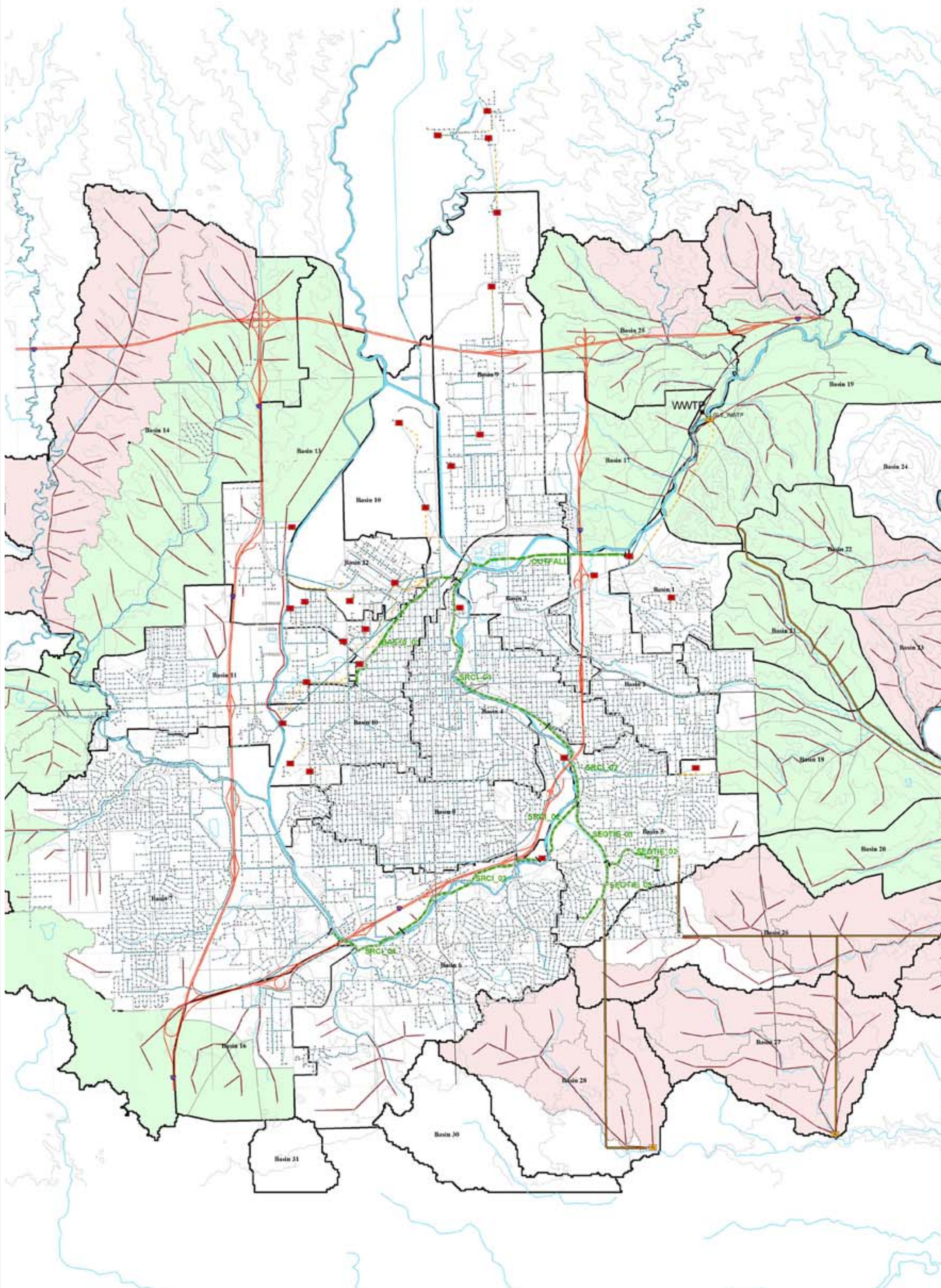
F I N A L   R E P O R T

Sanitary Sewer Collection System Facilities Plan



PREPARED FOR

SEAL OF THE CITY OF SIOUX FALLS
SOUTH DAKOTA

BLACK & VEATCH

Howard R. Green Company

2002

Project No. 66571
City CIP No. 068077

BLACK & VEATCH

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Black & Veatch Corporation

September 4, 2002

Mark Perry, P.E.
City Engineer
City of Sioux Falls
224 West 9th Street
Sioux Falls, South Dakota 57014

Subject: Final Report – Sanitary Sewer Collection System Facilities Plan

Dear Mr. Perry:

Enclosed for the City's distribution are twenty (20) copies of the final report for the Sanitary Sewer Collection System Facilities Plan.

The report provides long range planning of the sanitary sewer collection system facilities for the next 25 years. The report presents population and land use planning, the results of flow monitoring and flow projections, system inventory, trunk sewer system modeling, and a phased capital improvements plan.

We have enjoyed working with the City of Sioux Falls, yourself, and other members of the City staff during the preparation of this report. Please let us know if there is anything else needed for this project. We look forward to continued opportunities for Black & Veatch and the City of Sioux Falls to work together.

Sincerely,

BLACK & VEATCH CORPORATION

Chad Hill by M.C.M.

Chad Hill, Project Manager

Jonathan P. Gray

Jonathan P. Gray, Project Manager
Infrastructure Planning Department

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Acknowledgments

Black & Veatch gratefully acknowledges the cooperation and assistance of many people in the execution of this project. The names of the people who participated in the study and who were instrumental in executing the project are presented below.

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Rick McKee	Wastewater Superintendent
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Hoard R. Green Co. – Local Support/Flow Monitoring

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Cecil Coombs	Technical Manager

In addition we would like to thank the support staff of the City of Sioux Falls, South Dakota and Black & Veatch Corporation.

Executive Summary

Executive Summary

The City of Sioux Falls (City) authorized Black & Veatch to develop a Sanitary Sewer Collection System Facilities Plan through the Year 2025. This report presents the results of the Facilities Plan and recommends a program of system improvements.

1. Project Background and Scope

The project Study Area consists of approximately 80,585 acres; 37,365 acres are within the Sioux Falls city limits. Facilities Plan preparation included a review of land use, population, and wastewater production; update of the collection system geographic information system (GIS); a review of existing wastewater system facilities, development of a trunk sewer model, a hydraulic analysis of the existing collection system, alternative plans for growth area development, and a recommended capital improvements plan. Following evaluation of the collection system for hydraulic capacity, Black & Veatch developed recommendations for:

- Priority I, Priority II, and Priority III infrastructure improvements.
- Pipeline replacement and rehabilitation.
- Long range Capital Improvement Program.

2. Population and Land Use

The current and future land use was provided by the City's planning department. Current land use in the Study Area is largely residential (about 78 percent). Other uses include commercial, light industrial, and municipal. The Study Area will be about 91 percent developed by Year 2025. Additional growth will come from undeveloped areas in the West and East. No additional future land use categories were considered in the evaluations.

The base year (year 2000) Study Area population is about 124,000. The medium-range population growth of 1.8 percent per annum was assumed for population projections through year 2025. The Study Area year 2025 population is projected to be 185,000.

3. Wastewater Flows

Flow and rainfall was monitored to estimate average daily dry weather flow, average annual daily flow, and to quantify the collection system response to rain events. The flow and rainfall monitoring began in August 2001 and was completed in September 2001. Flow was monitored at seven key locations throughout the system. Rainfall was monitored at four locations. Flow data was also collected at twelve permanent meters at lift stations and the wastewater treatment facility.

The rainfall monitoring data was supplemented with remote sensed rainfall data obtained from NEXRAIN. NEXRAIN supplies gauge-adjusted radar rainfall intensity in greater detail than is economically possible with rain gauges alone. The rainfall data was analyzed to determine the return frequency of the measured storm events.

Peak wet weather flow was projected based on estimates of the unit infiltration rate, peak dry weather flow, and inflow. The 25-year storm event was selected as the design rainfall. Peak flows during the 25-year storm event were estimated for each basin based on analysis of the flow and rainfall monitoring data and used for planning level design. Estimates of flows for other conditions were made for model calibration and timing of project phasing.

4. Existing Wastewater System Facilities

Components of the existing system include:

- Drainage Basins
- Sewer Inventory

The drainage basin is a convenient unit of analysis that defines the areas within which sanitary flow is collected by gravity. Subbasins are the smallest unit used in this analysis and represent hydraulically homogeneous areas. The City supplied basin and subbasin definitions for the existing sanitary sewer service area and for the projected growth areas. The existing sewer service area includes 42,000 acres.

The sewer inventory includes the following:

- Manhole and Sewer Pipe Information
- 16 Modeled Flow Diversions
- 32 Modeled Pumping Stations
- 32 Modeled Force Mains
- 6 Modeled Siphons

- 1 Modeled Flow Equalization Facility
- 1 Wastewater Treatment Facility

The inventory of the existing system was compiled from the following sources:

- City's GIS
- City records and as-built drawings
- Previous reports

The completed system inventory, as imported to Black & Veatch's SSMS, contained 11,777 gravity mains and 27 force mains. A total of 2,559 line segments representing a total length of 144 miles of the trunk sewer composed of 10 inch diameter and greater pipes were selected for modeling from the total imported. Details of the diversions, pump stations, force mains, siphons and equalization basin were entered into the XP-SWMM model.

5. Trunk Sewer Model Development

The City's sanitary trunk sewer system was evaluated using computer modeling to simulate flows under a variety of conditions. SSMS is Black & Veatch's standard proprietary system for inspection, maintenance management, collection system inventory management, and pre- and post-processing model interface for three modeling packages, including XP-SWMM.

XP-SWMM is a fully dynamic model package developed by XP Software. Hydraulic calculations estimate the flows through links (pipes, diversions, pump stations) and nodes (manholes, wet wells, storage basins). Hydrologic scenarios permit evaluation of the performance of the collection system under different storm conditions.

The hydraulic model was calibrated by comparing model flow outputs at dry-weather conditions and wet-weather to measurements and estimates. Adjustments were made to the inflow parameters to match the expected model results.

Once the model produced output in agreement with expected results, the model was used to generate hydraulic capacity analyses for three projected development conditions (2015, 2025, Build-out) and the existing conditions. Hydraulic capacity analysis included three weather conditions (average dry-weather, 1-year storm, 25-year storm).

The model results indicated that the existing system is sufficient for average dry-weather conditions through year 2025. Depending on the development year, the 1-year storm event caused flows in 140 to 180 line segments that are greater than 100 percent of the existing trunk sewer capacity. The 25-year storm event caused from 900 to 1,010 line segments to have flows greater than 100 percent of the existing trunk sewer capacity. The line segments with year 2025/25-year storm flows greater than 100 percent of existing capacity were identified as candidates for relief projects. These projects were reviewed and prioritized in meetings with the City staff.

6. Growth Area Analysis

The western growth areas may be served by gravity to the existing system, including existing pump stations for serving the lower areas to the West. The eastern areas of current development are bounded by a ridge beyond which sanitary sewers have not been built. Four alternative development plans for the eastern growth areas were prepared, discussed and compared. The alternatives are summarized as follows:

- Plan 1, pump over the eastern ridge to the existing gravity trunk sewers.
- Plan 2, pump directly to the existing wastewater treatment facility.
- Plan 3, pump flow from the eastern basins directly to the wastewater facility and pump the flow from the southeastern basins to the existing gravity trunk sewers.
- Plan 4, construct a new Southeast Wastewater Treatment Facility to serve the east side growth.

Plan 4 was evaluated as two sub-alternatives to compare costs of gravity versus pumping flow to the Southeast Wastewater Treatment Facility. Comparison of the alternatives showed that the costs are within the same order of magnitude, either Plan 4 being 10 percent less than the next best alternative, Plan 1. The construction of the new Southeast Wastewater Treatment Facility need not begin for 10 years. Economic and development conditions may change and should be re-evaluated before a decision in made.

The City also requested a plan for recovery of development costs. Four areas were defined for this analysis and the average cost of development per acre is listed in Table 1.

Table 1
East Side Growth Area Cost Recovery

Planning Area	Definition	Recoverable Capital Cost Per Acre ($)
Area 1	Subbasins 18, 20, 21, 22C, 22D, and 23	5,631
Area 2	Subbasin 26B	4,255
Area 3	Subbasins 17, 19 (except 19E and 19H), 25 (except 25F)	3,436
Area 4	Subbasins 26 (except 26B), 27, 28, 29	4,459

7. Capital Improvement Program

The recommended capital improvements program includes:

- Construction of replacement sewers to provide protection from the 25-year storm event.
- Infiltration/Inflow reduction program in the area of the Stock Yards.
- Construction of pump station and force main upgrades to accommodate future growth.
- Construction of new sanitary gravity sewer, pump stations, and force mains to serve future growth.
- Monitoring of existing sanitary sewers on the “Watch List”.

