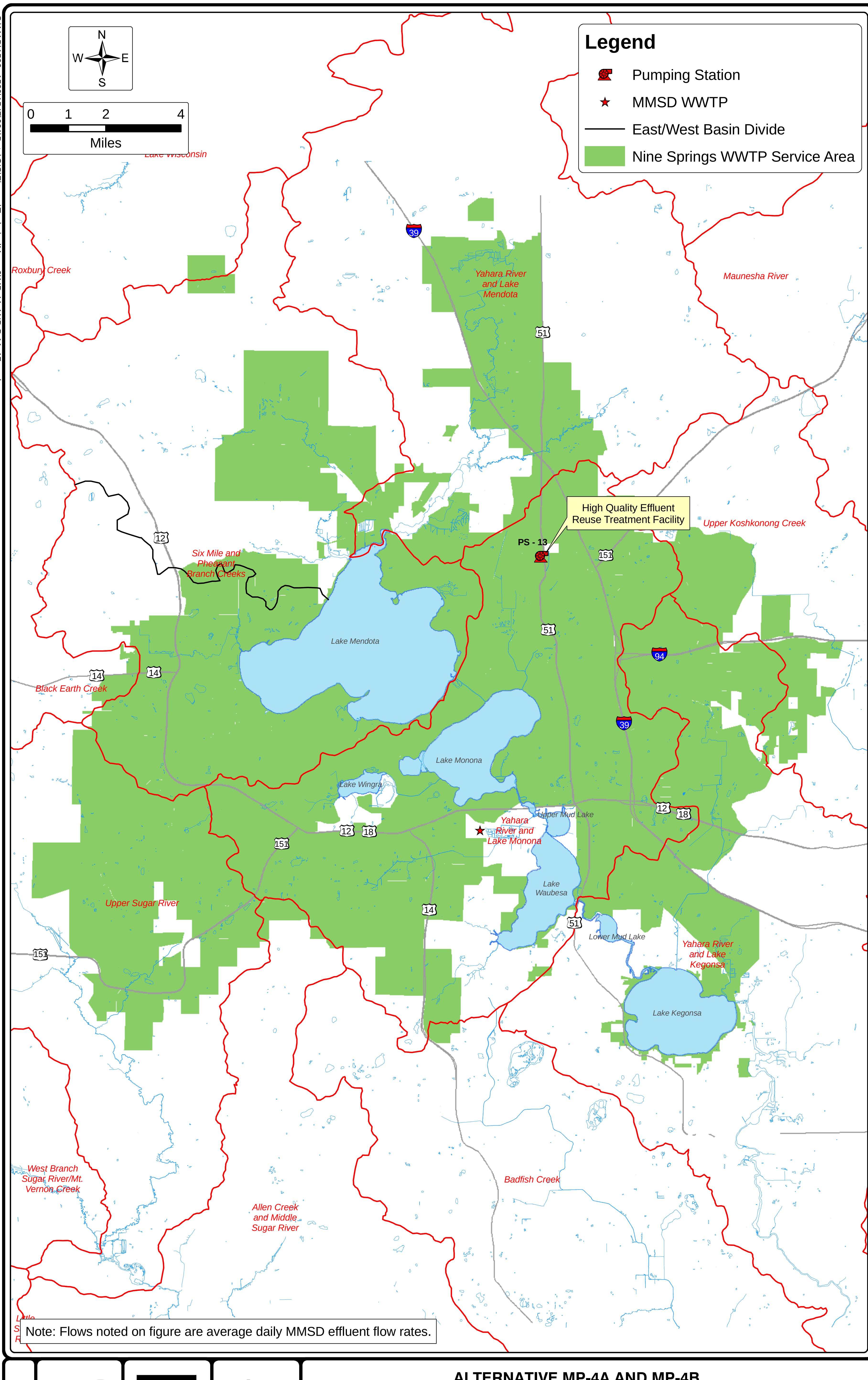

Note: Flows noted on figure are average daily MMSD effluent flow rates.

FIGURE 3.03-1
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ALTERNATIVE MP-4A AND MP-4B

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- **Mendota WWTP –** This project includes construction of a new WWTP north of Lake Mendota near the Yahara River to serve the Yahara River watershed north of Lake Mendota. The new plant would discharge high quality effluent into the Yahara River upstream of Lake Mendota.
- **Sun Prairie WWTP Expansion –** This project provides sewer service for the portion of the District's future service area in the Koshkonong Creek watershed by directing flow from this area to the City of Sun Prairie WWTP for treatment.
- **Stoughton WWTP Expansion –** This project would redirect flow from PS 7 and 9 service areas to an expanded existing Stoughton WWTP. Implementation of this project includes the construction of a parallel treatment plant to treat the wastewater diverted from the MMSD system. Biosolids treatment would be provided by expanding the existing biosolids treatment train at the Stoughton WWTP.

3.02 Alternative MP-3: Centralized High Quality Effluent Treatment and Distribution Facilities

The new facility would be constructed after 2030 with a capacity of either 4 mgd (Alternative MP-3A) or 10 mgd (Alternative MP-3B). Alternative MP-3A is directly comparable to Alternative MP-4A, and Alternative MP-3B is directly comparable to Alternative MP-4B.

Planning alternatives MP-3A and MP-3B include the additional treatment facilities at the NSWTP to produce high quality effluent as well as the pumping facilities to return the high quality effluent to the PS 13 site northeast of the Dane County Regional Airport. Actual implementation of effluent reuse may not require water quality as stringent as would be produced by a high quality effluent facility if the end use water quality requirements are lower, or if the end user provides additional treatment that would meet their specific needs and comply with applicable Wisconsin Administrative Code requirements. The component projects included in this alternative are listed in the following table.

Table 3.02.1 Component Projects of Master Planning Alternative MP-3

Facility Name	Component Project
High Quality Effluent Treatment Facilities at NSWTP	Construction of new high quality effluent facilities with capacity of either 4 mgd (Alternative MP-3A) or 10 mgd (Alternative MP-3B) in NSWTP. Facilities would include processes for effluent polishing to meet the 5 mg/L limit for BOD_5 and TSS. Facility would also be designed to meet a potential 0.075 mg/L TP limit and a 3 mg/L Total N. The level of treatment of the facilities will be further determined after the treated effluent utilizations become better defined in the future.
Effluent Return Pumping Station	Construction of a new pumping station at the NSWTP to return high quality effluent to PS 13 area.
Effluent Return Force Main	Construction of a new force main to return high quality effluent to PS 13 area.

Alternative MP-3 Evaluation

Life Cycle Cost

Life cycle costs were calculated for this planning alternative based on the costs of construction, operating and maintenance costs, and replacement/rehabilitation costs. The results of the life cycle analysis are presented in the following table. Detailed alternative life cycle costs are attached in Appendix A.

Table 3.02.2 Alternative MP-3 Life Cycle Cost Analysis

Item	Alternative MP-3A	Alternative MP-3B
Initial Capital Costs	$27,100,000	$45,500,000
Life Cycle O/M Costs	$20,024,000	$47,949,000
Life Cycle Costs for Facility Improvement & Replacement	$4,631,000	$8,362,000
Salvage Value	($1,103,000)	($1,730,000)
50-Year Total Present Value	$50,652,000	$100,081,000

Public Acceptance

In Alternatives MP-3A and MP-3B, the new high quality effluent treatment facilities and effluent pumping station will be constructed at the current NSWTP property, and a new piping system will be constructed to convey treated high quality effluent to the PS 13 area or other effluent reuse locations. The treatment and pumping facilities will be constructed within MMSD's property; therefore, the public acceptance would be high, although there may be some public resistance related to the construction of the new effluent conveyance system.

Since Starkweather Creek discharges to Lake Monona, if the treated effluent was used for stream flow augmentation in Starkweather Creek, the public may have concerns. More public education programs and effluent reuse application demonstrations could be used to determine if this type of discharge would provide a net environmental benefit in the opinion of the regulators and the public.

Since there is no significant amount of effluent reuse occurring in either Madison or the State of Wisconsin, the public's acceptance of reuse is unknown. However, water reuse is a commonly accepted practice in other parts of the United States where water resources are limited. Infiltration of highly treated effluent is already occurring in the Madison area in relatively small scale. MMSD is providing effluent for a small demonstration of turf irrigation with treated effluent at the City of Fitchburg's Nine Springs Golf Course and has not encountered any public concerns. Public acceptance for most types of effluent reuse will be dependent on both the quality of the effluent and the perceived need to minimize further groundwater table depletions.

Based on the above discussion, the overall ranking of public acceptance for this alternative is 6.

Watershed Balance

Effluent reuse has the potential for supplementing surface and groundwater resources in the Madison area. Infiltration could have a positive impact on stream base flow in the areas where infiltration occurs. Direct stream flow augmentation would accomplish a similar result. The 2004 Dane County groundwater model predicted a net base flow loss of 47 mgd at the Yahara River outlet of Lake Waubesa in 2030 if the current effluent diversion practices continue.

Alternative MP-3A or MP-3B would pump 4 mgd or 10 mgd, respectively, of high quality effluent to the PS 13 area northeast of the Dane County Regional Airport within the Yahara River watershed. Since all effluent currently produced at the NSWTP is pumped south of the Madison lakes, these alternatives will result in a redistribution of water within the watershed. A variety of reuse options could be available, including, flow augmentation, turf irrigation, groundwater infiltration, and industrial uses. Starkweather Creek has experienced a reduction in dry weather base flow over the years, caused by the high percentage of impervious surfaces in the watershed and groundwater pumping in Madison area, and could be a good candidate to receive flow augmentation.

Due to a decreasing net loss of water from the Yahara River watershed with this alternative, the watershed balance ranking for this alternative is 8.

Flexibility/Expandability/Compatibility

The new high quality effluent treatment and pumping facilities will be constructed at the NSWTP and use the effluent from the existing secondary treatment process as source water to produce high quality effluent for various reuse options. The new facilities will be essentially tertiary treatment facilities to the existing secondary treatment facilities at the NSWTP. These facilities will probably require in-plant pumping since they will not be compatible with the existing hydraulic profile. The District currently owns 627 acres of land at the NSWTP site with approximately 204 acres available for future expansion. Therefore, the expandability for this alternative is high. With the new facilities the District will have the flexibility to convey high quality effluent to the Sugar River watershed, the Badfish Creek watershed, or the Yahara River watershed in the vicinity of PS 13 for various effluent reuse options.

The ranking of flexibility/expandability/compatibility for this alternative is 7.

Effluent Reuse

Under this alternative, either a additional 4 mgd or 10 mgd of high quality effluent will be available for various effluent reuse options in the Sugar River watershed, the Badfish Creek watershed, the Yahara River watershed in the vicinity of PS 13, the Nine Springs area, or other areas served by MMSD effluent return force mains. The effluent reuse applications include stream flow augmentation, groundwater infiltration, industrial reuse, and turf irrigation. The high quality flow can also be used for stream augmentation at the Yahara River upstream of Lake Waubesa via Nine Springs Creek. The NSWTP location also provides an excellent location for potential industrial reuse since it is located in an area zoned for commercial and industrial uses. The ranking of potential for effluent reuse for this alternative is 8.

Regulatory Constraints

This alternative includes a new discharge location in the vicinity of PS13 northeast of the Dane County Regional Airport and will likely have more stringent regulatory constraints. The new discharge may need to meet more stringent phosphorus and total nitrogen discharging limits. Potential limits could be 0.075 mg/L of total phosphorus and 3 mg/L of total nitrogen. Thermal effluent limits would also likely be applicable for selected reuse options.

This alternative offers the potential to pursue a variety of effluent reuse options, some of which may have even more stringent regulatory limits than those identified in the above paragraph. For example, effluent may need to meet more stringent phosphorus and nitrogen limits if it is used to augment flow in Starkweather Creek, since this creek discharges to Lake Monona. If the effluent is going to be used for groundwater infiltration, there is a limitation of chlorides based on the provisions in Chapter NR 140 of the Wisconsin Administrative Code.

Considering all the regulatory issues, the ranking of regulatory constraints for this alternative is 3.

Proven Effectiveness

Few of the available technologies for effluent reuse are currently employed in Wisconsin, but have proven effective in California and other states with high effluent reuse. The only exceptions to this would be the ability of this facility to consistently meet a 0.075 mg/L TP limit and a 3 mg/L TN limit. There is limited data available on facilities capable of meeting these limits on a consistent basis. The ranking of proven effectiveness for this alternative is 4.

Carbon Footprint

Since a higher level of treatment is required for this alternative, additional energy will be consumed. This alternative also includes pumping a maximum of 4 mgd or 10 mgd of high quality effluent to the PS 13 area for effluent reuse options. As a result, electricity consumption would be higher.

Compared to the alternative MP-4 discussed in the next section, Alternative MP-3 would have higher energy consumption and therefore a larger carbon footprint. The ranking of the carbon footprint for this alternative is 5.

Ease of Operation

The new facilities would represent a distinct change in types of facilities currently operated by MMSD staff. However, the new facilities would be built at the NSWTP. The District's experienced operators, lab testing facilities, instrumentation and control system, and other resources will be readily available to support the operations of the new facilities. With all these factors considered, the ranking of ease of operation for this alternative is 5.

Staffing Implications

Since this alternative would be housed at the NSWTP, the overall staffing would likely remain similar to the current level or slightly higher. However, some required skills for operating the new facilities could be different from what the plant staff has acquired. Additional training for the plant staff or recruiting of workers with certain skill sets may become necessary for successful operation of the new facilities. With all those impacts considered, the overall ranking of staffing implications for this alternative is 5.

3.03 Alternative MP-4: Decentralized High Quality Effluent Treatment Facilities

The new facility could be constructed in 2030 with a capacity of either 4 mgd (Alternative MP-4A, Starkweather Creek WWTP) or 10 mgd (Alternative MP-4B, PS13 and PS14 Service Area WWTP). Alternative MP-4A is directly comparable to Alternative MP-3A, and Alternative MP-4B is directly comparable to Alternative MP-3B.

Actual implementation of effluent reuse may not require water quality as stringent as would be produced by a high quality effluent facility if the end use would not require such a high quality or if the end user would provide additional treatment that would meet their specific needs and comply with Wisconsin Administrative Code requirements. The component projects included in this alternative are listed in the following table.

Table 3.03.1 Component Projects of Master Planning Alternative MP-4

Facility Name	Component Project
Starkweather Creek WWTP	Construction of a new high quality effluent treatment plant with a capacity of 4 mgd. The plant would include processes for effluent polishing to meet the 5 mg/L limit for BOD_5 and TSS. Facility would also be designed to meet a potential 0.075 mg/L TP limit and a 3 mg/L Total N limit.
PS13 and PS14 Service Area WWTP	Construction of a new high quality effluent treatment plant with a capacity of 10 mgd. The plant would include processes for effluent polishing to meet the 5 mg/L limit for BOD_5 and TSS. Facility would also be designed to meet a potential 0.075 mg/L TP limit and a 3 mg/L Total N limit.

Alternative MP-4 Evaluation

Life Cycle Cost

Life cycle costs were calculated for this planning alternative based on the costs of construction, operating and maintenance costs, and replacement/rehabilitation costs. The results of the life cycle cost analysis are presented in the following table. Detailed alternative life cycle costs are attached in Appendix A.

Table 3.03.2 Alternative MP-4 Life Cycle Cost Analysis

Item	Alternative MP-4A	Alternative MP-4B
Initial Capital Costs	$40,000,000	$80,000,000
Life Cycle O/M Costs	$26,167,000	$56,670,000
Life Cycle Costs for Facility Improvement & Replacement	$10,292,000	$20,584,000
Salvage Value	($912,000)	($1,825,000)
50-Year Total Present Value	$75,547,000	$155,429,000

Public Acceptance

In the Alternatives MP-4A and MP-4B, the new high quality effluent facilities will be constructed in the vicinity of PS 13, northeast of the Dane County Regional Airport. The treated high quality effluent will be discharged in the PS 13 area or other effluent reuse locations. The new plant may be highly visible from USH 51, but it would likely not pose too large an issue for public acceptance if the facility was properly designed to blend in with the industrial nature of the airport area.

Since Starkweather Creek discharges to Lake Monona, if the treated effluent was used for stream flow augmentation in Starkweather Creek, the public may have concerns. More public education programs and effluent reuse application demonstrations could be used to determine if this type of discharge would provide a net environmental benefit in the opinion of the regulators and the public.

Since there is no significant amount of effluent reuse occurring in either Madison or the State of Wisconsin, the public's acceptance of reuse is unknown. Many municipalities around the United States routinely use wastewater effluent for turf irrigation, particularly on golf courses. Infiltration of highly treated effluent is already occurring in the Madison area on a relatively small scale. MMSD is providing effluent for a small demonstration of turf irrigation with treated effluent at the City of Fitchburg's Nine Springs Golf Course with minimal public concerns. Public acceptance for most types of effluent reuse will be dependent on both the quality of the effluent and the perceived need to minimize further groundwater table depletions.

With all the considerations above, the overall ranking of public acceptance for this alternative is 4.

Watershed Balance

Effluent reuse has the potential for supplementing surface and groundwater resources in the Madison area. Infiltration could have a positive impact on stream base flow in the areas where infiltration occurs. Direct stream flow augmentation would accomplish a similar result. The 2004 Dane County groundwater model predicted a net base flow loss of 47 mgd at the Yahara River outlet at Lake Waubesa in 2030 if the current effluent diversion practices continue.

Alternative MP-4A or MP-4B would treat a maximum of 4 mgd or 10 mgd of wastewater respectively to a high effluent quality in the vicinity of PS 13 northeast of the Dane County Regional Airport within the Yahara River watershed, and would improve the imbalanced inter-basin water transfer. Since all effluent currently produced at the NSWTP is pumped south of the Madison lakes, these alternatives will result in a redistribution of water within the watershed. The high quality effluent could be used for flow augmentation in Starkweather Creek, turf irrigation, groundwater infiltration, and industrial uses. Starkweather Creek has experienced a reduction in dry weather base flows over the years caused by the high percentage of impervious surfaces in the watershed and groundwater pumping in the Madison area, and could be a good candidate to receive flow augmentation.

Due to a decreasing net loss of water from the Yahara River watershed with this alternative, the watershed balance ranking for this alternative is 8.

Flexibility/Expandability/Compatibility

The new high quality effluent treatment plant will be a stand-alone facility located northeast of the Dane County Regional Airport. The new plant will receive wastewater flows generated in the PS 13 and PS 14 service areas to produce high quality effluent for various reuse options. Since the new plant will include a complete process line at a remote site, its compatibility to the existing system may not be as good as the Alternative MP-3. The new plant site can be sized and designed to allow future expansion to accommodate potential future growth. Compared to the Alternative MP-3, this alternative has more limited options for effluent reuse locations. The high quality effluent from the new plant can only be utilized in the PS 13 and nearby areas. With all these factors being considered, the ranking of flexibility/expandability/compatibility for this alternative is 5.

Effluent Reuse

Under this alternative, either a maximum of 4 mgd or 10 mgd of high quality effluent will be available for various effluent reuse options in the vicinity of PS 13. The effluent reuse applications include stream flow augmentation in Starkweather Creek, groundwater infiltration, industrial reuse, and turf irrigation. However, compared to the Alternative MP-3, this alternative has fewer options for effluent reuse locations. Without the conveyance system to move treated effluent to other areas served by MMSD, the potential of effluent utilization would rely primarily on opportunities in the vicinity of PS 13. The ranking of potential for effluent reuse for this alternative is 6.

Regulatory Constraints

This alternative includes a new discharge location in the vicinity of PS13 northeast of the Dane County Regional Airport and will likely have more stringent regulatory constraints. The new discharge in the PS 13 area may need to meet more stringent phosphorus and total nitrogen discharge limits. Potential limits could be 0.075 mg/L of total phosphorus and 3mg/L of total nitrogen. Thermal effluent limits would also likely be applicable for select reuse options.

This alternative offers the potential to pursue a variety of effluent reuse options, some of which may have even more stringent regulatory limits than those identified in the above paragraph. For example, effluent may need to meet more stringent phosphorus and nitrogen limits if it is used to augment flow in Starkweather Creek, since this creek discharges to Lake Monona. If the effluent is going to be used for groundwater infiltration, there is a limitation of chlorides based on the provisions in Chapter NR 140 of the Wisconsin Administrative Code.

With all the above considerations, the ranking of regulatory constraints for this alternative is 3.

Proven Effectiveness

Few of the available technologies for effluent reuse are currently employed in Wisconsin, but have proven effective in California and other states with high effluent reuse. The only exceptions to this would be the ability of this facility to consistently meet a 0.075 mg/L TP limit and a 3 mg/L TN limit. There is limited data available on facilities capable of meeting these limits on a consistent basis. Therefore the ranking for proven effectiveness for this alternative is 4.

Carbon Footprint

Compared to the alternative MP-3 discussed in the previous section, Alternative MP-4 has lower energy consumption and hence a smaller carbon footprint due to the elimination of pumping wastewater from PS 13 to the NSWTP and then returning the treated high quality effluent back to PS 13 area. The ranking of the carbon footprint for this alternative is 6.

Ease of Operation

In this alternative, the District staff needs to provide full scale treatment plant operation at a remote site, and the new facilities would represent a distinct change in types of facilities currently operated by MMSD staff. In addition, the District’s resources will not be as readily available in supporting operation as the Alternative MP-3. With all these factors considered, the ranking of ease of operation for this alternative is 2.