Sizing of pipes was based on preliminary alignments, modeled flows, and the existing slopes for sewer relief or the ground surface slope for proposed extensions. Relief sewer and proposed sewer extension locations are preliminary. The final locations will be defined following a detailed alignment survey performed under a sewer design contract. The final sewer size and slope should be based on the actual route as well as the flow estimates presented in this plan.

The recommended capital improvements, including relief sewers and future extensions, were grouped by priority as follows:

- Priority 1, improvements required to address immediate needs or near term deficiencies that can be implemented within the next 5 years.
- Priority 2, additional facilities or improvements required by year 2015.
- Priority 3, additional facilities or improvements required by year 2025.

- Watch List, pipes that are currently marginally overloaded by large storm events, should be inspected and/or monitored to detect the presence of defects.

Table 2 summarizes the projected capital improvement costs by priority.

Table 2
Implementation Plan Project Cost Summary

Improvements	Capital Cost Summary $
Priority 1 – 2003-2007	
Relief Sewers	17,112,000
Pumping Station	6,316,500
Force Mains	2,996,000
Basin 3 Inflow Reduction	100,000
Growth Area Extensions	31,858,500
Total Priority 1	58,383,000
Priority 2 – 2008-2015	
Relief Sewers	4,114,000
Pumping Station	1,256,000
Force Mains	0
Growth Area Extensions	34,914,000
Total Priority 2	40,284,500
Priority 3 – 2016-2025	
Relief Sewers	22,384,000
Pumping Station	7,281,000
Force Mains	4,360,500
Growth Area Extensions	41,830,500
Total Priority 3	75,856,500
Grand Total	**174,524,000**

(1) Assumed City cost for private sector inflow source removal program

Additional recommendations include the development of a sewer system management database which would bring together inspection data, flow data, rainfall data, and modeling.

1.0 Introduction

1.0 Introduction

1.1 Purpose of Study

This Sanitary Sewer Collection System Facilities Plan provides long range planning of the wastewater collection system facilities for the City of Sioux Falls to manage the anticipated growth for the next 25 years. The study was necessary to update and revise the Facilities Plan Wastewater Collection System dated February 1990. This study incorporated population and land use projections as presented in the Sioux Falls 2015 Growth Management Plan adopted by a resolution of the City Council on December 16, 1996 which had been previously adopted by the Minnehaha and Lincoln County Commissions.

This study is driven by a significant increase in population in the last decade and the concern for adequate facilities to best serve the City through planning year 2025. The plan focuses on evaluating the service area needs for interceptors, and pumping facilities for the wastewater collection system. Both present and future service area configurations were evaluated.

Spatially distributed population projections were developed for the 2015 and 2025 planning year scenarios and incorporated in each of the system evaluations. Service area boundaries were reviewed and assessed to identify those areas to be included in future collection scenarios.

The wastewater collection system was evaluated using a temporary flow monitoring program from July 31, 2001 to September 26, 2001. Key flow parameters (average daily dry weather flow, infiltration and inflow) developed from the data were used to project peak wet-weather capacity assessments utilizing the XP-SWMM dynamic hydraulic model.

The calibrated model for the wastewater collection system was used to perform the planning year evaluations. The results of these evaluations served as the basis for identifying major capital improvement programs (CIP) to meet the City's anticipated wastewater flows resulting from population growth and development for the years 2015 and 2025. Financial impact to each entity and financing options to pay for the proposed CIP projects were also evaluated.

1.2 Description of System

Sioux Falls serves a residential population of approximately 124,000 people and a number of industries.

The wastewater collection system infrastructure includes:

- Collection system totaling 594 miles ranging from 6 inches through 66 inches diameter.
- Twenty-seven wastewater pumping stations with 16 miles of associated force main.
- One treatment plant with a design capacity of nearly 19 million gallons per day.

Average wastewater daily flow for past three years was over 13.8 million gallons per day with the historical maximum month of 19.61 million gallons per day.

1.3 Scope of Analysis

The scope of services for the project was defined to meet Sioux Falls' CIP, and included the following:

- Provide projections of the service area population and impact on wastewater flows for the planning years from 2015 through 2025.
- Evaluate the City's wastewater collection facilities for system capacity and the need for new facilities or modifications to the existing system.
- Develop a long-range CIP that will meet existing and future requirements and provide for cost-effective system reliability.

1.4 Study Area

The Study Area for this report includes 28 primary basins which have been established by the City. Figure 2-1 shows thirty-one drainage basins considered to serve Sioux Falls. Basin 24 was not included in the study since the property will be developed and serviced by the City of Brandon. Basins 31 and 30 are not expected to develop within the planning period and were not included in this study.

1.5 Abbreviations

Abbreviations used in this report are as follows:

AD	Average Day
ADDF	average daily dry weather flow
ADF	average annual daily flow
CIP	Capital Improvement Plan
EPS	Extended Period Simulation
Fps	feet per second
ft	Feet
ft/day	Feet per day
gcd	Gallons per capita per day
GIS	Geographical information system
gpm	Gallons per minute
hp	Horsepower
ICI	Industrial/Commercial/Institutional
in	Inch
LS	Lift Station
MD	Maximum Day
MG	Million Gallons
mg/L	Milligram per Liter
mgd	Million gallons per day
MH	Maximum Hour
Min	minutes
MMAD	Maximum Month Average Day
psi	Pounds per square inch
SCADA	Supervisory control and data acquisition
TAZ	Traffic Analysis Zone
TDH	Total Dynamic Head
TOC	Total Organic Carbon
UTM	Universal Tranferse Mercader
WWP	wastewater production
WWTP	Wastewater Treatment Plant

2.0 Population and Land Use Planning

2.0 Population and Land Use Planning

This chapter discusses population and land use data that was used in calculating future domestic wastewater flows within the City's sanitary sewer collection system.

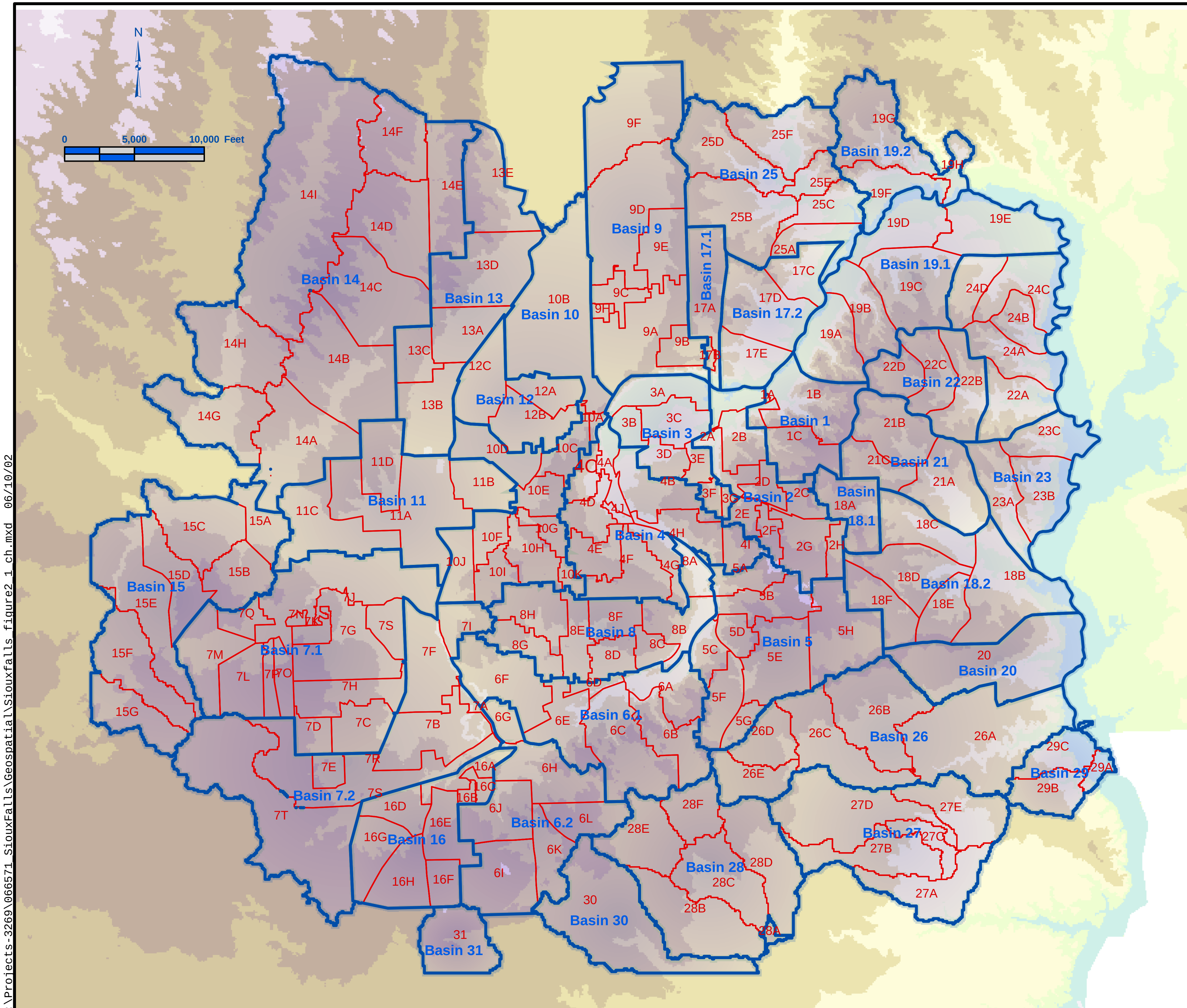
The year 2000 census population of Sioux Falls was 123,955. This represents an increase of more than 23,000 people compared to the 1990 census. According to the City's Growth Management Plan, this rate of population growth is expected to continue through the year 2025. The medium-range population projections assume an average population growth rate of about 1.8 percent per year for the foreseeable future. This equates to a 2015 population of 156,000 and a 2025 population of 185,000. Table 2-1 summarizes historical and projected population for the City of Sioux Falls.

Table 2-1
City of Sioux Falls Population

Year	Population
1970	72,488
1980	81,343
1990	100,814
2000	123,975
2015	156,000
2025	185,000

Data concerning current and future land use within the service area was provided by the City's planning department. Land use was broken down by use category and by traffic analysis zone (TAZ). There are approximately 260 traffic analysis zones within the wastewater master plan study area. Exhibit 2-1 shows the TAZ boundaries and the projected growth areas for years 2015 and 2025 as determined by the City's planning department. As can be seen, it is anticipated that growth will be taking place on all sides on the city, with the most significant development occurring to the east and southeast.

In order to facilitate the development of flow assignments for the wastewater collection system model, it is useful to calculate wastewater flows by drainage area basin. As shown on Figure 2-1, the wastewater service area can be divided into about 29 primary drainage basins, which can be further subdivided into approximately 150 sub-basins.



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Basin

Sub-Basin

Elevation Data (Feet)

- 1250 - 1300
- 1301 - 1350
- 1351 - 1400
- 1401 - 1450
- 1451 - 1500
- 1501 - 1550
- 1551 - 1600
- >1600
- No Data

Sioux Falls, South Dakota
Sanitary Sewer Collection System
Facilities Plan

Wastewater Service
Area Drainage Basins

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Figure 2-1

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By overlaying the drainage basin boundaries with the TAZ boundaries, it was possible to determine current and future land use within each drainage basin. Table 2-2 summarizes the population by primary drainage basin. The residential land use category shown in Table 2-3 includes single family homes, duplexes, townhouses, apartments, mobile homes, and dormitories. The industrial-commercial-institutional (ICI) category includes manufacturing, wholesale and retail stores, commercial and government offices, transportation facilities, utilities, schools, churches, hospitals, etc.

As can be seen from Table 2-3, basins 2, 3, 4, 8, 10, and 12 are expected to experience little or no growth during the study period. In general, these basins represent areas of the City that are essentially fully developed. It is therefore understandable that there would be relatively little growth projected within these basins. Conversely, it is anticipated that basins 6, 7, 14, 15, 16, and 18 through 29 will experience significant levels of development in the future because of their location on the periphery of the City. As was mentioned earlier, the areas that are expected to have the most significant levels of development are to the east and southeast of the City, and to lesser extents to the west and northwest.