Staffing Implications

This alternative requires more staff for operation than the Alternative MP-3. Based on the experience of similar sized facilities, approximately 2 full time staff would be needed to provide the routine day to day treatment operations and routine maintenance. Most of the lab testing can be conducted at the NSWTP, however samples have to be transported to the NSWTP. If a centralized biosolids disposal solution is adopted, waste biosolids generated in the new plant will be transported to the NSWTP for disposal after being thickened onsite. The overall ranking of staff implications for this alternative is 3.

3.04 Long-Term Master Plan Alternative Evaluation

The rankings for planning alternatives MP-3A and MP-4A are summarized in Table 3.04.1 and 3.04.2.

Table 3.04.1 Long-Term Master Planning Alternative Summary (4 MGD Plant)

	High Quality Effluent Treatment Plant Alternatives	
Project Variable	**MP-3A**	**MP-4A**
Treatment Plant Location	Nine Springs	Northeast of the Dane County Regional Airport
Treatment Plant Design ADF (mgd)	4.0	4.0
Effluent Discharge Location	Yahara River Watershed	Yahara River Watershed
Effluent Return Pump Capacity (mgd)	4.0	0
Effluent Return Force Main Capacity (mgd)	10.0	0

Table 3.04.2 Long-Term Master Planning Alternative Evaluation (4 MGD Plant)

	High Quality Effluent Treatment Plant Alternatives	
Project Variable	**MP-3A**	**MP-4A**
Treatment Plant Location	Nine Springs	Northeast of the Dane County Regional Airport
Evaluation Criteria		
Life Cycle Cost	50.7	75.5
Relative Life Cycle Cost	1.0	1.5
Level of Importance	26	
Weighted Score	**260**	**175**
Public Acceptance		
Ranking Score	6	4
Level of Importance	13	
Weighted Score	**78**	**52**
Watershed Balance		
Ranking Score	8	8

Project Variable	High Quality Effluent Treatment Plant Alternatives MP-3A	MP-4A
Level of Importance	11	
Weighted Score	**88**	**88**
Flexibility/Expandability/Compatibility		
Ranking Score	7	5
Level of Importance	9	
Weighted Score	**63**	**45**
Effluent Reuse		
Ranking Score	8	6
Level of Importance	9	
Weighted Score	**72**	**54**
Regulatory Constraints		
Ranking Score	3	3
Level of Importance	9	
Weighted Score	**27**	**27**
Proven Effectiveness		
Ranking Score	4	4
Level of Importance	8	
Weighted Score	**32**	**32**
Carbon Footprint		
Ranking Score	5	6
Level of Importance	5	
Weighted Score	**25**	**30**
Ease of Operation		
Ranking Score	5	2
Level of Importance	5	
Weighted Score	**25**	**10**
Staffing Implications		
Ranking Score	5	3
Level of Importance	5	
Weighted Score	**25**	**15**
Total	**695**	**528**

Table 3.04.3 Long-Term Master Planning Alternative Summary (10 MGD Plant)

Project Variable	High Quality Effluent Treatment Plant Alternatives	
	MP-3B	MP-4B
Treatment Plant Location	Nine Springs	Northeast of the Dane County Regional Airport
Treatment Plant Design ADF (mgd)	10.0	10.0
Effluent Discharge Location	Yahara River Watershed	Yahara River Watershed
Effluent Return Pump Capacity (mgd)	10.0	0
Effluent Return Force Main Capacity (mgd)	10.0	0

Table 3.04.4 Long-Term Master Planning Alternative Evaluation (10 MGD Plant)

Project Variable	High Quality Effluent Treatment Plant Alternatives	
	MP-3B	MP-4B
Treatment Plant Location	Nine Springs	Northeast of the Dane County Regional Airport
Evaluation Criteria		
Life Cycle Cost	100.1	155.4
Relative Life Cycle Cost	1.0	1.6
Level of Importance	26	
Weighted Score	**260**	**167**
Public Acceptance		
Ranking Score	6	4
Level of Importance	13	
Weighted Score	**78**	**52**
Watershed Balance		
Ranking Score	8	8
Level of Importance	11	
Weighted Score	**88**	**88**
Flexibility/Expandability/Compatibility		
Ranking Score	7	5
Level of Importance	9	
Weighted Score	**63**	**45**
Effluent Reuse		
Ranking Score	8	6
Level of Importance	9	
Weighted Score	**72**	**54**
Regulatory Constraints		
Ranking Score	3	3
Level of Importance	9	
Weighted Score	**27**	**27**
Proven Effectiveness		
Ranking Score	4	4
Level of Importance	8	

Project Variable	High Quality Effluent Treatment Plant Alternatives	
	MP-3B	MP-4B
Weighted Score	**32**	**32**
Carbon Footprint		
Ranking Score	5	6
Level of Importance	5	
Weighted Score	**25**	**30**
Ease of Operation		
Ranking Score	5	2
Level of Importance	5	
Weighted Score	**25**	**10**
Staffing Implications		
Ranking Score	5	3
Level of Importance	5	
Weighted Score	**25**	**15**
Total	**695**	**520**

4.01 Near-Term Master Plan Alternative Implementation Recommendation

Based on the evaluation of the six planning alternatives for MMSD's operation in the Sugar River watershed, alternative MP-1A has the highest total score. Its high score is due largely to its lowest life cycle cost, fewer regulatory constraints, long track record of proven effectiveness, ease of operation and minimal staffing implications. Overall alternative MP-1A achieves the highest cost efficiency in providing wastewater conveyance and treatment service in MMSD's westside service area. However, alternative MP-1A will not be able to alleviate the issue of imbalanced inter-basin water transfer. By pumping an additional 4.3 mgd of treated effluent to the Sugar River watershed, the Sugar River base flow reduction would be avoided. However, the additional total life cycle costs would be $34 million, assuming the current discharge limits to Badger Mill Creek and Badfish Creek stay unchanged, but higher quality effluent discharge limits would be required for discharges to the Sugar River. If future regulations require higher quality effluent for both Badfish Creek and Badger Mill Creek discharges, the cost to avoid this base flow reduction may be insignificant.

Alternative MP-1B includes construction of high quality effluent treatment facilities at the NSWTP and pumping of both regular and high quality treated effluent to the Badger Mill Creek outfall through the existing force main. This alternative has the second highest total score and also can achieve high efficiency in providing MMSD's current service in the area. In 2060, this alternative returns a total of 7.9 mgd of treated effluent to the Sugar River via Badger Mill Creek, and can effectively alleviate the imbalanced inter-basin water transfer issue. Since the increased flow is a higher quality effluent, it will not significantly increase the current TP or TN loads to Badger Mill Creek, and therefore it may have less regulatory constraints for implementation.

Alternatives MP-1C, MP-1D and MP-2A represent centralized and decentralized approaches to solve the watershed balance issue. These three alternatives discharge the same amount of treated effluent to Badger Mill Creek and the Sugar River, and therefore will achieve similar benefits of watershed balance. Alternatives MP-1C and MP-2A would have identical discharge locations. Alternative MP-1D would use a Sugar River headwaters discharge location. Alternatives MP-1C and MP-1D would provide more potential for effluent reuse than alternative MP-2A. Alternatives MP-1C, MP-1D and MP-2A have similar life cycle costs; however, Alternative MP-1C is favored over MP -1D and MP-2A due to its higher rankings in public acceptance, effluent reuse potential, ease of operation, and staffing implications.

Alternative MP-2B represents a decentralized approach to solve the watershed balance issue. This alternative discharges the same amount of treated effluent to Badger Mill Creek and the Sugar River as alternatives MP-1C, MP-1D and MP-2A, but the Sugar River discharge is split between a headwaters location and a location downstream of the confluence with Badger Mill Creek. It will achieve slightly better benefits of watershed balance compared to alternatives MP-1C, MP-1D and MP-2A and slightly higher potential for effluent reuse than alternative MP-2A. The total life cycle cost increases by $6 million to achieve this better result.

According to the evaluation results, Alternative MP-1A appears to be the most effective alternative for providing service in the Sugar River watershed. If more stringent discharge limits are implemented, a high quality effluent treatment process will also be added at the NSWTP, which will make this alternative less favorable over the other alternatives. Currently there is no impact on the base flow in Badger Mill Creek or the Sugar River due to the return of effluent to Badger Mill Creek. As more development occurs in the Sugar River basin, base flow will be reduced in the Sugar River. If the reduction in base flow in the Sugar River were to become an issue that required mitigation, alternative MP-1B should then be considered for implementation to alleviate the base flow reduction while still maintaining relatively high cost efficiency. Alternatives MP-1C and MP-1D address base flow augmentation in the Sugar River and reduce the flow in Badger Mill Creek to its more normal levels. If the higher flows in Badger Mill Creek became an issue, alternative MP-1C or MP-1D could then be considered for implementation. Since the life cycle costs for alternatives MP-1B, MP-1C, MP-1D, MP-2A, and MP-2B are relatively close, more detailed facility planning is recommended to more accurately determine the costs of these alternatives before a final decision is made.

Reduction of inflow/infiltration (I/I) to the existing conveyance system is an important element for the areas that experience high groundwater in wet weather conditions. Effective I/I reduction could delay the need for major capital improvement projects required to expand the capacities of the conveyance system and treatment facilities. Therefore programs to reduce I/I are recommended for all planning alternatives.

4.02 Long-Term Master Plan Alternative (4 MGD Capacity) Implementation Recommendation

Based on the evaluation of the two long-term planning alternatives for a high quality effluent treatment plant with 4 mgd capacity, Alternative MP-3A has higher ranking than MP-4A for almost all ranking criteria except for carbon footprint. Alternative MP-3A has significantly lower life cycle cost than MP-4A due to its lower operational cost achieved through economy of scale. Alternative MP-3A also has higher public acceptance since most of the new facilities will be constructed at the current NSWTP property. Alternative MP-3A also has higher flexibility for effluent reuse options. Therefore MP-3A is recommended for implementation of a high quality effluent treatment plant with 4 mgd capacity.

4.03 Long-Term Master Plan Alternative (10 MGD Capacity) Implementation Recommendation

Based on the evaluation of the two long-term planning alternatives for a high quality effluent treatment plant with 10 mgd capacity, Alternative MP-3B has higher ranking than MP-4B for almost all ranking criteria except for carbon footprint. Alternative MP-3B has significantly lower life cycle cost than MP-4B due to its lower operational cost achieved through economy of scale. Alternative MP-3B also has higher public acceptance since most of the new facilities will be constructed at the current NSWTP property. Alternative MP-3B also has higher flexibility for effluent reuse options. Therefore MP-3B is recommended for implementation of a high quality effluent treatment plant with 10 mgd capacity.

Appendix A

Master Planning Alternative Life Cycle Cost Calculations

MALCOLM PIRNIE

Master Planning Alternative Life Cycle Cost Calculations

BACKGROUND

This master planning alternative life cycle cost calculation was prepared as part of the Master Plan Technical Memorandum 9 – Planning Alternative Ranking and Evaluation (TM-9). Life cycle costs were calculated for all near-term and long-term planning alternatives being evaluated in TM-9. The results of the life cycle cost calculations will be used in TM-9 for alternative evaluation and to determine the optimum near-term and long-term alternatives to be implemented during the planning period.

METHODS

The life cycle cost is the total discounted dollar cost over the 50 year planning period and includes the following four components listed below:

1. Initial Capital Costs – It includes the purchase of land, buildings, equipment, and construction activities to complete all the component projects associated with a fully operable status.

   In this analysis, the initial capital costs include the construction costs for wastewater treatment plants, pumping stations, wastewater interceptors, effluent force mains, etc.

2. Annual Operation/Maintenance Costs – It includes all the expenses including labor, materials, and other expenses for maintaining day-to-day facility functions and preserving the operating efficiency and physical condition of the facilities included in a planning alternative.

   In this analysis the annual operating costs include pumping costs for conveying the flow to and from the treatment facilities and operational costs for high quality effluent treatment facilities. The unit treatment costs of high quality effluent facilities in the NSWTP will be lower than the stand-alone plants due to the economy of scale. The unit treatment costs for stand-along plants and high quality effluent facility at the NSWTP were determined to be $450/mgd and $300/mgd, respectively. The annual maintenance costs are assumed to be 0.5% of the capital costs.

3. 50-Year Facility Improvement & Replacement Costs – It includes all of the costs associated with the replacement of the structures, equipment, and other major components of the facilities included in a planning alternative to maintain the proper operation efficiency and physical conditions of the facilities during the 50 year planning period.

   In this analysis, it is assumed that equipment will be replaced every 20 years, structures every 50 years, and interceptors and force mains every 75 years.

4. Salvage value – The remaining value of the new facilities included in the planning alternatives at the end of the 50-year planning period.

The following assumptions have been made in the planning alternative calculations:

1. A discount rate of 3% over the 50 year planning period was used in life cycle cost calculations for all planning alternatives.
2. The following unit electricity costs were used in life cycle cost calculations.
   - PS11: $0.109 / kwHr
   - PS12: $0.105 / kwHr
   - PS17: $0.102 / kwHr
   - NSWTP: $0.074 / kwHr
   - Stand-alone remote sites: $0.10 kwHr
3. The expected useful service life is expected to be 20 years for equipment, 50 years for structures, and 75 years for force mains and interceptors.
4. The cost for a MMSD full time staff equivalent including benefits is $80,000 per year.
5. The unit construction cost for standalone high quality effluent WWTP is $10 per gpd of treatment capacity for the plant with 4 mgd capacity. Considering the economy of scale, a unit construction price of $8 per gpd of treatment capacity was used for the treatment plant with 10 mgd capacity.
6. Unit construction costs for high quality effluent treatment facilities at the NSWTP will be lower than stand-alone treatment plants since the new facilities will only treat effluent of the existing secondary treatment processes in lieu of raw wastewater. The unit construction cost of high quality effluent treatment process at the NSWTP is $4 per gpd of treatment capacity for the 4 mgd plant. Considering the economy of scale, unit construction cost for 10 mgd plant was determined to be $3 per gpd of treatment capacity.

The piping unit price used for capital costs are shown in the following table.

Piping Unit Cost

Pipe Diameter	Interceptor Cost (per LF)	Force Main Cost (per LF)	NSVI Lining Cost (per LF)
18"	$275	$175	-
21"	$300	$200	-
24"	$450	$250	-
27"	$475	-	-
30"	$500	$325	$125
33"	-	-	$150
36"	$600	$400	$175
42"	$700	$500	$200
48"	$800	-	$225
54"	$950	-	$250
60"	$1,100	-	-

The detailed planning alternative life cycle cost calculation spreadsheets for all master planning alternatives are attached in the following order:

Table A-0: Summary of Life Cycle Cost Analysis

Table A-1: Alternative MP-1A: Westside Conveyance System Expansion (current operation condition)

Table A-2: Alternative MP-1B: Westside Conveyance System Expansion (with increased discharge to the Badger Mill Creek)

Table A-3: Alternative MP-1C: Westside Conveyance System Expansion (with discharge to downstream of confluence of the Badger Mill Creek and the Sugar River)

Table A-4: Alternative MP-1D: Westside Conveyance System Expansion (with discharges to downstream of the confluence of the Badger Mill Creek and the Sugar River, and the Sugar River headwaters near CTH PD)

Table A-5: Alternative MP-2A: Sugar River WWTP (with discharge to downstream of the confluence of the Badger Mill Creek and the Sugar River)

Table A-6: Alternative MP-2B: Sugar River WWTP (with discharges to downstream of confluence of the Badger Mill Creek and the Sugar River, and the Sugar River headwaters near CTH PD)

Table A-7: Alternative MP-3A: Centralized High Quality Effluent Treatment & Distribution (4 mgd daily average treatment capacity)

Table A-8: Alternative MP-3B: Centralized High Quality Effluent Treatment Facilities (10 mgd daily average treatment capacity)

Table A-9: Alternative MP-4A: Decentralized High Quality Effluent Treatment & Distribution (4 mgd daily average treatment capacity)

Table A-10: Alternative MP-4B: Decentralized High Quality Effluent Treatment Facilities (10 mgd daily average treatment capacity)

Table A-11: Conceptual Cost Estimate for NSVI Relining

Table A-12: Conceptual Planning Alternative Operating Costs

TABLE A-0
SUMMARY OF CONCEPTUAL LIFE CYCLE COST ANALYSIS
MMSD PROJECT NO. 8425001

Planning Alternative	Initial Capital Cost	Life Cycle O/M Cost	Life Cycle Improvement & Replacement Cost	Life Cycle Salvage Value	50-Year Life Cycle Cost
Near-Term Planning Alternatives					
MP-1A	$50,881,000	$18,881,000	$2,758,000	($3,298,000)	$69,222,000
MP-1B	$68,581,000	$30,843,000	$7,312,000	($3,702,000)	$103,034,000
MP-1C	$75,068,000	$32,901,000	$7,698,000	($4,115,000)	$111,552,000
MP-1D	$75,068,000	$33,029,000	$7,698,000	($4,115,000)	$111,680,000
MP-2A	$67,905,000	$32,407,000	$14,028,000	($2,261,000)	$112,079,000
MP-2B	$72,305,000	$34,036,000	$14,414,000	($2,516,000)	$118,239,000
Long-Term Planning Alternatives					
MP-3A	$27,100,000	$20,024,000	$4,631,000	($1,103,000)	$50,652,000
MP-3B	$45,500,000	$47,949,000	$8,362,000	($1,730,000)	$100,081,000
MP-4A	$40,000,000	$26,167,000	$10,292,000	($912,000)	$75,547,000
MP-4B	$80,000,000	$56,670,000	$20,584,000	($1,825,000)	$155,429,000

TABLE A-1
CONCEPTUAL LIFE CYCLE COST ANALYSIS FOR ALTERNATIVE MP-1A
MMSD PROJECT NO. 8425001

NO.	ITEM DESCRIPTION	UNITS	UNIT COST	QUANTITY	TOTAL COST	NOTES
	INIITIAL CAPITAL COST					
1	PS 17 Firm Pumping Capacity Expansion	LS	$2,200,000	1	$2,200,000	Refer to TM-3, Table 4.02-1
2	PS 17 Force Main Expansion	LS	$2,952,000	1	$2,952,000	Refer to TM-3, Table 4.03-1
3	NSVI Expansion (PS 11 to PS 12)	LS	$25,526,000	1	$25,526,000	Refer to TM-3, Table 4.04-1
4	NSVI Expansion (Upstream of PS12)	LS	$2,817,000	1	$2,817,000	Refer to TM-3, Table 4.04-1
5	PS 11 Force Main	LS	$1,050,000	1	$1,050,000	Refer to TM-3, Table 4.03-1
6	Relining of the NSVI	LS	$7,815,500	1	$7,815,500	Refer to Table A-11
7	PS 11 Capacity and Condition Improvement	LS	$4,260,000	1	$4,260,000	Refer to TM-3, Table 4.02-1
8	PS 12 Capacity and Condition Improvement	LS	$4,260,000	1	$4,260,000	Refer to TM-3, Table 4.02-1
	TOTAL CAPITAL COST				**$50,881,000**	
	OPERATION & MAINTAINENCE COST					
1	Annual Operating Cost	LS	$479,413	1	$479,413	Refer to Table A-12
2	Annual Maintenance Cost	LS	$254,405	1	$254,405	
	Subtotal				**$733,818**	
	Present Worth Factor				**25.73**	
	TOTAL LIFE CYCLE O/M COST				**$18,881,000**	
	FACILITY IMPROVEMENT & REPLACEMENT COST					
1	Annual Facility Improvement & Replacement Cost	LS	$107,200	1	$107,200	Equipment
	Subtotal				**$107,200**	
	Present Worth Factor				**25.73**	
	TOTAL LIFE CYCLE IMPROVEMENT & REPLACEMENT COST				**$2,758,000**	
	SALVAGE VALUE					
1	Equipment	LS	$1,072,000	1	$1,072,000	
2	Force Main & Interceptors	LS	$13,386,833	1	$13,386,833	
3	Structure	LS	$0	1	$0	
	Subtotal				**$14,458,833**	
	Present Worth Factor				**0.23**	
	TOTAL LIFE CYCLE SALVAGE VALUE				**$3,298,000**	
	TOTAL LIFE CYCLE COST					
	Project Life (Year)		50			
	WDNR Discount Rate		3.0%			
	TOTAL LIFE CYCLE COST				**$69,222,000**	