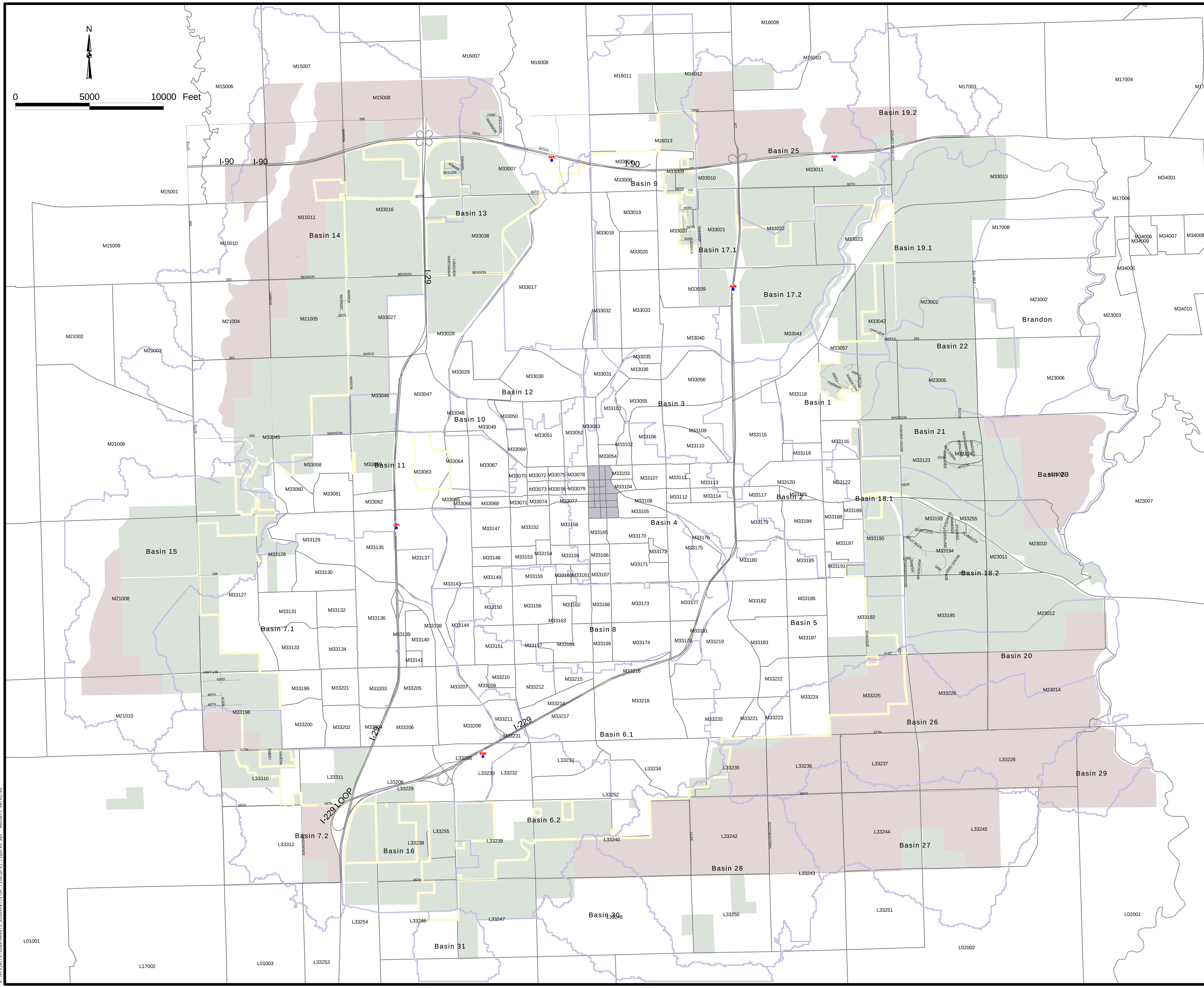
Planning Department supplied population projections through year 2025 and "ultimate build-out" developed area. The "ultimate build-out" represents an indefinite future development condition beyond year 2025 and includes alternative development scenarios. The study area includes all of the "ultimate build-out" therefore significant areas within the study area will not fully developed by year 2025. Development areas for year 2025 were based on assumptions of population densities and not the "ultimate build-out."

Table 2-2
Population by Drainage Basin

Basin No.	Year 2000	Year 2015	Year 2025
1	2,100	2,700	2,700
2	8,700	8,700	8,700
3	4,800	4,800	4,800
4	16,500	16,500	16,500
5	14,200	15,000	15,100
6	10,900	13,400	14,200
7	25,100	27,900	29,300
8	12,900	12,900	12,900
9	2,100	2,400	2,400
10	15,800	15,800	15,800
11	6,500	7,200	7,200
12	200	200	200
13	300	300	300
14	900	5,600	9,900
15	400	4,300	5,700
16	700	2,500	2,500
17	200	200	200
18	800	5,300	5,700
19	0	2,500	2,500
20	0	900	2,500
21	0	2,300	2,700
22	0	1,800	1,800
23	0	600	2,200
25	0	200	200
26	900	1,300	8,900
27	0	0	4,200
28	0	700	4,300
29	0	0	1,600
Total	**124,000**	**156,000**	**185,000**

Table 2-3
Land Use by Drainage Basin

Basin No.	Residential (acres)				ICI (acres)			
	2000	2015	2025	Ultimate	2000	2015	2025	Ultimate
1	250	440	440	440	370	380	380	380
2	540	540	540	630	440	440	440	500
3	310	310	310	310	400	400	400	400
4	800	800	800	800	760	760	760	760
5	1,190	1,490	1,530	1,530	300	370	370	370
6	1,270	2,190	2,470	2,470	1,090	1,180	1,200	1,200
7	1,710	2,770	3,250	3,670	1,190	1,370	1,370	1,750
8	860	860	860	860	490	490	490	490
9	290	390	390	740	1,080	1,280	1,550	1,900
10	750	750	750	750	1,900	1,910	1,910	1,910
11	490	720	720	720	1,100	1,300	1,300	1,300
12	10	10	10	10	580	580	580	580
13	20	20	20	110	510	1,580	1,660	1,720
14	100	1,860	3,330	4,150	100	1,930	3,190	3,600
15	60	1,490	1,970	2,470	20	130	130	320
16	120	760	760	810	60	430	430	470
17	30	30	30	60	840	1,390	1,390	1,450
18	100	1,780	1,910	1,960	20	520	560	570
19	0	890	890	1,290	180	1,230	1,380	1,600
20	0	330	900	900	0	20	30	30
21	0	850	1,000	1,000	40	160	250	250
22	0	650	650	650	0	40	40	40
23	0	230	790	820	0	0	0	10
25	0	90	90	230	110	800	1,500	1,570
26	100	270	2,900	2,900	20	80	600	600
27	0	0	1,480	1,780	0	10	320	400
28	0	280	1,530	1,760	0	10	40	100
29	0	0	580	580	0	10	30	30
Total	**9,000**	**20,800**	**30,900**	**34,400**	**11,600**	**18,800**	**22,300**	**24,300**



Sioux Falls, South Dakota
Sanitary Sewer Collection System
Facilities Plan

Growth Area & Traffic Area Zones

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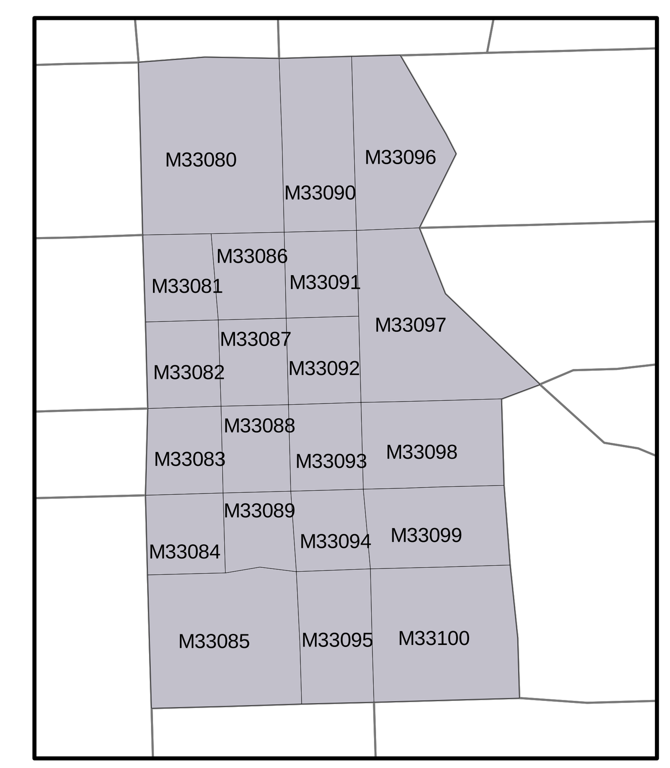
- Major Streets
- Basins
- City Limits
- 2000 Traffic Area Zones
- Downtown
- 2015 Growth Area
- 2025 Growth Area



Downtown Sioux Falls,
Traffic Area Zones

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Exhibit 2-1

3.0 Wastewater Flows

3.0 Wastewater Flows

3.1 Introduction

This chapter presents the flow and rainfall monitoring program, the monitoring program results, and the projected wastewater flows. Monitoring was performed to gather and analyze rainfall and wastewater flow, quantify ADDF, average annual daily flow, infiltration, and inflow at the temporary flow monitor locations, for use in projecting future flows throughout the Sioux Falls wastewater collection system.

3.2 Flow and Rainfall Monitoring Program

The Flow and Rainfall Monitoring Plan was submitted on July 9, 2001. Temporary rainfall gauges and open channel flow monitors were installed on July 30 and 31, 2001. Flow and rainfall monitoring was performed in August and September of 2001. A Flow Monitoring Update Memorandum was submitted on August 21, 2000, to provide the final locations for the monitors and initial results. The temporary rain gauges and open channel flow monitors were removed on September 25 and 26, 2001. Details of the program are presented in the following sections.

3.2.1 Monitor Locations

Seven temporary flow monitors and three temporary and one permanent ground-based rain gauges, along with NEXRAD radar measured rainfall collection, were used for recording flow and rainfall during the monitoring period. In addition, data recorded at 12 permanent flow monitors at lift stations and the wastewater treatment plant were collected.

3.2.1.1 Temporary Flow Monitors

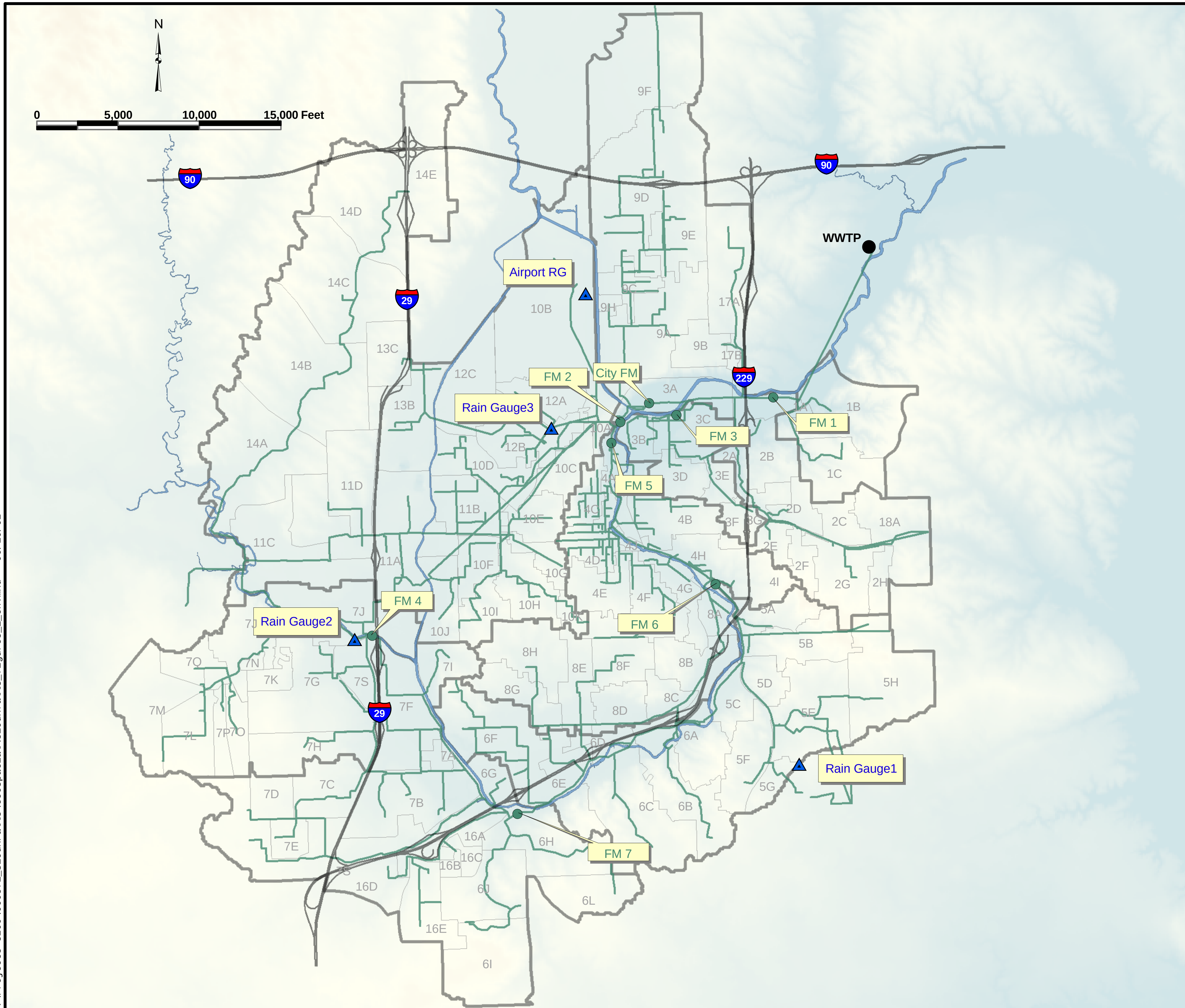
The Flow and Rainfall Monitoring Plan proposed at least two alternative manhole locations for each flow meter. At least one of the proposed locations for each meter was found to be acceptable in the field, except for FM2 and FM3. Hydraulic constraints in the pipe segments proposed for these monitors required alternative locations to obtain reliable flow measurements. All monitoring locations were selected in consultation with the City. The final temporary flow monitor locations are summarized in Table 3-1 and shown on Figure 3-1.

All of the temporary flow monitor sensors were mounted to record flow in pipes upstream of the designated manhole. The temporary flow monitors were maintained for eight (8) weeks to collect data during both dry weather and wet weather periods. Flow data was recorded at 15 minute intervals.

See Appendix D for table that defines the relationships among flow monitoring areas, basins, and subbasins.

Table 3-1
Temporary Flow Metering Sites

Flow Monitor	Location	Manhole Number	Sewer Size (in)	Meter Type
FM1	Outfall at Glenwood Avenue	02A0003	66	Flo-Dar
FM2	Riverside Pl., 4 MH Upstream of Diversion	03A0020	60	Flo-Dar
FM3	Sioux Nation	03C0003	37	Flo-Tote
FM4	Skunk Creek & West Water Reservoir	07J0001	24	Flo-Tote
FM5	North of Falls Park	04A0004	60	Flo-Dar
FM6	Cherry Rock Park	05A0001	41	Flo-Tote
FM7	57th & Western	06H0007	41	Flo-Tote



Legend

- Rain Gauge
- Flow Meter
- Major Highways
- Modeled Trunk Sewers
- Big Sioux River
- Drainage Sub-basins
- Flow Monitor Areas

Sioux Falls, South Dakota
Sanitary Sewer Collection System
Facilities Plan

Temporary Flowmeter Locations

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Figure 3-1

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3.2.1.2 Permanent, SCADA Flow Monitors

Permanent flow monitors are located at the wastewater treatment plant, the equalization basin, and several major lift stations. The flows and operational patterns observed at SCADA enabled facilities were reviewed and compared against flows at temporary flow monitor locations. This comparison is elaborated in the Flow Analysis section. The permanent flow monitors reviewed for this project are summarized in Table 3-2.

Table 3-2
SCADA Enabled Lift Stations

Station Number	Station Name	Address
203	Cherokee & "C"	Cherokee and C Avenue
204	Modern Press	806 N. West Avenue
206	Burnside	1800 Burnside
217	26th & Dubuque	5211 E. 26th St.
218	Tuthill Park	3500 S. Blauvelt
221	Madison & Vail	1116 N. Sycamore
224	50th Street North	50th Street North
225	40th Street North	210 E. 40th Street North
227	Highway 38A LS	201 Powderhouse Road
233	Renner #1	-

3.2.1.3 Ground-based Rainfall Gauges

The location, address, and UTM Zone 14 North (NAD 83) coordinates of each temporary ground-based rain gauge are summarized in Table 3-3. The temporary ground-based rain gauge locations are shown on Figure 3-1.

Table 3-3
Ground-based Rainfall Gauge Locations

Station Number	Location & Address	X_Coordinate (ft)	Y_Coordinate (ft)
RG1	Laurel Oaks Pool, 49th & Southeastern Avenue	2,254,825	15,812,777
RG2	West Water Reservoir, I-29 and Skunk Creek	2,227,479	15,820,454
RG3	Cherokee & “C” Lift Station, 1413 “C” Avenue	2,239,583	15,833,442
Airport RG	River Gauge North of Airport on Highway 38a	2,241,688	15,841,717

3.2.1.4 Radar-Measured Rainfall Collection

NEXRAD radar-generated rainfall data for the three weeks covering the largest observed storms was obtained for this analysis. Data was obtained within an area defined by the UTM Zone 14 North (NAD 83) coordinates specified in Table 3-4.

Table 3-4
Study Area Boundary Coordinates

Boundary Location	X_Coordinate (ft)	Y_Coordinate (ft)
Northwest Corner of Study Area	2,214,262	15,858,572
Southeast Corner of Study Area	2,282,118	15,799,052

The NEXRAD radar-generated rainfall data was adjusted, or calibrated, to match the ground-based rainfall gauge data. The adjusted NEXRAD rainfall data provided high-resolution rainfall measurement for the entire study area.

3.2.2 Flow and Rainfall Monitoring Equipment

3.2.2.1 Temporary Flow Monitoring

Flo-Dar and Flo-Tote temporary flow meters, manufactured by Marsh-McBirney, Inc., were used to measure open channel flow for this project. The Flo-Dar meters were used in the sites where the interceptor diameter is greater than 42 inches, and the Flo-Tote meters were used in the remaining sites.

Each monitoring unit includes sensors that measure depth of flow and velocity. The sensor type used by the two different types of meters is shown in Table 3-5. The Flo-Tote sensors were mounted in the wastewater flow on an expandable aluminum ring installed in the interceptor pipe, normally upstream of the manhole invert, as shown on Figure 3-2. The Flo-Dar Sensors were mounted to a bracket above the flow in the manhole. The signal from the sensors was sent through the communication cable to the monitor. The units operate on a battery power supply.

Table 3-5
Temporary Flow Meter Sensor Type

Flow Meter Type	Depth Sensor	Velocity Sensor
Flo-Dar	Pulsed Doppler profiling	Pulsed Doppler profiling
Flo-Tote	Pressure sensor	Electromagnetic field

The monitoring units were suspended from brackets mounted in the manhole wall near the top of each manhole and were set to collect and store depth of flow and velocity readings at 15-minute intervals. Data from the monitors was retrieved using a portable laptop computer.

3.2.2.2 Ground-based Rain Gauge Network

The gauges used for direct rainfall measurements were tipping-bucket type rainfall gauges with electronic recorders. The gauges continuously recorded each 0.01-inch depth of rainfall occurring during the monitoring period. The continuous data record was processed to define each rainfall event and determine the rainfall occurring over 15-minute intervals. The temporary rainfall gauges were serviced and the data retrieved weekly.

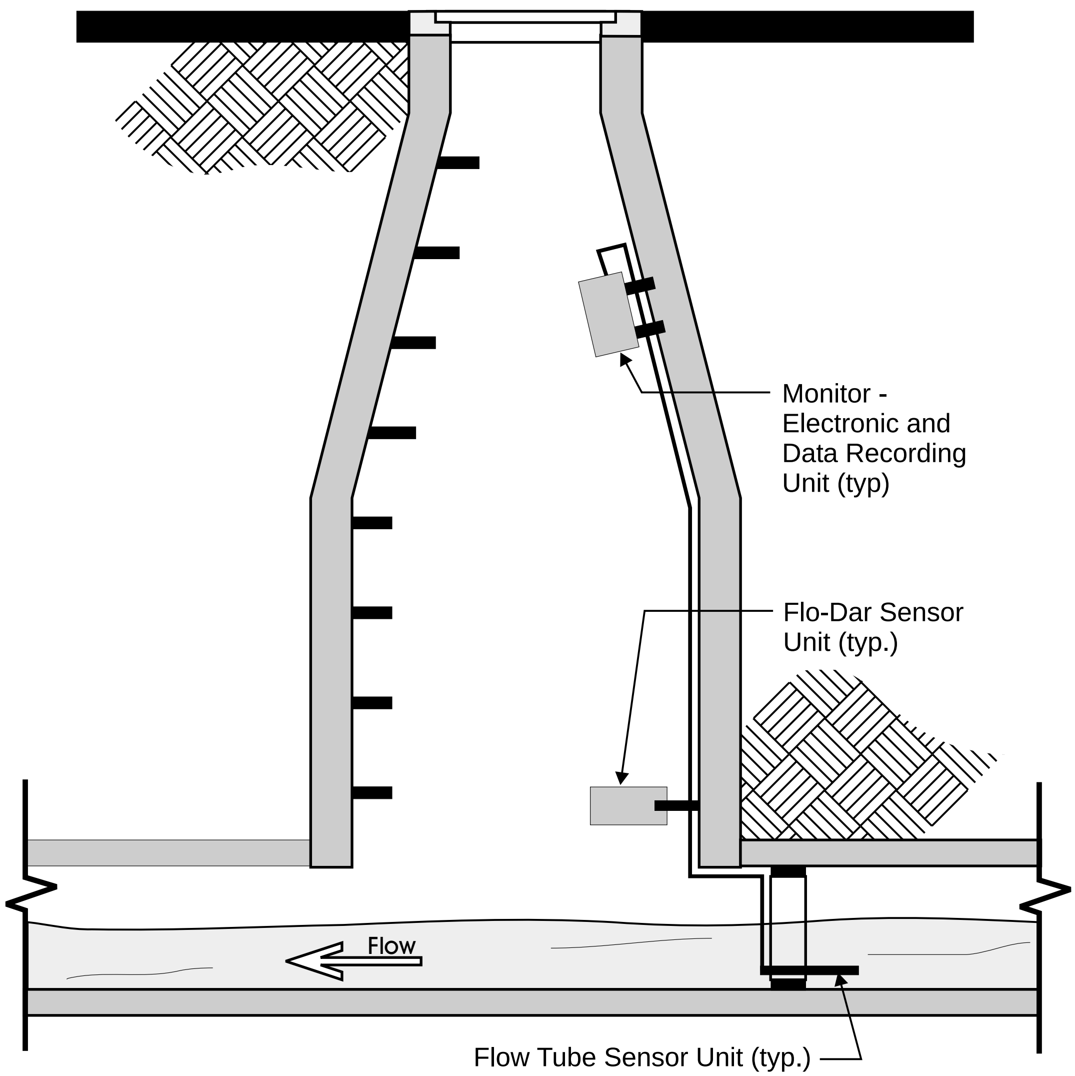
3.2.2.3 Remote Sensing Rainfall Collection

Ground-based rainfall data was supplemented with gauge-adjusted radar rainfall estimates. Radar rainfall data was obtained from NEXRAIN Corporation. The value of rainfall radar data is summarized as follows in the report sent by NEXRAIN at the time of data delivery.

> *The strength of a rain gauge network is its ability to consistently estimate rain falling on a number of discrete points. Its weakness is the network's inability to estimate rain falling between the gauges. On the other hand, radar's strength is its ability to see between the gauges but radar lacks the consistency in estimating rainfall at a specific point.*

The gauge-adjusted radar rainfall data used the data obtained at the ground-based rainfall gauges to calibrate data collected by the National Weather Service WSR-88D radar network.

Radar rainfall data gathering procedures, adjustment methodology, and calibration results are explained in detail in the NEXRAIN report. The report is attached as Appendix E of this Report.



Sioux Falls, SD
Wastewater Master Plan - 2001

Temporary Flow Meter Installation

Figure 3-2

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3.2.3 Monitoring Methodology

3.2.3.1 Pre-Installation Calibration

Each temporary flow monitor and rainfall gauge was checked for accuracy before installation and inspected once a week to check performance. A formal log of each performance check was recorded and filed.