TABLE A-2
CONCEPTUAL LIFE CYCLE COST ANALYSIS FOR ALTERNATIVE MP-1B
MMSD PROJECT NO. 8425001

NO.	ITEM DESCRIPTION	UNITS	UNIT COST	QUANTITY	TOTAL COST	NOTES
	INIITIAL CAPITAL COST					
1	PS 17 Firm Pumping Capacity Expansion	LS	$2,200,000	1	$2,200,000	Refer to TM-3, Table 4.02-1
2	PS 17 Force Main Expansion	LS	$2,952,000	1	$2,952,000	Refer to TM-3, Table 4.03-1
3	NSVI Expansion (PS 11 to PS 12)	LS	$25,526,000	1	$25,526,000	Refer to TM-3, Table 4.04-1
4	NSVI Expansion (Upstream of PS12)	LS	$2,817,000	1	$2,817,000	Refer to TM-3, Table 4.04-1
5	PS 11 Force Main	LS	$1,050,000	1	$1,050,000	Refer to TM-3, Table 4.03-1
6	Relining of the NSVI	LS	$7,815,500	1	$7,815,500	Refer to Table A-11
7	PS 11 Capacity and Condition Improvement	LS	$4,260,000	1	$4,260,000	Refer to TM-3, Table 4.02-1
8	PS 12 Capacity and Condition Improvement	LS	$4,260,000	1	$4,260,000	Refer to TM-3, Table 4.02-1
9	Badger Mill Creek Effluent Pump Capacity Expansion	LS	$500,000	1	$500,000	
10	High Quality Effluent Treatment Process at NSWTP	LS	$17,200,000	1	$17,200,000	
	TOTAL CAPITAL COST				**$68,581,000**	
	OPERATION & MAINTAINENCE COST					
1	Annual Operating Cost	LS	$855,810	1	$855,810	Refer to Table A-12
2	Annual Maintenance Cost	LS	$342,905	1	$342,905	
	Subtotal				**$1,198,715**	
	Present Worth Factor				**25.73**	
	TOTAL LIFE CYCLE O/M COST				**$30,843,000**	
	FACILITY IMPROVEMENT & REPLACEMENT COST					
1	Annual Facility Improvement & Replacement Cost	LS	$284,200	1	$284,200	Equipment
	Subtotal				**$284,200**	
	Present Worth Factor				**25.73**	
	TOTAL LIFE CYCLE IMPROVEMENT & REPLACEMENT COST				**$7,312,000**	
	SALVAGE VALUE					
1	Equipment	LS	$2,842,000	1	$2,842,000	
2	Force Main & Interceptors	LS	$13,386,833	1	$13,386,833	
3	Structure	LS	$0	1	$0	
	Subtotal				**$16,228,833**	
	Present Worth Factor				**0.23**	
	TOTAL SALVAGE VALUE				**$3,702,000**	
	TOTAL LIFE CYCLE COST					
	Project Life (Year)		50			
	WDNR Discount Rate		3.0%			
	TOTAL LIFE CYCLE COST				**$103,034,000**	

TABLE A-3
CONCEPTUAL LIFE CYCLE COST ANALYSIS FOR ALTERNATIVE MP-1C
MMSD PROJECT NO. 8425001

NO.	ITEM DESCRIPTION	UNITS	UNIT COST	QUANTITY	TOTAL COST	NOTES
	INITIAL CAPITAL COST					
1	PS 17 Firm Pumping Capacity Expansion	LS	$2,200,000	1	$2,200,000	Refer to TM-3, Table 4.02-1
2	PS 17 Force Main Expansion	LS	$2,952,000	1	$2,952,000	Refer to TM-3, Table 4.03-1
3	NSVI Expansion (PS 11 to PS 12)	LS	$25,526,000	1	$25,526,000	Refer to TM-3, Table 4.04-1
4	NSVI Expansion (Upstream of PS12)	LS	$2,817,000	1	$2,817,000	Refer to TM-3, Table 4.04-1
5	PS 11 Force Main	LS	$1,050,000	1	$1,050,000	Refer to TM-3, Table 4.03-1
6	Relining of the NSVI	LS	$7,815,500	1	$7,815,500	Refer to Table A-11
7	PS 11 Capacity and Condition Improvement	LS	$4,260,000	1	$4,260,000	Refer to TM-3, Table 4.02-1
8	PS 12 Capacity and Condition Improvement	LS	$4,260,000	1	$4,260,000	Refer to TM-3, Table 4.02-1
9	Sugar River Effluent Pumping Station	LS	$1,500,000	1	$1,500,000	
10	Sugar River Effluent Pumping Station Force Main	LF	$175	28,500	$4,987,500	
11	Badger Mill Creek Effluent Pump Capacity Expansion	LS	$500,000	1	$500,000	
12	High Quality Effluent Treatment Process at NSWTP	LS	$17,200,000	1	$17,200,000	
	TOTAL CAPITAL COST				**$75,068,000**	
	OPERATION & MAINTAINENCE COST					
1	Annual Operating Cost	LS	$903,360	1	$903,360	Refer to Table A-12
2	Annual Maintenance Cost	LS	$375,340	1	$375,340	
	Subtotal				**$1,278,700**	
	Present Worth Factor				**25.73**	
	TOTAL LIFE CYCLE O/M COST				**$32,901,000**	
	FACILITY IMPROVEMENT & REPLACEMENT COST					
1	Annual Facility Improvement & Replacement Cost	LS	$299,200	1	$299,200	Equipment
	Subtotal				**$299,200**	
	Present Worth Factor				**25.73**	
	TOTAL LIFE CYCLE IMPROVEMENT & REPLACEMENT COST				**$7,698,000**	
	SALVAGE VALUE					
1	Equipment	LS	$2,992,000	1	$2,992,000	
2	Force Main & Interceptors	LS	$15,049,333	1	$15,049,333	
3	Structure	LS	$0	1	$0	
	Subtotal				**$18,041,333**	
	Present Worth Factor				**0.23**	
	TOTAL LIFE CYCLE SALVAGE VALUE				**$4,115,000**	
	TOTAL LIFE CYCLE COST					
	Project Life (Year)		50			
	WDNR Discount Rate		3.0%			
	TOTAL LIFE CYCLE COST				**$111,552,000**	

TABLE A-4
CONCEPTUAL LIFE CYCLE COST ANALYSIS FOR ALTERNATIVE MP-1D
MMSD PROJECT NO. 8425001

NO.	ITEM DESCRIPTION	UNITS	UNIT COST	QUANTITY	TOTAL COST	NOTES
	INIITIAL CAPITAL COST					
1	PS 17 Firm Pumping Capacity Expansion	LS	$2,200,000	1	$2,200,000	Refer to TM-3, Table 4.02-1
2	PS 17 Force Main Expansion	LS	$2,952,000	1	$2,952,000	Refer to TM-3, Table 4.03-1
3	NSVI Expansion (PS 11 to PS 12)	LS	$25,526,000	1	$25,526,000	Refer to TM-3, Table 4.04-1
4	NSVI Expansion (Upstream of PS12)	LS	$2,817,000	1	$2,817,000	Refer to TM-3, Table 4.04-1
5	PS 11 Force Main	LS	$1,050,000	1	$1,050,000	Refer to TM-3, Table 4.03-1
6	Relining of the NSVI	LS	$7,815,500	1	$7,815,500	Refer to Table A-11
7	PS 11 Capacity and Condition Improvement	LS	$4,260,000	1	$4,260,000	Refer to TM-3, Table 4.02-1
8	PS 12 Capacity and Condition Improvement	LS	$4,260,000	1	$4,260,000	Refer to TM-3, Table 4.02-1
9	Sugar River Effluent Pumping Station	LS	$1,500,000	1	$1,500,000	
10	Sugar River Effluent Pumping Station Force Main	LF	$175	28,500	$4,987,500	
11	Badger Mill Creek Effluent Pump Capacity Expansion	LS	$500,000	1	$500,000	
12	High Quality Effluent Treatment Process at NSWTP	LS	$17,200,000	1	$17,200,000	
	TOTAL CAPITAL COST				**$75,068,000**	
	OPERATION & MAINTAINENCE COST					
1	Annual Operating Cost	LS	$908,348	1	$908,348	Refer to Table A-12
2	Annual Maintenance Cost	LS	$375,340	1	$375,340	
	Subtotal				**$1,283,688**	
	Present Worth Factor				**25.73**	
	TOTAL LIFE CYCLE O/M COST				**$33,029,000**	
	FACILITY IMPROVEMENT & REPLACEMENT COST					
1	Annual Facility Improvement & Replacement Cost	LS	$299,200	1	$299,200	Equipment
	Subtotal				**$299,200**	
	Present Worth Factor				**25.73**	
	TOTAL LIFE CYCLE IMPROVEMENT & REPLACEMENT COST				**$7,698,000**	
	SALVAGE VALUE					
1	Equipment	LS	$2,992,000	1	$2,992,000	
2	Force Main & Interceptors	LS	$15,049,333	1	$15,049,333	
3	Structure	LS	$0	1	$0	
	Subtotal				**$18,041,333**	
	Present Worth Factor				**0.23**	
	TOTAL LIFE CYCLE SALVAGE VALUE				**$4,115,000**	
	TOTAL LIFE CYCLE COST					
	Project Life (Year)		50			
	WDNR Discount Rate		3.0%			
	TOTAL LIFE CYCLE COST				**$111,680,000**	

TABLE A-5
CONCEPTUAL LIFE CYCLE COST ANALYSIS FOR ALTERNATIVE MP-2A
MMSD PROJECT NO. 8425001

NO.	ITEM DESCRIPTION	UNITS	UNIT COST	QUANTITY	TOTAL COST	NOTES
	INIITIAL CAPITAL COST					
1	Sugar River WWTP	LS	$43,000,000	1	$43,000,000	
2	PS 17 Firm Capacity Expansion	LS	$3,000,000	1	$3,000,000	
3	PS 17 Forcemain	LS	$3,225,000	1	$3,225,000	
4	Relining of the NSVI	LS	$10,160,150	1	$10,160,150	Including by-pass pumping cost
5	PS 11 Capacity and Condition Improvement	LS	$4,260,000	1	$4,260,000	Refer to TM-3, Table 4.02-1
6	PS 12 Capacity and Condition Improvement	LS	$4,260,000	1	$4,260,000	Refer to TM-3, Table 4.02-1
	TOTAL CAPITAL COST				**$67,905,000**	
	OPERATION & MAINTAINENCE COST					
1	Annual Operating Cost	LS	$919,996	1	$919,996	Refer to Table A-12
2	Annual Maintenance Cost	LS	$339,525	1	$339,525	
	Subtotal				**$1,259,521**	
	Present Worth Factor				**25.73**	
	TOTAL LIFE CYCLE O/M COST				**$32,407,000**	
	FACILITY IMPROVEMENT & REPLACEMENT COST					
1	Annual Facility Improvement & Replacement Cost	LS	$545,200	1	$545,200	Equipment
	Subtotal				**$545,200**	
	Present Worth Factor				**25.73**	
	TOTAL LIFE CYCLE IMPROVEMENT & REPLACEMENT COST				**$14,028,000**	
	SALVAGE VALUE					
1	Equipment	LS	$5,452,000	1	$5,452,000	
2	Force Main & Interceptors	LS	$4,461,717	1	$4,461,717	
3	Structure	LS	$0	1	$0	
	Subtotal				**$9,913,717**	
	Present Worth Factor				**0.23**	
	TOTAL LIFE CYCLE SALVAGE VALUE				**$2,261,000**	
	TOTAL LIFE CYCLE COST					
	Project Life (Year)		50			
	WDNR Discount Rate		3.0%			
	TOTAL LIFE CYCLE COST				**$112,079,000**	