3.2.3.2 Installation Procedures

After completion of the site investigations and monitor pre-installation calibration, the temporary flow monitors and rainfall gauges were installed. An inspection form for each temporary flow and rainfall monitoring site was completed. Each proposed temporary flow monitoring location was inspected for acceptable flow hydraulics as required for accurate flow recording. The site-specific hydraulic considerations that were reviewed before placement of temporary meters included:

- Uniformly shaped pipe.
- Smooth (laminar) flow away from the influence of flow entries or hydraulic jumps.
- Sufficient elevation differences to counter capacity problems that cause backup conditions.

3.2.3.3 Monitoring

During the monitoring, steps were taken to assure the integrity of the collected data. The quality of the field data was analyzed throughout the project. The performance checks performed during regular field visits to each flow monitor are described in the following sections.

3.2.3.3.1 Quality Assurance

The following performance checks were performed during regular field visits to each flow monitor:

- Download Data - The time, depth, and sensed velocity data accumulated in the monitor's memory were downloaded to a portable laptop computer on each site visit.
- Measure Power Supply - Power levels were recorded and batteries replaced, when necessary. A battery powers the monitor. A long life battery provided

back-up power to the memory, which allows the primary battery to be replaced without loss of data.

- Confirmation of Monitor Synchronization - The field crew checked the flow monitor's timing against the project master clock to ensure that all readings were taken simultaneously.

- Documentation of Field Condition - During the field checks, the field crew documented field conditions on daily field logs.

3.2.3.3.2 Flow Monitoring

The following reviews of the flow monitor locations and flow data were performed during the monitoring period:

- Verified Depth and Velocity of Flow - During the weekly site visits, manual measurements of the depth and velocity of flow in the invert were made from the ground surface. The manual measurements were compared to the monitor readings to check accuracy of the monitors.

- Measure Deposition Level - The depth of debris or sediment at the sensor was measured by the field crew.

3.2.3.3.3 Owner Assistance

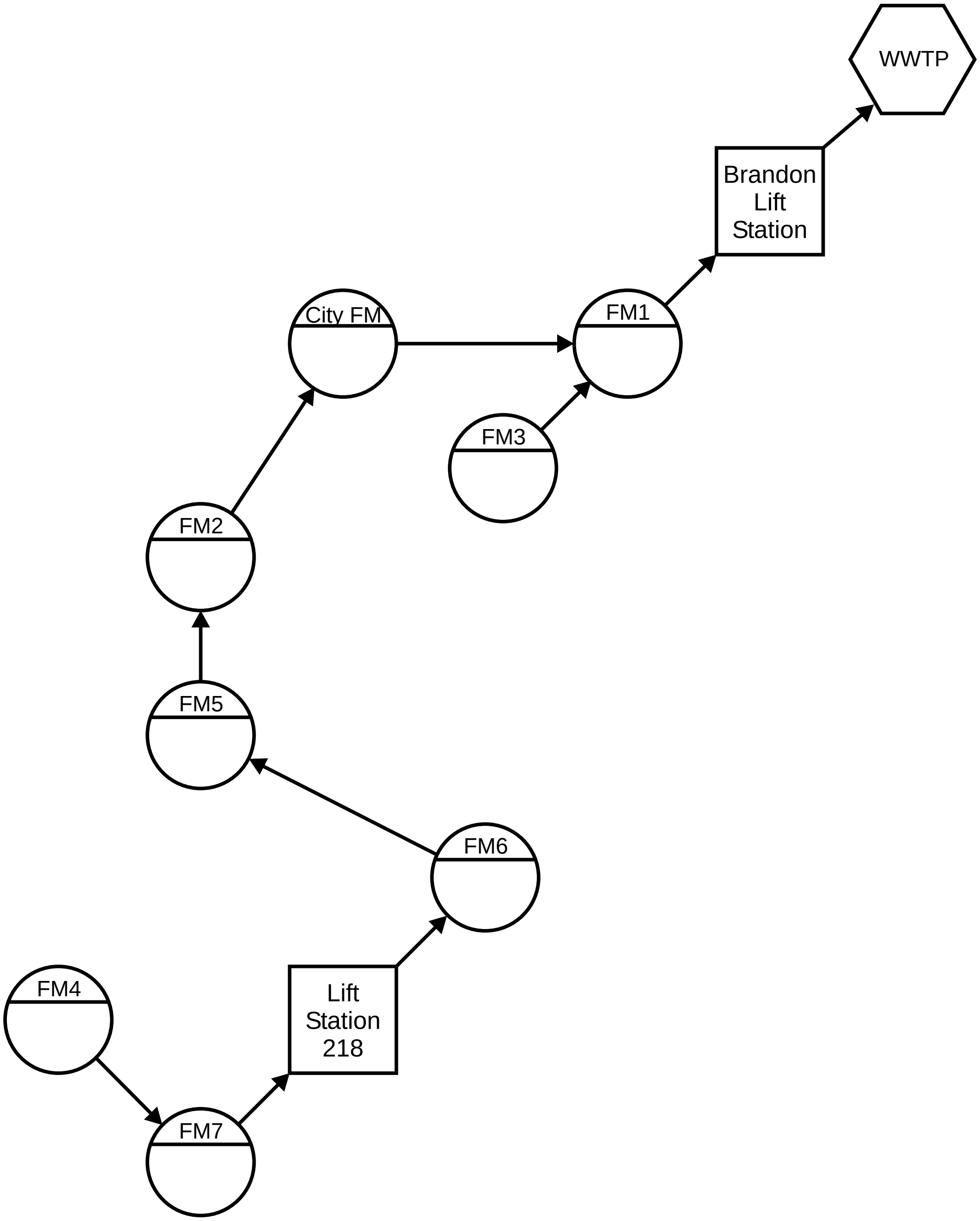
The Owner provided the following information and assistance during the temporary flow and rainfall monitoring program:

- Assistance in locating manholes for the temporary metering stations and locating sites for rainfall gauge placement.

- Access to manholes.

- Safe entry to manholes including ventilation.

- Provision of the permanent monitor SCADA data.



Sioux Falls, SD
Sanitary Sewer Collection System Facilities Plan

Flow Monitoring Schematic

Figure 3-3

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3.2.4 Preliminary Data Analysis and Review

A schematic drawing of the relationship between the monitored areas or subsystems is shown on Figure 3-3. The City in a previous study defined basins and subbasins. A schematic drawing of the flow relationship between the basins is shown in Appendix D.

3.2.4.1 Flow Monitor Profiling and Calibration

Flow monitor profiling and calibration was performed to determine hydraulic conditions at each flow monitoring site. Monitor profiling consists of punctual flow velocity and depth checks to compute the actual flow and observed hydraulic gradient at the monitoring site. Profiling was performed in conjunction with weekly data collection.

Information collected during monitor profiling was analyzed to determine flow monitor calibration. Flow monitor calibration served two purposes. First, any necessary adjustments to flow monitoring data were identified by comparing profiled measurements to data recorded simultaneously by the flow monitor. These changes were made by modifying parameters in flow monitor manufacturer's data collection software after the monitoring period has concluded. Second, the observed hydraulic gradient was used to calculate the calibrated pipe capacity at the flow monitor.

This capacity is characteristic of the reach of pipe in the immediate vicinity of the flow monitor. The theoretical design capacity is calculated by Manning's formula for uniform flow conditions using the modeled slope, the nominal pipe size, and the energy gradient. The theoretical design capacity is the average capacity over the length of pipe with the indicated slope. The hydraulic conditions and the calibrated capacities at each temporary meter site during the monitoring period were summarized in Table 3-6.

Table 3-6
Temporary Monitoring Site Descriptions

Monitoring Site	Pipe Diameter (in)	Average Flow Depth(1) (in)	Average Velocity(1) (fps)	Existing Conditions: Energy Gradient ($s^{1/2}/n$)	Existing Conditions: Calibrated Pipe Capacity(2) (mgd)	Design Parameters: Modeled Slope (%)	Design Parameters: Pipe Capacity(3) (mgd)
FM1	66	19.0	4.39	4.027	113.6	0.095	66.9
FM2	60	24.7	2.81	2.308	50.5	0.095	51.9
FM3	37	9.1	1.07	1.579	9.5	0.050	10.4
FM4	24	8.2	2.10	3.441	6.5	0.099	4.6
FM5	60	20.0	2.43	2.257	49.4	0.100	53.2
FM6	41	13.6	4.06	4.823	38.2	0.145	23.2
FM7	41.5	27.3	0.89	0.706	5.8	0.078	17.6

(1) Average depth and velocity from calibration site visits.
(2) Capacity based on calibrated energy gradient.
(3) Capacity based on modeled slope and diameter.

Flow monitor calibration for this project proved challenging. Large flow depths and toxic (H2S) atmospheric conditions hampered accurate manual depth measurements. The wet well and pump operation practices at the Tuthill and Brandon Lift Stations may have caused variations in hydraulic gradient dependent upon the time of observation. Final monitor calibration was accomplished by using manual profiled flow velocities but ignoring manual depth measurements in favor of values recorded by the flow monitors. This methodology permitted calibration of flow monitors to acceptable statistical confidence ranges. Site calibration worksheets were included in Technical Memorandum 2.

3.2.4.2 Subsystem Areas

Developed area is used in calculating rates of ADDF, infiltration and inflow as discussed later in this chapter. Residential and ICI acres were determined during the land use analysis presented in Chapter 2. Summing the residential and ICI acres tributary to each flow meter provided the developed area in each monitored subsystem. The current incremental and cumulative developed acres information for each temporary flow monitoring area is listed in Table 3-7.

Table 3-7
Developed Areas by Flow Monitor Drainage Area

	Drainage Area Developed Area (acres)	
Flow Monitor	Incremental	Cumulative
FM1	2,798	18,730
FM2	5,201	15,383
FM3	549	549
FM4	1,526	1526
FM5	2,936	10,182
FM6	2,932	7,246
FM7	2,788	4,314
Total	**18,730**	--

3.3 Rainfall Data Analysis

The purpose of the rainfall monitoring was to evaluate observed rainfall events for use in determination of inflow parameters. These values form part of the basis for analyzing existing wastewater collection system capacity and projecting future system requirements.

3.3.1 Design Flow and Probability

Design flow for a sewer is defined as the maximum flow that a specified structure can pass without overload. Since a significant portion of the peak flows in sanitary sewers is inflow resulting from rainfall, the design flow that the sewer must convey is related to the probability of occurrence of a design storm event. Design flow for a selected rainfall event is the sum of three components: (1) peak wastewater production; (2) total infiltration; and (3) inflow. As presented later, inflow is a function of the local intensity-duration-frequency relationship for rainfall. This relationship introduces a probability consideration to the development of the design flow.

A summary of the probability that a storm event having a prescribed recurrence interval will not be equaled or exceeded during a specified period is given in Table 3-8. For example, a design based on a 10-year storm event has a 59 percent chance of not being exceeded during a five-year period.

Table 3-8
Probability of Non-Exceedance

Design Storm (years)	Period (years)							
	1	5	10	20	50	100	200	500
1	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)
2	0.50	0.03	0.01	(1)	(1)	(1)	(1)	(1)
5	0.80	0.33	0.12	0.01	(1)	(1)	(1)	(1)
10	0.90	0.59	0.35	0.12	(1)	(1)	(1)	(1)
25	0.96	0.82	0.66	0.44	0.13	0.02	(1)	(1)
50	0.98	0.90	0.82	0.67	0.36	0.13	0.02	(1)
100	0.99	0.95	0.90	0.78	0.61	0.37	0.13	0.01

(1) Values are near 0.