TABLE A-6
CONCEPTUAL LIFE CYCLE COST ANALYSIS FOR ALTERNATIVE MP-2B
MMSD PROJECT NO. 8425001

NO.	ITEM DESCRIPTION	UNITS	UNIT COST	QUANTITY	TOTAL COST	NOTES
	INITIAL CAPITAL COST					
1	Sugar River WWTP	LS	$43,000,000	1	$43,000,000	
2	PS 17 Firm Capacity Expansion	LS	$3,000,000	1	$3,000,000	
3	PS 17 Forcemain	LS	$3,225,000	1	$3,225,000	
4	Relining of the NSVI	LS	$10,160,150	1	$10,160,150	Including by-pass pumping cost
5	Sugar River WWTP Effluent Pumping Station	LS	$1,500,000	1	$1,500,000	
6	Sugar River WWTP Effluent Force Main	LS	$2,900,000	1	$2,900,000	
7	PS 11 Capacity and Condition Improvement	LS	$4,260,000	1	$4,260,000	Refer to TM-3, Table 4.02-1
8	PS 12 Capacity and Condition Improvement	LS	$4,260,000	1	$4,260,000	Refer to TM-3, Table 4.02-1
	TOTAL CAPITAL COST				**$72,305,000**	
	OPERATION & MAINTAINENCE COST					
1	Annual Operating Cost	LS	$961,294	1	$961,294	Refer to Table A-12
2	Annual Maintenance Cost	LS	$361,525	1	$361,525	
	Subtotal				**$1,322,819**	
	Present Worth Factor				**25.73**	
	TOTAL LIFE CYCLE O/M COST				**$34,036,000**	
	FACILITY IMPROVEMENT & REPLACEMENT COST					
1	Annual Facility Improvement & Replacement Cost	LS	$560,200	1	$560,200	Equipment
	Subtotal				**$560,200**	
	Present Worth Factor				**25.73**	
	TOTAL LIFE CYCLE IMPROVEMENT & REPLACEMENT COST				**$14,414,000**	
	SALVAGE VALUE					
1	Equipment	LS	$5,602,000	1	$5,602,000	
2	Force Main & Interceptors	LS	$5,428,383	1	$5,428,383	
3	Structure	LS	$0	1	$0	
	Subtotal				**$11,030,383**	
	Present Worth Factor				**0.23**	
	TOTAL LIFE CYCLE SALVAGE VALUE				**$2,516,000**	
	TOTAL LIFE CYCLE COST					
	Project Life (Year)		50			
	WDNR Discount Rate		3.0%			
	TOTAL LIFE CYCLE COST				**$118,239,000**	

TABLE A-7
CONCEPTUAL LIFE CYCLE COST ANALYSIS FOR ALTERNATIVE MP-3A
MMSD PROJECT NO. 8425001

NO.	ITEM DESCRIPTION	UNITS	UNIT COST	QUANTITY	TOTAL COST	NOTES
	INITIAL CAPITAL COST					
1	NSWTP High Quality Effluent Treatment Facilities	LS	$16,000,000	1	$16,000,000	
2	NSWTP High Quality Effluent Pumping Station	LS	$2,000,000	1	$2,000,000	4 mgd capacity
3	NSWTP High Quality Effluent Return Force Main	LF	$175	52000	$9,100,000	18" Pipe
	TOTAL CAPITAL COST				**$27,100,000**	
	OPERATION & MAINTAINENCE COST					
1	Annual Operating Cost	LS	$642,750	1	$642,750	Refer to Table A-12
2	Annual Maintenance Cost	LS	$135,500	1	$135,500	
	Subtotal				**$778,250**	
	Present Worth Factor				**25.73**	
	TOTAL LIFE CYCLE O/M COST				**$20,024,000**	
	FACILITY IMPROVEMENT & REPLACEMENT COST					
1	Annual Facility Improvement & Replacement Cost	LS	$180,000	1	$180,000	Equipment
	Subtotal				**$180,000**	
	Present Worth Factor				**25.73**	
	TOTAL LIFE CYCLE IMPROVEMENT & REPLACEMENT COST				**$4,631,000**	
	SALVAGE VALUE					
1	Equipment	LS	$1,800,000	1	$1,800,000	
2	Force Main & Interceptors	LS	$3,033,333	1	$3,033,333	
3	Structure	LS	$0	1	$0	
	Subtotal				**$4,833,333**	
	Present Worth Factor				**0.23**	
	TOTAL LIFE CYCLE SALVAGE VALUE				**$1,103,000**	
	TOTAL LIFE CYCLE COST					
	Project Life (Year)		50			
	WDNR Discount Rate		3.0%			
	TOTAL LIFE CYCLE COST				**$50,652,000**	

TABLE A-8
CONCEPTUAL LIFE CYCLE COST ANALYSIS FOR ALTERNATIVE MP-3B
MMSD PROJECT NO. 8425001

NO.	ITEM DESCRIPTION	UNITS	UNIT COST	QUANTITY	TOTAL COST	NOTES
	INITIAL CAPITAL COST					
1	NSWTP High Quality Effluent Treatment Facilities	LS	$30,000,000	1	$30,000,000	
2	NSWTP High Quality Effluent Pumping Station	LS	$2,500,000	1	$2,500,000	10 mgd capacity
3	NSWTP High Quality Effluent Return Force Main	LF	$250	52000	$13,000,000	24" Pipe
	TOTAL CAPITAL COST				**$45,500,000**	
	OPERATION & MAINTAINENCE COST					
1	Annual Operating Cost	LS	$1,636,076	1	$1,636,076	Refer to Table A-12
2	Annual Maintenance Cost	LS	$227,500	1	$227,500	
	Subtotal				**$1,863,576**	
	Present Worth Factor				**25.73**	
	TOTAL LIFE CYCLE O/M COST				**$47,949,000**	
	FACILITY IMPROVEMENT & REPLACEMENT COST					
1	Annual Facility Improvement & Replacement Cost	LS	$325,000	1	$325,000	Equipment
	Subtotal				**$325,000**	
	Present Worth Factor				**25.73**	
	TOTAL LIFE CYCLE IMPROVEMENT & REPLACEMENT COST				**$8,362,000**	
	SALVAGE VALUE					
1	Equipment	LS	$3,250,000	1	$3,250,000	
2	Force Main & Interceptors	LS	$4,333,333	1	$4,333,333	
3	Structure	LS	$0	1	$0	
	Subtotal				**$7,583,333**	
	Present Worth Factor				**0.23**	
	TOTAL LIFE CYCLE SALVAGE VALUE				**$1,730,000**	
	TOTAL LIFE CYCLE COST					
	Project Life (Year)		50			
	WDNR Discount Rate		3.0%			
	TOTAL LIFE CYCLE COST				**$100,081,000**	

TABLE A-9
CONCEPTUAL LIFE CYCLE COST ANALYSIS FOR ALTERNATIVE MP-4A
MMSD PROJECT NO. 8425001

NO.	ITEM DESCRIPTION	UNITS	UNIT COST	QUANTITY	TOTAL COST	NOTES
	INITIAL CAPITAL COST					
1	Starkweather Creek WWTP	LS	$40,000,000	1	$40,000,000	
	TOTAL CAPITAL COST				**$40,000,000**	
	OPERATION & MAINTAINENCE COST					
1	Annual Operating Cost	LS	$817,000	1	$817,000	Refer to Table A-12
2	Annual Maintenance Cost	LS	$200,000	1	$200,000	
	Subtotal				**$1,017,000**	
	Present Worth Factor				**25.73**	
	TOTAL LIFE CYCLE O/M COST				**$26,167,000**	
	FACILITY IMPROVEMENT & REPLACEMENT COST					
1	Annual Facility Improvement & Replacement Cost	LS	$400,000	1	$400,000	Equipment
	Subtotal				**$400,000**	
	Present Worth Factor				**25.73**	
	TOTAL LIFE CYCLE IMPROVEMENT & REPLACEMENT COST				**$10,292,000**	
	SALVAGE VALUE					
1	Equipment	LS	$4,000,000	1	$4,000,000	
2	Force Main & Interceptors	LS	$0	1	$0	
3	Structure	LS	$0	1	$0	
	Subtotal				**$4,000,000**	
	Present Worth Factor				**0.23**	
	TOTAL LIFE CYCLE SALVAGE VALUE				**$912,000**	
	TOTAL LIFE CYCLE COST					
	Project Life (Year)		50			
	WDNR Discount Rate		3.0%			
	TOTAL LIFE CYCLE COST				**$75,547,000**	

TABLE A-10
CONCEPTUAL LIFE CYCLE COST ANALYSIS FOR ALTERNATIVE MP-4B
MMSD PROJECT NO. 8425001

NO.	ITEM DESCRIPTION	UNITS	UNIT COST	QUANTITY	TOTAL COST	NOTES
	INIITIAL CAPITAL COST					
1	Starkweather Creek WWTP	LS	$80,000,000	1	$80,000,000	
	TOTAL CAPITAL COST				**$80,000,000**	
	OPERATION & MAINTAINENCE COST					
1	Annual Operating Cost	LS	$1,802,500	1	$1,802,500	Refer to Table A-12
2	Annual Maintenance Cost	LS	$400,000	1	$400,000	
	Subtotal				**$2,202,500**	
	Present Worth Factor				**25.73**	
	TOTAL LIFE CYCLE O/M COST				**$56,670,000**	
	FACILITY IMPROVEMENT & REPLACEMENT COST					
1	Annual Facility Improvement & Replacement Cost	LS	$800,000	1	$800,000	Equipment
	Subtotal				**$800,000**	
	Present Worth Factor				**25.73**	
	TOTAL LIFE CYCLE IMPROVEMENT & REPLACEMENT COST				**$20,584,000**	
	SALVAGE VALUE					
1	Equipment	LS	$8,000,000	1	$8,000,000	
2	Force Main & Interceptors	LS	$0	1	$0	
3	Structure	LS	$0	1	$0	
	Subtotal				**$8,000,000**	
	Present Worth Factor				**0.23**	
	TOTAL LIFE CYCLE SALVAGE VALUE				**$1,825,000**	
	TOTAL LIFE CYCLE COST					
	Project Life (Year)		50			
	WDNR Discount Rate		3.0%			
	TOTAL LIFE CYCLE COST				**$155,429,000**	