3.3.2 Analysis of Rainfall Data

3.3.2.1 Background

The normal annual average rainfall for the study area is 23.86 inches as summarized from climatological data from the National Oceanic and Atmospheric Administration (NOAA). Historical data on average monthly rainfall amounts and rainfall intensity-duration relationships are presented in Tables 3-9 and 3-10 and shown graphically on Figure 3-4. The rainfall intensity-duration relationships for Sioux Falls were developed from Technical Paper 40, "Rainfall Frequency Atlas of the United States", published by the former U.S. Weather Bureau. This source is also the basis for the intensity-duration-frequency (IDF) curve in the City of Sioux Falls Drainage

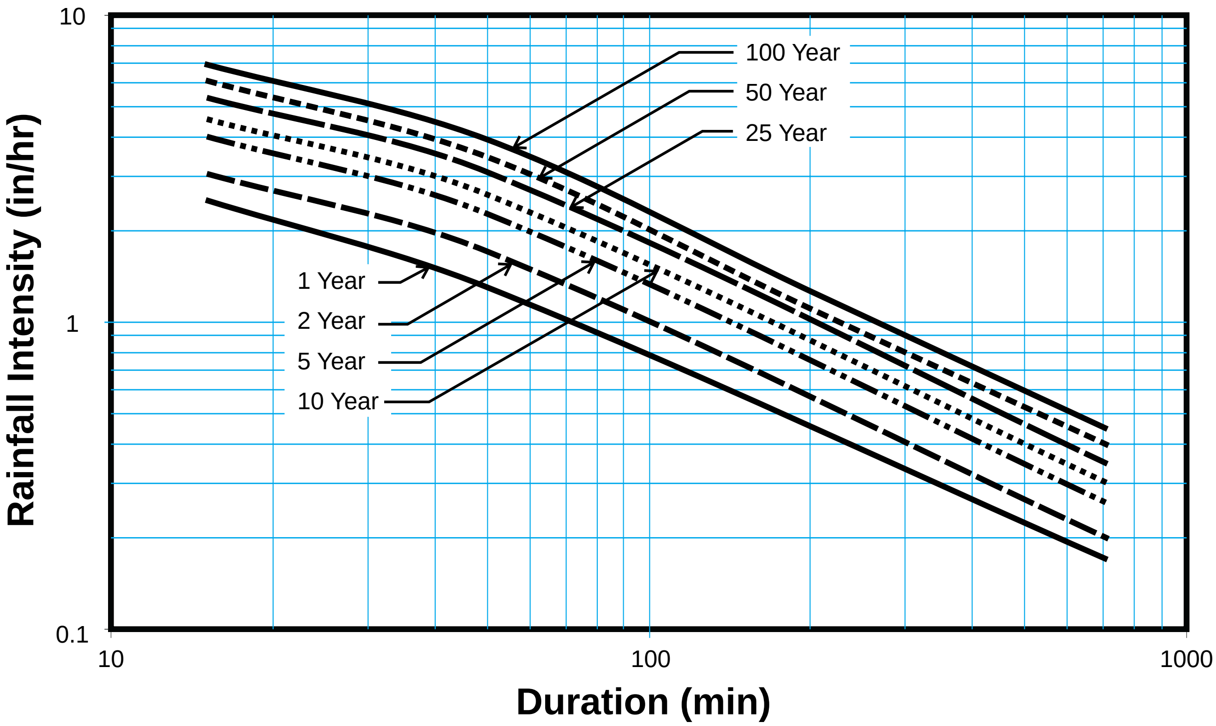
Improvements (Sioux Falls Engineering Design Standards for Public Improvements – Chapter 11, Figure 11.1).

Table 3-9
Historical Average Rainfall
Sioux Falls, SD

Month	Average Precipitation (in)	Cumulative Precipitation (in)
January	0.51	0.51
February	0.64	1.15
March	1.64	2.79
April	2.52	5.31
May	3.03	8.34
June	3.40	11.74
July	2.68	14.42
August	2.84	17.26
September	3.04	20.30
October	1.78	22.08
November	1.08	23.16
December	0.70	23.86

Table 3-10
Rainfall Depth – Duration – Frequency Relationship,
Sioux Falls, SD

Return Period (Years)	Total Rainfall (inches) for Duration Indicated						
	30 Min	60 Min	2 Hrs	3 Hrs	6 Hrs	12 Hrs	24 Hrs
1	0.9	1.2	1.4	1.5	1.7	2.0	2.2
2	1.2	1.5	1.7	1.8	2.0	2.4	2.7
5	1.5	2.0	2.2	2.4	2.7	3.1	3.4
10	1.8	2.3	2.7	2.8	3.2	3.6	4.0
25	2.1	2.7	3.0	3.2	3.6	4.2	4.8
50	2.4	3.0	3.4	3.6	4.2	4.7	5.3
100	2.7	3.4	3.8	4.1	4.6	5.3	5.8



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Sioux Falls, SD
Wastewater Master Plan - 2001

Rainfall Intensity Duration Frequency Relationship

Figure 3-4

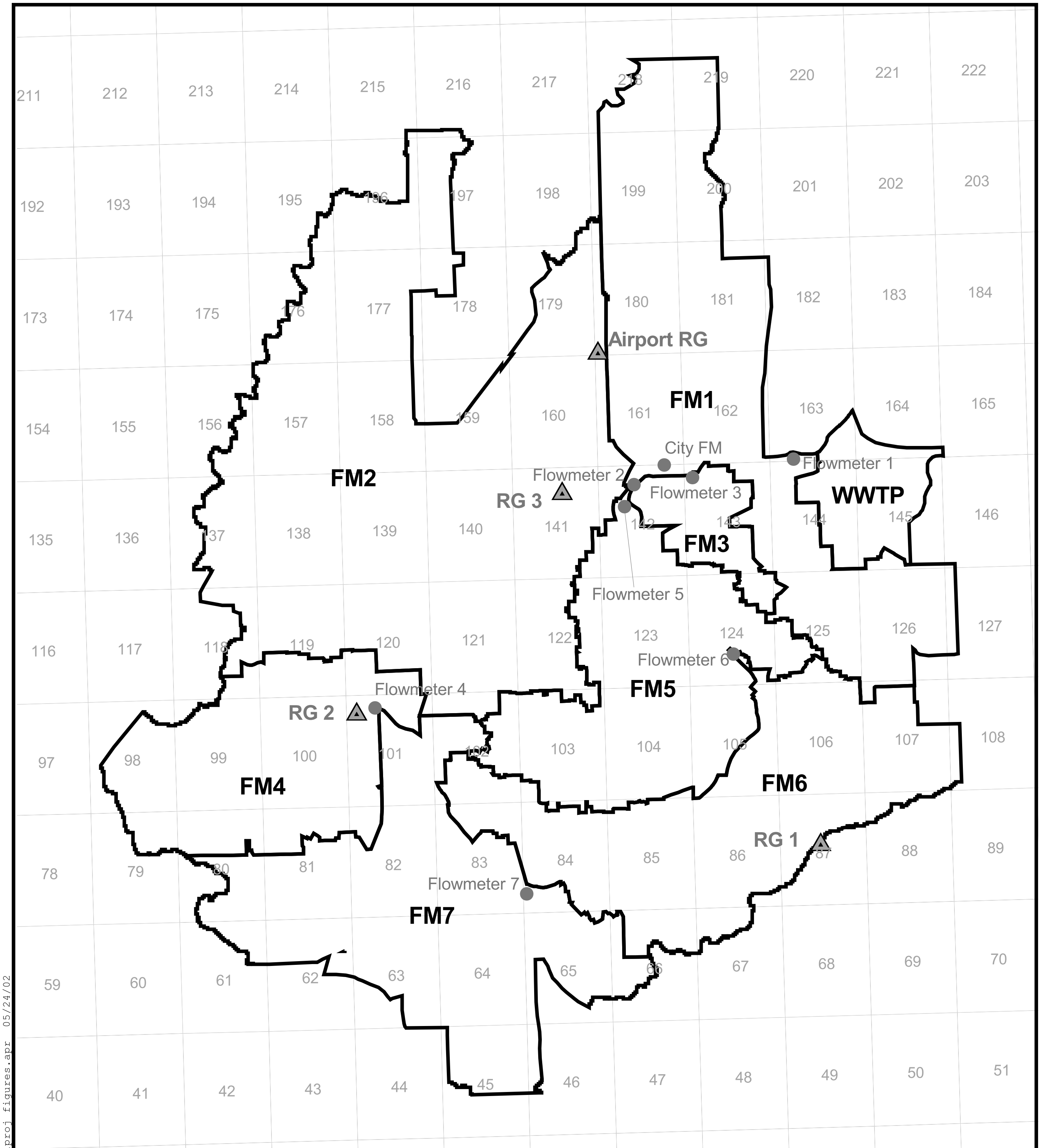
3.3.2.2 Monitored Rainfall

The ground-based rainfall data was reviewed and specific rainfall events were selected for analysis based on storm magnitude and duration. Rainfall totals and distributions were developed with the calibrated NEXRAD rainfall data for each subsystem tributary area as defined by the permanent monitor location. The data was compared against the known rainfall intensity-duration-frequency relationship for the study area to determine the return interval of each storm event.

Rainfall intensities were evaluated for correlation of peak rain intensity to the peak flow rate in the interceptors. The highest flow for a given storm event is generated when the storm duration has reached the travel time from the farthest point in the system to the flow monitor location.

Eight storms of varying total measured rainfall and duration were recorded during the flow monitoring period of July 30 to September 26, 2001. Three of the storm events were selected for analyses based on significant rainfall and observation of a definable flow response. A rainfall or storm event is defined as continuous recorded rainfall with each event separated by a minimum of six hours. Each of selected three storm events totaled at least 0.50 inches in depth. The storm event with the largest total rainfall occurred on September 13, 2001 and averaged about 1.09 inches in a 38-hour period over the entire study area. This total is over 35 percent of the historical average of 3.04 inches for the month of September.

For the analysis of inflow versus rainfall, it was necessary to determine the rainfall pattern for each rain event applicable to each flow monitor's tributary area. The rainfall in each NEXRAD grid pixel within the tributary area was averaged for each 15-minute time step. Most pixels do not fall entirely into a single subsystem. Some pixels fall into more than one subsystem. The contribution of each pixel to the average rainfall in a subsystem was weighted by the percentage of the pixel in the subsystem. This procedure resulted in a highly detailed calibrated NEXRAD rainfall pattern.. Figure 3-5 shows the relationship between NEXRAD pixels and monitored subsystems. Table 3-11 shows the number of pixels used to calculate average rainfall for each subsystem cumulative tributary area. The subsystem cumulative area includes the incremental area of the subsystem plus all upstream areas. Refer to Figure 3-3, the Flow Monitoring schematic, for subsystem cumulative relationships.



Legend

- Flow Meter
- Rain Gauge
- Metered Sub System
- Pixel

N

0 5000 10000 15000 Feet

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Sioux Falls, SD
Wastewater Master Plan - 2001

NEXRAD Pixels and Rain Gauge Locations

Figure 3-5

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Table 3-11
NEXRAD Virtual Rain Gauges Per Cumulative Tributary Area

Flow Monitoring Subsystem	Number of NEXRAD Pixels Used to Calculate Average Rainfall in the Tributary Area
FM1	80
FM2	67
FM3	6
FM4	27
FM5	60
FM6	55
FM7	41

Summaries of the observed daily total rain for the total area tributary to each subsystem are given in Tables 3-12 and 3-13. Each rainfall event was further analyzed to determine the return interval for selected rainfall durations by comparing the recorded data to the rainfall intensity-duration-frequency curves for Sioux Falls. For example, the peak rainfall intensity/duration relationship during each selected storm event for monitor FM3, is given in Table 3-13. At a duration of 60-minutes, the peak rainfall intensity for the August 29, 2001 storm was thirty-six percent of a 1-year storm event. Figure 3-6 shows rainfall event totals for the August 29, 2001 storm by NEXRAD pixel across the monitored area.

Table 3-12
Monitored Rainfall Totals

Rain Date 2001	Total Rainfall for Each Rain Event by Subsystem (Inches)						
	FM1	FM2	FM3	FM4	FM5	FM6	FM7
1st 08/29	0.01	0	0	0.02	0	0.01	0.01
2nd 08/29(1)	0.94	0.78	1.07	0.91	0.84	0.77	0.75
1st 09/07	0.15	0.15	0.1	0.12	0.17	0.16	0.13
1st 09/08	0.34	0.29	0.34	0.3	0.34	0.32	0.28
1st 09/09	0	0	0.01	0	0	0	0
1st 09/13(1)	1.11	0.99	1.1	1.13	1.06	1.06	1.04
1st 09/15	0	0	0.01	0	0.01	0.01	0
2nd 09/15(1)	0.69	0.53	0.99	0.51	0.63	0.59	0.51
Total	3.24	2.74	3.62	2.99	3.05	2.92	2.72

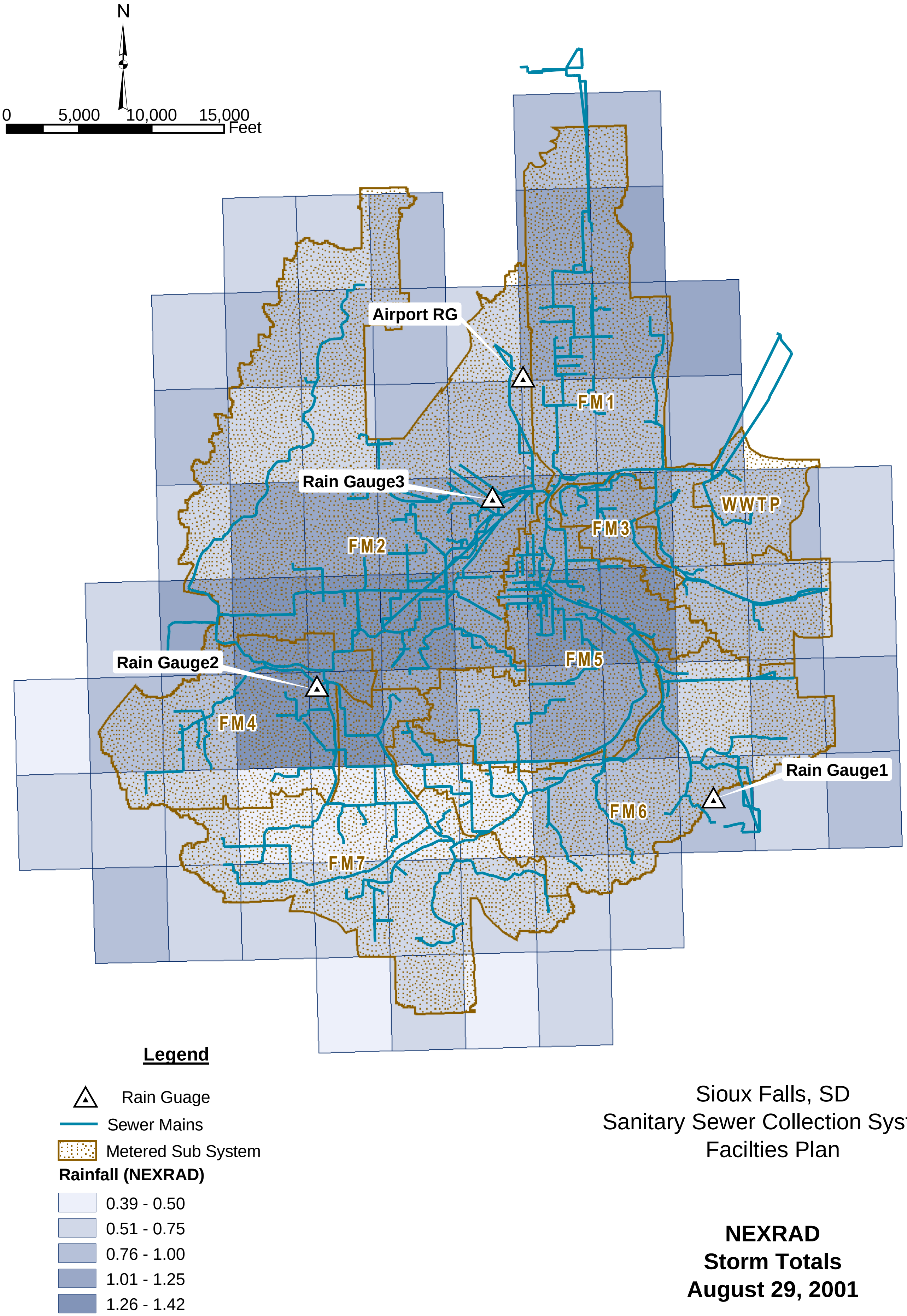
(1) Significant rain dates selected for inflow analysis. 1st or 2nd event of the day.

Table 3-13

Monitored Peak Rainfall Depth vs. Duration for Significant Storms

Date In 2001	Peak Rainfall Depth (in.) For Duration Indicated					
	30 (min)	60 (min)	120 (min)	180 (min)	240 (min)	600 (min)
Standard 1-Yr. Storm						
-	0.9	1.2	1.4	1.5	1.7	1.9
Observed Storm Events						
2nd 08/29	0.256	0.431	0.633	0.751	0.970	1.171
1st 09/13	0.060	0.120	0.237	0.347	0.439	0.621
2nd 09/15	0.160	0.267	0.521	0.609	0.761	1.064

Note: This Table shows representative data for subsystem FM3.



Sioux Falls, SD
Sanitary Sewer Collection System
Facilties Plan

NEXRAD
Storm Totals
August 29, 2001

Figure 3-6

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3.4 Flow Data Analysis

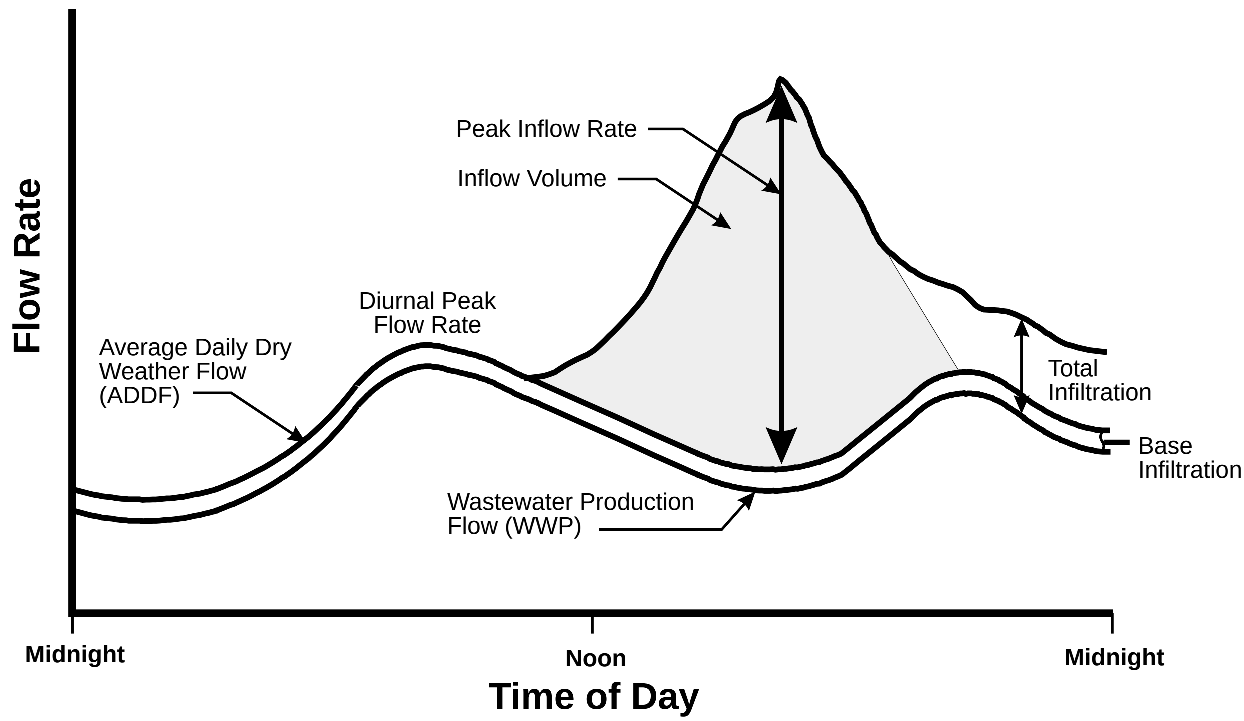
The wastewater flow data was reviewed to select the most representative days of data recorded for use in the determination of dry and wet weather flow parameters. Dry weather days were selected to provide the best estimation of base wastewater production. The analysis of wet weather flow data corresponded with the selected rain events.

3.4.1 Flow Components

For purposes of this report, WWP is defined as the wastewater exclusive of infiltration and inflow. The daily wastewater production flow rate can be approximated using (1) direct measurement of ADDF during dry weather/low groundwater conditions or (2) winter month water consumption data. Winter month water consumption was not investigated for this project. The instantaneous wastewater production flow rate varies throughout each day, with the highest rates normally occurring between 8:00 and 11:00 a.m. The ratio of peak 60-minute flow to total average daily flow is defined as the dry weather peaking factor.

Infiltration is groundwater entering the wastewater collection system and private building lines through defective pipes, pipe joints, and manhole structures below the manhole corbel and chimney. The rate of infiltration depends on the depth of groundwater above the defects, the size of the defects, and the percentage of the collection system submerged. The variation in groundwater levels and the associated infiltration is seasonal and weather-dependent. Low groundwater/dry weather infiltration is infiltration that occurs year-round and is measured during dry weather when previous rainfall is no longer having an effect on flows. High groundwater/dry weather infiltration is the additional infiltration that occurs due to higher groundwater conditions following rain events.

Inflow is rainfall-related water which enters the collection system from sources such as private sewer laterals, downspouts, foundation drains, yard and area drains, storm water sump pumps, manholes, defective piping, and cross-connections with storm drains. Inflow is directly influenced by the intensity and duration of a storm event, and therefore is not a fixed quantity. Figure 3-7 illustrates these flow components.



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Sioux Falls, SD
Wastewater Master Plan - 2001

Flow Components

Figure 3-7

3.4.2 Flow Monitoring Data

3.4.2.1 Determination of Average Daily Dry Weather Flow

Daily fluctuations in flows are attributable to variations in domestic, industrial, and commercial wastewater production. ADDF is a flow parameter measured directly by flow monitoring and includes WWP plus the portion of total infiltration that occurs during low groundwater conditions. The ADDF for each monitor was determined using the average flow at the monitor for selected 7-day periods based on data availability. Typically, during this evaluation dry weather/low groundwater period infiltration is negligible.

A flow balance was performed using the ADDF recorded at each temporary flow monitor site. This process is an accounting procedure for balancing flows recorded throughout the system. At the same time, flows were checked against the developed acres tributary to each meter (to determine the per developed acre use rate (gpad) for each subsystem). In order to provide reasonable values for incremental flows throughout the system, flows at FM2 and FM3 were balanced using cumulative system unit rates at FM1. The subsystem and cumulative ADDF values and rates are shown in Table 3-14. The ADDF per developed acre rates range from 510 gpad to 1009 gpad.

Dry weather peaking factors (the ratio of the cumulative peak 60-minute flow to cumulative average daily flow measured during dry weather/low groundwater conditions) were determined for each monitor. The system-wide average was about 1.4. In the computer model dry-weather diurnal curves were input for each monitor area to dynamically generate the peaking factors. The shape of each curve was determined from the dry weather flow data. Diurnal curves generally have two peaks, the largest peak occurring in the morning and the second occurring in the evening. The diurnal peaking factors are shown by subsystem in Table 3-14. Diurnal curves for each cumulative area tributary to each monitor are included in Appendix A.

Table 3-14
Subsystem ADDF and Peak Dry Weather Flow Summary

Subsystem	Measured ADDF (mgd)	ADDF Rates			Peak dry weather flow	
		Subsystem ADDF Rate (gpad)	Cumulative ADDF Rate (gpad)	Peaking Factor (Qp/Qa)	Subsystem (mgd)	Cumulative (mgd)
FM1	14.506	510	774	1.286	1.835	18.655
FM2	12.559	703	816	1.222	4.470	15.347
FM3	0.520	947	947	1.499	0.779	0.779
FM4	1.459	806	806	2.009	2.931	2.931
FM5	8.901	942	874	1.226	3.392	10.913
FM6	6.134	1,009	847	1.272	4.420	7.802
FM7	2.659	603	699	1.452	1.742	3.861

3.4.2.2 Determination of Infiltration

Total infiltration consists of base (dry weather/low groundwater) infiltration and dry weather/high groundwater infiltration. Infiltration during high groundwater periods is measured on days after significant rainfall events. The total flow measured during these infiltration periods includes WWP plus both base and high groundwater infiltration flows.

The observed infiltration values by subsystem are shown in Table 3-15. The infiltration rates ranged from 52 gpd/acre to 421 gpd/acre. For reference, the 1990 facility plan assigned 400 gpd/acre for infiltration, under a 50 year frequency wet month.

Table 3-15
ADDF and Total Infiltration Flows

Subsystem	Subsystem Developed Area (acres)	Subsystem ADDF + Total Infiltration (mgd)	Subsystem ADDF (mgd)	Subsystem Total Infiltration (mgd)	Subsystem Infiltration Rate (gpd/acre)
FM1	2,798	2.437	1.427	1.010	361
FM2	5,201	3.927	3.658	0.269	52
FM3	549	0.751	0.520	0.231	421
FM4	1,811	1.806	1.459	0.347	192
FM5	2,936	3.597	2.767	0.830	283
FM6	3,444	4.328	3.475	0.853	248
FM7	1,991	1.666(1)	1.200	0.466(1)	234
Total	**18,730**	**18.512**	**14.506**	**4.006**	**214**

(1) Infiltration in FM7 was adjusted to match cumulative system unit rates because preferred days for high groundwater source data were not available.