TABLE A-11
CONCEPTUAL COST ESTIMATE FOR NSVI RELINING
MMSD PROJECT NO. 8425001

Pipe Section	Start	End	Pipe Length	Pipe Diameter	Unit Cost	Cost
			(LF)	(inch)		
11Aii	11-171	11-169	812	42	$200	$162,400
11Aiii	11-169	11-167	465	42	$200	$93,000
11Aiv	11-167	111-161E	1,436	42	$200	$287,200
11Avi	11-161A	11-159	1,321	36	$175	$231,175
11Bi	11-159	11-158	340	36	$175	$59,500
11Biii	11-156	11-151A	2,220	42	$200	$444,000
11C	11-151A	11-145	3,784	42	$200	$756,800
11Di	11-145	11-141	3,784	36	$175	$662,200
11Dii	11-141	11-137	1,648	30	$125	$206,000
11Diii	11-137	11-129	3,995	33	$150	$599,250
11Div	11-129	11-127	733	36	$175	$128,275
11Dv	11-127	11-116A	4,855	54	$250	$1,213,750
11Fi	11-116A	11-111A	2,788	54	$250	$697,000
11Fii	11-111A	11-106A	2,716	54	$250	$679,000
11Fiii	11-106A	11-104	1,689	54	$250	$422,250
11Fiv	11-104	PS11	1,525	54	$250	$381,250
12Hi	12-110	12-101	3,484	48	$225	$783,900
12Hii	12-101	PS 12	38	48	$225	$8,550
Total			**37,633**			**$7,815,500**

TABLE A-12
CONCEPTUAL PLANNING ALTERNATIVE OPERATING COSTS
MMSD PROJECT NO. 8425001

Alternative MP-1A

No.	Item	Unit	Quantity (per year)	Unit Price	Annual Total Cost
1	PS11 Pumping	million gallon	1,559	$27.9	$43,421
2	PS 12 Pumping	million gallon	1,559	$23.8	$37,016
3	PS 17 Pumping	million gallon	1,559	$83.4	$129,999
4	Badfish Creek Pumping	million gallon	1,559	$27.5	$42,922
5	Conventional Treatment @ NSWTP	million gallon	1,559	$145.0	$226,055
	Total				**$479,413**

Alternative MP-1B

No.	Item	Unit	Quantity (per year)	Unit Price	Annual Total Cost
1	PS11 Pumping	million gallon	1,559	$27.9	$43,421
2	PS 12 Pumping	million gallon	1,559	$23.8	$37,016
3	PS 17 Pumping	million gallon	1,559	$83.4	$129,999
4	Badger Mill Creek Pumping	million gallon	1,559	$114.0	$177,675
5	High Quality Effluent Facility	million gallon	1,559	$300.0	$467,700
	Total				**$855,810**

Alternative MP-1C

No.	Item	Unit	Quantity (per year)	Unit Price	Annual Total Cost
1	PS11 Pumping	million gallon	1,559	$27.9	$43,421
2	PS 12 Pumping	million gallon	1,559	$23.8	$37,016
3	PS 17 Pumping	million gallon	1,559	$83.4	$129,999
4	Badger Mill Creek Pumping	million gallon	1,559	$114.0	$177,675
5	Sugar River Pumping	million gallon	1,559	$30.5	$47,550
6	High Quality Effluent Facility	million gallon	1,559	$300.0	$467,700
	Total				**$903,360**

Alternative MP-1D

No.	Item	Unit	Quantity (per year)	Unit Price	Annual Total Cost
1	PS11 Pumping	million gallon	1,559	$27.9	$43,421
2	PS 12 Pumping	million gallon	1,559	$23.8	$37,016
3	PS 17 Pumping	million gallon	1,559	$83.4	$129,999
4	Badger Mill Creek Pumping	million gallon	1,559	$114.0	$177,675
5	Sugar River Headwaters Pumping	million gallon	1,559	$33.7	$52,538
6	High Quality Effluent Facility	million gallon	1,559	$300.0	$467,700
	Total				**$908,348**

Alternative MP-2A

No.	Item	Unit	Quantity (per year)	Unit Price	Annual Total Cost
1	PS 17 Pumping	million gallon	1,559	$37.5	$58,446
2	Plant Operator	staff equivalent	2	$80,000	$160,000
3	High Quality Effluent Facility	million gallon	1,559	$450	$701,550
	Total				**$919,996**

Alternative MP-2B

No.	Item	Unit	Quantity (per year)	Unit Price	Annual Total Cost
1	PS 17 Pumping	million gallon	1,559	$37.5	$58,446
2	Sugar River Headwaters Pumping	million gallon	547	$75.5	$41,299
3	Plant Operator	staff equivalent	2	$80,000	$160,000
4	High Quality Effluent Facility	million gallon	1,559	$450	$701,550
	Total				**$961,294**

Alternative MP-3A

No.	Item	Unit	Quantity (per year)	Unit Price	Annual Total Cost
1	PS 13 Pumping	million gallon	1,460	$10.5	$15,374
3	PS 10 Pumping	million gallon	1,460	$38.1	$55,553
4	PS 7 Pumping	million gallon	1,460	$23.4	$34,106
5	High Quality Effluent Pumping	million gallon	1,460	$68.3	$99,718
6	High Quality Effluent Facility	million gallon	1,460	$300.0	$438,000
	Total				**$642,750**

Alternative MP-3B

No.	Item	Unit	Quantity (per year)	Unit Price	Annual Total Cost
1	PS 13 Pumping	million gallon	3,650	$10.5	$38,435
3	PS 10 Pumping	million gallon	3,650	$38.1	$138,883
4	PS 7 Pumping	million gallon	3,650	$23.4	$85,264
5	High Quality Effluent Pumping	million gallon	3,650	$76.3	$278,495
6	High Quality Effluent Facility	million gallon	3,650	$300.0	$1,095,000
	Total				**$1,636,076**

Alternative MP-4A

No.	Item	Unit	Quantity (per year)	Unit Price	Annual Total Cost
1	Plant Operator	staff equivalent	2	$80,000	$160,000
2	High Quality Effluent Facility	million gallon	1,460	$450	$657,000
	Total				**$817,000**

Alternative MP-4B

No.	Item	Unit	Quantity (per year)	Unit Price	Annual Total Cost
1	Plant Operator	staff equivalent	2	$80,000	$160,000
2	High Quality Effluent Facility	million gallon	3,650	$450	$1,642,500
	Total				**$1,802,500**

Appendix B

Master Planning Alternative Carbon Footprint Calculations

Master Planning Alternative Carbon Footprint Calculations

BACKGROUND

This master planning alternative carbon footprint calculation was prepared as part of the Master Plan Technical Memorandum 9 – Planning Alternative Ranking and Evaluation (TM-9). Life cycle costs were calculated for all near-term and long-term planning alternatives being evaluated in TM-9. The results of the carbon footprint calculations will be used in TM-9 for master planning alternative evaluation and to determine the optimum near-term and long-term alternatives to be implemented during the planning period.

METHODS

Estimates of annual greenhouse gas and several criteria air pollutant emissions due to the production of electricity consumed off the grid were performed using emission factors obtained from the United Stated Environmental Protection Agency's (EPA) Emissions & Generation Resource Integrated Database (eGRID) 2007 (version 1.1). The eGRID system is designed to aggregate electric production and greenhouse gas emission data from the individual power production unit level to the regional and nationwide level. Emission factors used for this project were based on the Midwest Reliability Organization East (MROE) eGRID subregion.

The carbon dioxide (CO_2), methane (CH_4), and nitrous oxide (N_2O) emission factors were converted to equivalent carbon dioxide (CO2e) emission factors using global warming potential (GWP) factors obtained from Table A-1 of the proposed EPA greenhouse gas reporting rule (Federal Register Docket ID No. EPA-HQ-OAR-2008-0508). The total greenhouse gas emission factor is equal to the sum of the products of the individual greenhouse gas emission factors and corresponding GWP factors, as described by the following equation:

$$EF_{CO2e} = \Sigma^{i} (EF_i \times GWP_i)$$

where:

EF_{CO2e} = total greenhouse gas emission factor (lb CO_2e per MWh)
EF_i = individual greenhouse gas emission factor for compound i (lb of i per MWh)
GWP_i = global warming potential of compound i (lb of CO_2e per lb of i)

Total greenhouse gas emissions were estimated as the product of the total greenhouse gas emission factor and the estimated electrical consumption from the grid for each planning altertive. Estimated air emissions of oxides of nitrogen (NOx), sulfur dioxide (SO2), and mercury (Hg) were also provided and based on the MROE eGRID subregion emission factors.

A summary of the planning alternative carbon footprint calculations for all master planning alternatives are attached in the following order:

1. **Alternative MP-1A:** Westside Conveyance System Expansion (current operation condition)

2. **Alternative MP-1B:** Westside Conveyance System Expansion (with increased discharge to the Badger Mill Creek)

3. **Alternative MP-1C:** Westside Conveyance System Expansion (with discharge to downstream of confluence of the Badger Mill Creek and the Sugar River)

4. **Alternative MP-1D:** Westside Conveyance System Expansion (with discharges to downstream of the confluence of the Badger Mill Creek and the Sugar River, and the Sugar River headwaters near CTH PD)

5. **Alternative MP-2A:** Sugar River WWTP (with discharge to downstream of the confluence of the Badger Mill Creek and the Sugar River)

6. **Alternative MP-2B:** Sugar River WWTP (with discharges to downstream of confluence of the Badger Mill Creek and the Sugar River, and the Sugar River headwaters near CTH PD)

7. **Alternative MP-3A:** Centralized High Quality Effluent Treatment & Distribution (4 mgd daily average treatment capacity)

8. **Alternative MP-3B:** Decentralized High Quality Effluent Treatment Facilities (4 mgd daily average treatment capacity)

9. **Alternative MP-4A:** Centralized High Quality Effluent Treatment & Distribution (10 mgd daily average treatment capacity)

10. **Alternative MP-4B:** Decentralized High Quality Effluent Treatment Facilities (10 mgd daily average treatment capacity)

Table 1. Estimated Annual Air Emission Rates

Scenario ID	Estimated Annual Electric Consumption		Estimated Annual Emission Rates (tons/year)						
	kwhr/yr	MWhr/yr	NO_x	SO_2	Hg	CO_2	CH_4	N_2O	CO_2e
MP-1A	7,263,706	7,264	10.0	26.0	1.0E-04	6,663	1.0E-01	1.1E-01	6,700
Pumping	4,222,633	4,223	5.8	15.1	5.8E-05	3,874	5.8E-02	6.4E-02	3,895
Treatment	3,041,073	3,041	4.2	10.9	4.2E-05	2,790	4.2E-02	4.6E-02	2,805
MP-1B	10,260,679	10,261	14.1	36.8	1.4E-04	9,413	1.4E-01	1.6E-01	9,464
Pumping	6,468,517	6,469	8.9	23.2	8.9E-05	5,934	8.9E-02	9.8E-02	5,966
Treatment	3,792,162	3,792	5.2	13.6	5.2E-05	3,479	5.2E-02	5.8E-02	3,498
MP-1C	11,053,179	11,053	15.2	39.6	1.5E-04	10,140	1.5E-01	1.7E-01	10,195
Pumping	7,261,017	7,261	10.0	26.0	9.9E-05	6,661	1.0E-01	1.1E-01	6,697
Treatment	3,792,162	3,792	5.2	13.6	5.2E-05	3,479	5.2E-02	5.8E-02	3,498
MP-1D	11,136,312	11,136	15.3	39.9	1.5E-04	10,216	1.5E-01	1.7E-01	10,272
Pumping	7,344,150	7,344	10.1	26.3	1.0E-04	6,737	1.0E-01	1.1E-01	6,774
Treatment	3,792,162	3,792	5.2	13.6	5.2E-05	3,479	5.2E-02	5.8E-02	3,498
MP-2A	5,884,950	5,885	8.1	21.1	8.1E-05	5,399	8.1E-02	8.9E-02	5,428
Pumping	974,100	974	1.3	3.5	1.3E-05	894	1.3E-02	1.5E-02	898
Treatment	4,910,850	4,911	6.7	17.6	6.7E-05	4,505	6.8E-02	7.5E-02	4,530
MP-2B	6,573,267	6,573	9.0	23.6	9.0E-05	6,030	9.1E-02	1.0E-01	6,063
Pumping	1,662,417	1,662	2.3	6.0	2.3E-05	1,525	2.3E-02	2.5E-02	1,533
Treatment	4,910,850	4,911	6.7	17.6	6.7E-05	4,505	6.8E-02	7.5E-02	4,530
MP-3A	7,792,500	7,793	10.7	27.9	1.1E-04	7,149	1.1E-01	1.2E-01	7,187
MP-3B	19,367,933	19,368	26.6	69.4	2.7E-04	17,767	2.7E-01	2.9E-01	17,864
MP-4A	6,750,000	6,750	9.3	24.2	9.2E-05	6,192	9.3E-02	1.0E-01	6,226
MP-4B	16,425,000	16,425	22.6	58.9	2.3E-04	15,068	2.3E-01	2.5E-01	15,150

Table 2. Emission Factors[(a)]

Pollutant	lb/MWh	GWP[(b)] (CO_2e lb/lb)
Criteria Pollutants		
NO_x	2.7473	0
SO_2	7.1664	0
Hg	2.74E-05	0
Greenhouse Gases		
CO_2	1834.72	1
CH_4	0.02759	21
N_2O	0.03036	310
CO_2e[(c)]	1845	1

Notes:

a) Emission Factors were obtained from eGRID 2007 v1.1 and based on the annual output from the MROE Subregion.

b) The global warming potential (GWP) factors were obtained from the proposed EPA mandatory greenhouse gas reporting rule (Federal Register Docket ID No. EPA-HQ-OAR-2008-0508) Table A-1

c) The equivalent CO_2 (CO_2e) emission factor was calculated based on eGRID factors for CO_2, CH_4, and N_2O and GWP factors from the proposed EPA mandatory greenhouse gas reporting rule as noted above.

Appendix C

Hydraulic Capacity Calculations for the Existing Badger Mill Force Main

MALCOLM PIRNIE

Hydraulic Capacity Calculations for the Existing Badger Mill Creek Force Main

BACKGROUND

This hydraulic capacity calculations for the existing Badger Mill Creek force main calculation was prepared as part of the Master Plan Technical Memorandum 9 – Planning Alternative Ranking and Evaluation (TM-9). The results of this analysis will be used for evaluating the feasibility of increasing treated effluent return to the Badger Mill Creek outfall through the existing force main. Currently, MMSD return approximately 3.7 mgd of treated effluent to the Badger Mill Creek with two centrifugal effluent return pumps located at the NSWTP. The effluent pumps and force main information are listed as follows:

Number of Pumps:	2
Type of Pumps:	Centrifugal
Speed:	Variable
Capacity:	2,000 gpm @ 190 psi

Total Force Main Length:	53,970 feet
"Pressured" Force Main Length:	41,580 feet (from pump discharge to force main highest point)
"Forced Gravity" Force Main Length:	12,390 feet (from force main highest point to outfall)
Force Main Diameter:	20 inch
Force Main Material:	Cement-lined ductile iron, Class 250

Force Main Centerline Elevation at the Highest Point	990.8	feet
Effluent Pump Centerline Elevation	842.9	feet
Min Water Level above Pump Center Line	8.5	feet
Force Main Centerline Elevation at the Outfall	950.0	feet
Min Water Level at the Outfall	965.5	feet

CALCULATIONS

The force main consists of two sections. The first section is a pressured force main and the second one is a forced gravity flow pipe. Some of the key parameters for these two sections are listed in Table C-1.

Table C-1 – Summary of the Force Main Sections

Piping Section No.	Piping Section	Length (feet)	Diameter (inches)	Delta H (feet)	Slope
1	From pump discharge to the force main high point	41,580	20	139.4	0.0034
2	From force main high point to the outfall	12,390	20	-25.3	-0.002

Under low flow conditions, the Section 2 could be gravity flow. Therefore Manning's equation was used to determine the maximum gravity flow capacity of the section.

$$V = (\frac{1.49}{n}) R^{2/3} \sqrt{S}$$

$$Q = VA = (\frac{1.49}{n}) A R^{2/3} \sqrt{S}$$

V = flow velocity; feet per second
n = Manning roughness coefficient; dimensionless (0.013 was used)
R = hydraulic radius, feet
S = slope, dimensionless
Q = flow rate; gpm
A = pipe section area; square feet

The calculation results indicate that the full gravity flow capacity of the section 2 is 4.03 mgd with a corresponding flow velocity of 2.86 feet per second. Therefore when flow rate is higher than 4.03 mgd, the pipe section will be pressurized. To fully realize the capacity of the existing force main, Hazen-William equation was used to calculate the headloss in the piping system at higher flowrates.

$$H_L = \frac{(10.44)(L)Q^{1.85}}{C^{1.85} d^{4.8655}}$$

H_L = friction head loss; feet of water
L = length of pipe; feet
Q = flow rate; gpm
C = Hazen-Williams roughness factor; dimensionless
d = pipe diameter; inches

The headloss at different flow rates and Hazen-Williams roughness factors are shown in Figure C-1 and Table C-2.

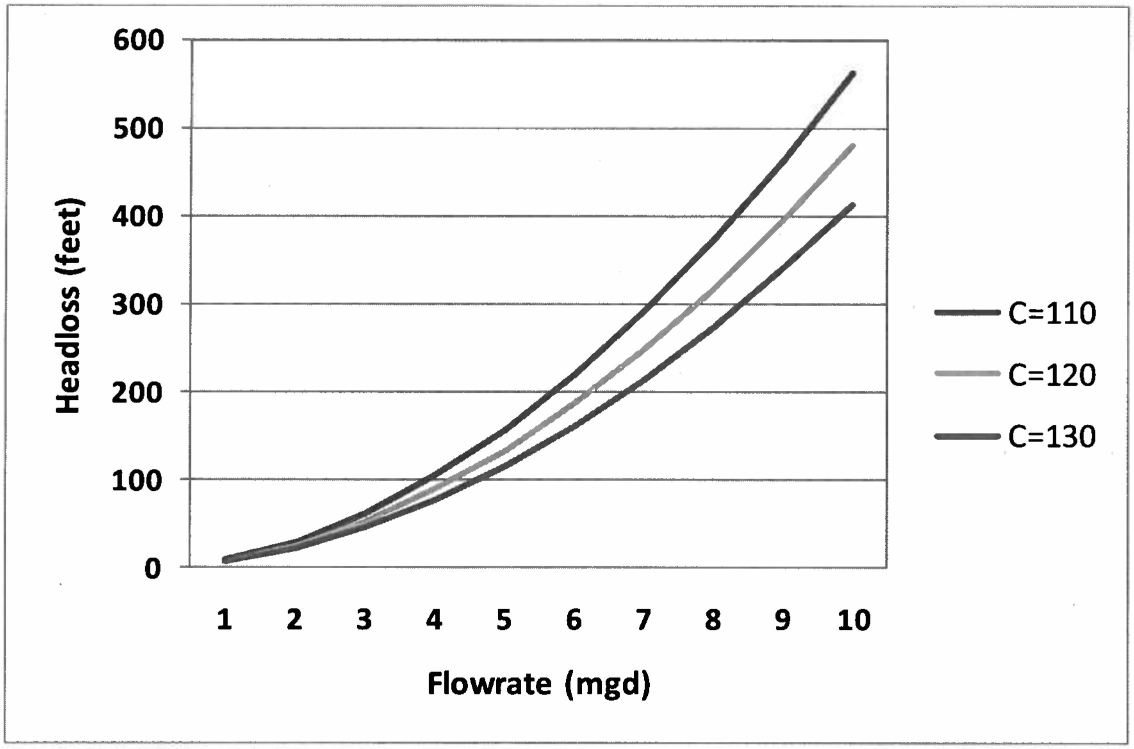

Figure C-1 – Headloss of the Existing Badger Mill Creek Force Main

Table C-2 – Headloss of the Existing Badger Mill Creek Force Main

Flow (mgd)	Headloss					
	C=110		C=120		C=130	
	Feet	PSI	Feet	PSI	Feet	PSI
1	7.96	3.45	6.78	2.94	5.85	2.53
2	28.71	12.43	24.44	10.58	21.08	9.12
3	60.79	26.31	51.75	22.40	44.63	19.32
4	103.50	44.81	88.11	38.14	75.99	32.89
5	156.40	67.71	133.15	57.64	114.82	49.71
6	219.14	94.87	186.56	80.76	160.88	69.64
7	291.45	126.17	248.12	107.41	213.97	92.63
8	373.13	161.53	317.65	137.51	273.93	118.58
9	463.97	200.85	394.98	170.99	340.62	147.45
10	563.82	244.08	479.99	207.79	413.92	179.19

According to MMSD's experience and previous test results, the existing force main has a C value of approximately 130. At a flow rate of 8 mgd, the headloss in the force main is 274 feet or 119 psi. Combined with the total static head, the maximum pressure in the force main at 8 mgd will be 413 feet or 179 psi.

The existing force main has a pressure rating of Class 250, which is defined as the rated working pressure of the pipe. Typically the pipe will allow for the rated water working pressure plus a surge allowance of 100 psi. However this needs to be confirmed by the original piping manufacturer. Several of the high points on the existing force main are open to the atmosphere. If the entire length of the force main is operating under pressure, modification will be required at these high points, including enclosing the pipe and adding air-release valves.

The calculation results show that the existing force main has sufficient capacity to transport 8.0 mgd of the treated effluent to the Badger Mill Creek outfall. However, the existing effluent return pumps need to be replaced with pumps with larger capacities.