3.4.2.3 Determination of Inflow

Inflow for a specific storm event includes all rainfall-induced flow, including direct storm water inflow and rapid infiltration. The flow data for each significant rainfall event was analyzed for inflow. The total peak flow measured during inflow periods includes wastewater production flow, infiltration, and inflow. Inflow for a particular rainfall event is determined by subtracting the wastewater production and infiltration flow from the measured peak flow.

The magnitude of peak inflow depends on rainfall distribution, intensity, antecedent groundwater conditions, types and locations of inflow sources, and time of concentration of the system to the monitoring point. A preliminary inflow coefficient "K" was determined for each rainfall event at each monitoring location. The inflow coefficient is an attempt to combine all system variables into a single parameter. The time of concentration is the time from initiation of peak rainfall to the time of peak inflow. Generally, the time of concentration increases as the total tributary area increases; and the inflow coefficient is greater for older systems.

The inflow coefficient developed for each flow monitoring area was based on specific inflow coefficients calculated for each monitored storm event producing discernable inflow response to rainfall. The average inflow coefficient is used to determine inflow for any selected recurrence interval storm event using the following inflow coefficient method relationship:

$$Q = KiA$$

where: Q = peak inflow (cfs)
K = inflow coefficient
i = rainfall intensity for selected recurrence interval and time of concentration (in/hr)
A = developed area (acres)

A summary of tributary areas, times of concentration and inflow coefficients is given in Table 3-16. Inflow for a storm with any selected recurrence interval can be determined using the inflow parameters.

Table 3-16
Summary of Inflow Parameters

Subsystem	Developed Area (acres)		Time of Concentration (min)		Inflow Coefficient, K	
	Subsystem	Cumulative	Subsystem	Cumulative	Subsystem	Cumulative
FM1	2,798	18,730	90	255	0.0064	0.0048
FM2	5,201	15,383	90	210	0.0037	0.0034
FM3	549	5,49	150	150	0.0372	0.0372
FM4(1)	1,811	1,811	75	75	0.0032	0.0032
FM5	2,936	10,182	75	195	0.0032	0.0032
FM6(1)	3,444	7,246	90	165	0.0032	0.0032
FM7(1)	1,991	3,802	90	120	0.0032	0.0032

(1) The inflow coefficient for subsystems FM4, FM6 and FM7 was based upon the inflow coefficient at the downstream subsystem (FM5).

One subsystem (FM3) is shown to have an inflow coefficient greater than 0.01. Subsystem FM3 also is the subsystem with the highest 1-year inflow rate. The 1990 Facilities Plan noted the presence of area drains in stockyard cattle pens in subsystem FM3.

Not all subsystems were directly assessed. Wet weather analysis was not performed on data from monitors FM4, FM6, and FM7 because measurable response to rainfall was difficult to differentiate from normal diurnal pattern variations. The rates for these monitors were assigned the rate determined for the downstream monitor, FM5.

It also should be noted that flow monitors FM1 and FM5 registered brief velocity spikes and corresponding flow surges on September 13. FM1 also registered this phenomenon on September 16. These spikes were of short duration and showed quadrupling of flow velocities with minimal variation in flow depth. Discussions with WWTP personnel suggest that these surges may be due to the operation of variable speed pumps and not indicative of additional inflow volume. These data points were thus neglected in the determination of inflow coefficients.

Cumulative and subsystem inflows were determined for each monitoring point for a one-year storm event as shown in Table 3-17. The 1-year inflow rate provides a comparison between subsystems, and will be used in the calibration of the hydraulic model. A comparison of cumulative inflow and subsystem-generated inflow rates shows that the cumulative inflow for interior subsystems is less than the sum of individual subsystem-generated inflows. This fact is consistent with expected system dynamics in which peak flows are dampened as they travel through the system and critical for any comparison of projected I/I source flow to monitored flow.

Table 3-17
Inflow Summary

Subsystem	Subsystem Developed Area (acres)	Time of Concentration (min)	1 Year Rainfall Intensity (in/hr)	1-year inflow (mgd)		
				Subsystem	Cumulative	Subsystem 1-year Inflow Rate (gpd/acre)
FM1	2,798	90	0.87	10.04	22.95	3,589
FM2	5,201	90	0.87	10.82	14.81	2,080
FM3	549	150	1.00	7.66	7.66	13,945
FM4(1)	1,811	75	1.00	3.79	3.79	2,094
FM5	2,936	75	0.58	6.15	10.66	2,094
FM6(1)	3,444	90	0.87	6.25	8.14	1,815
FM7(1)	1,991	90	0.87	3.61	5.57	1,815

(1) The inflow for subsystems FM4, FM6 and FM7 was based upon the inflow coefficient at the downstream subsystem (FM5).

3.4.2.4 Peak Flow vs. Existing Capacity

Projected peak flows for storm events with various recurrence intervals were compared to the pipe capacity at the monitoring locations. The existing capacity at each monitoring location was calculated based on the monitor profiling performed during flow monitoring. The approximate level of protection at each of these points was estimated by comparing peak flows to existing capacity. The level of protection refers to the return frequency of the storm event that would overload the sewer. The data are only representative of the system at the monitoring point, and may not represent upstream flow conditions. The data shows that two of the seven locations have less than 1-year storm protection. A summary of data is presented in Table 3-18. Appendix B contains graphs comparing peak flow versus rainfall intensities against pipe capacity at flow meter sites. The rainfall intensities for the 1-year and 5-year rainfall events are shown on each graph for reference.

Table 3-18 Existing Capacity and Peak Flows							
Subsystem	Existing Capacity (mgd)	Peak ADDF + Infiltration (mgd)	Cumulative Peak Flows (mgd)				Approximate Existing Level of Protection
			1-Year Storm	2-Year Storm	5-Year Storm	10-Year Storm	
FM1	113.6	22.7	45.6	50.0	59.3	65.6	10-year
FM2	50.5	18.1	32.9	35.8	41.8	45.8	10-year
FM3	9.5	1.0	8.7	10.2	13.2	15.5	1-year
FM4	6.5	3.3	7.1	8.0	9.5	10.6	< 1-year
FM5	49.4	13.4	24.1	26.2	30.5	33.3	10-year
FM6	38.2	9.5	17.6	19.3	22.4	24.8	10-year
FM7	5.8	4.7	10.2	11.4	13.4	15.4	< 1-year

Note: 1. Level of protection = storm recurrence interval which will overload the system.
2. Peak ADDF = Peak Daily Dry Weather Flow

3.4.3 Determination of Existing ADF

Having determined each of the wastewater flow components (ADDF, infiltration, inflow), it was possible to estimate the average annual daily flow (ADF) by extrapolating the results of the eight week monitoring period for a yearlong timeframe. The total annual contribution from infiltration assumed 180 days flow at the total infiltration rates shown in Table 3-15. The total annual inflow rate was estimated considering the inflow coefficient and annual rainfall. The results of ADF determination for each flow monitor are presented in Table 3-19. The ADF analysis indicated that, except for FM3, wet-weather induced flow accounts for less than 20 percent of total flow at each flow monitoring location.

Table 3-19 Average Annual Daily Flow						
Subsystem	ADDF (mgd)	ADDF (MG/yr)	Infiltration (MG/yr)	Inflow (MG/yr)	ADF (mgd)	ADF/ADDF Ratio
FM1	14.51	5,295	721	124	16.82	1.16
FM2	12.56	4,584	498	87	14.16	1.13
FM3	0.52	190	42	47	0.76	1.46
FM4	1.46	533	62	27	1.70	1.17
FM5	8.90	3,249	449	59	10.29	1.16
FM6	6.13	2,239	300	49	7.09	1.16
FM7	2.66	971	146	36	3.16	1.19

3.5 SCADA System Flows

3.5.1 Comparison of Temporary Flow Monitoring to SCADA-Recorded Flows

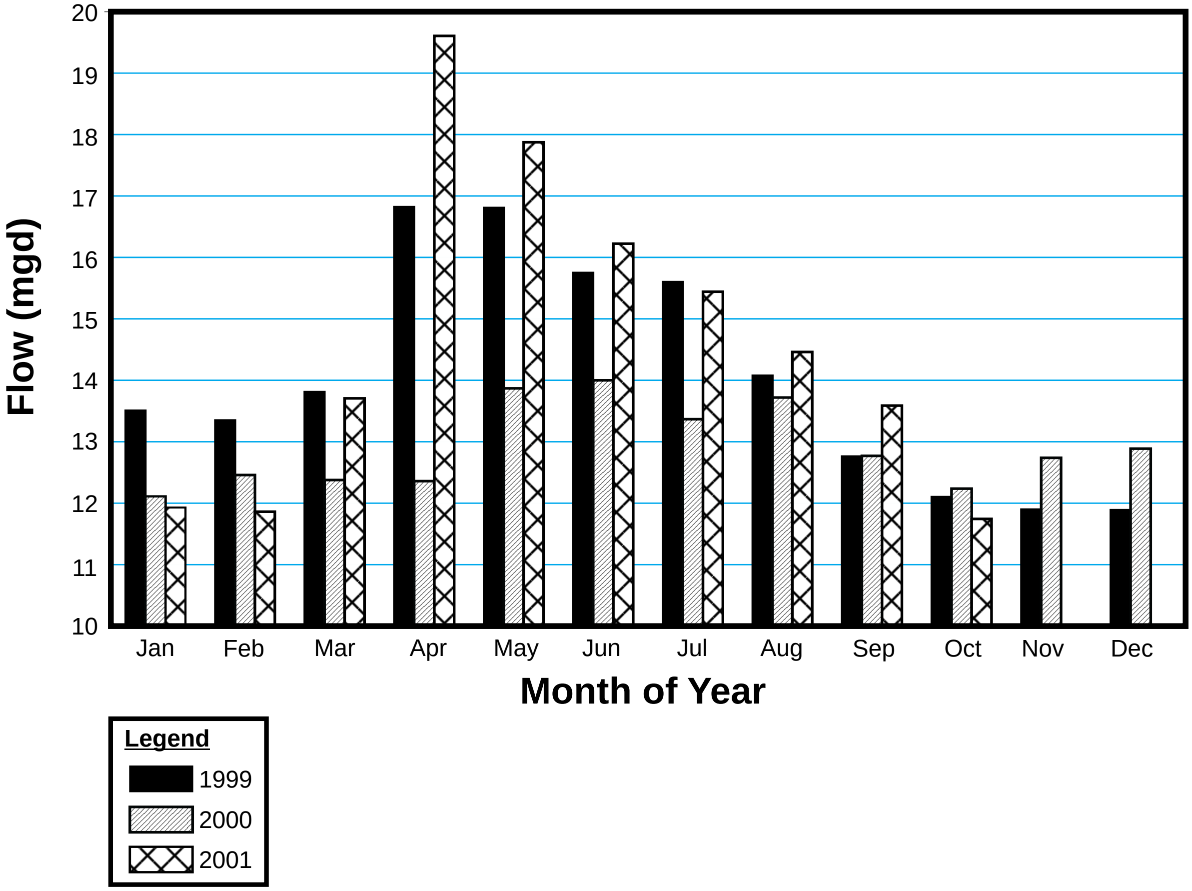
3.5.1.1 Historical WWTP Flows

Monthly WWTP flow records from January 1999 to October 2001 were reviewed to corroborate data acquired during temporary flow monitoring. Table 3-20 presents monthly average influent flows for this time period. There is a significant variation in flows, both between months of a given year and for given months in different years. For all three years, average annual flows are less than predicted by FM1. This data is shown in a graphical form on Figure 3-8.

A long, consistent recession of average flows is observed in the data from May 1999 through December 1999 again from April 2001 through October 2001. This suggests that short duration monitoring periods, such as used in this project, may not capture long term trends in infiltration, and thus longer monitoring periods (6 months or more) may allow more accurate quantification of I/I.

Table 3-20
Historical WWTP Monthly Average Flows (mgd)

Month	1999	2000	2001
January	13.51	12.11	11.93
February	13.35	12.46	11.86
March	13.81	12.38	13.71
April	16.82	12.36	19.61
May	16.81	13.87	17.88
June	15.75	14.00	16.22
July	15.60	13.37	15.44
August	14.08	13.72	14.46
September	12.76	12.77	13.59
October	12.10	12.24	11.75
November	11.90	12.74	-
December	11.89	12.89	-
Average	**14.03**	**12.91**	**-**
Minimum Month	11.89	12.11	11.75
Maximum Month	16.82	14.00	19.61



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Monthly Historical Average Monthly Flows

Figure 3-8

3.5.1.2 Comparison of FM to WWTP Flows

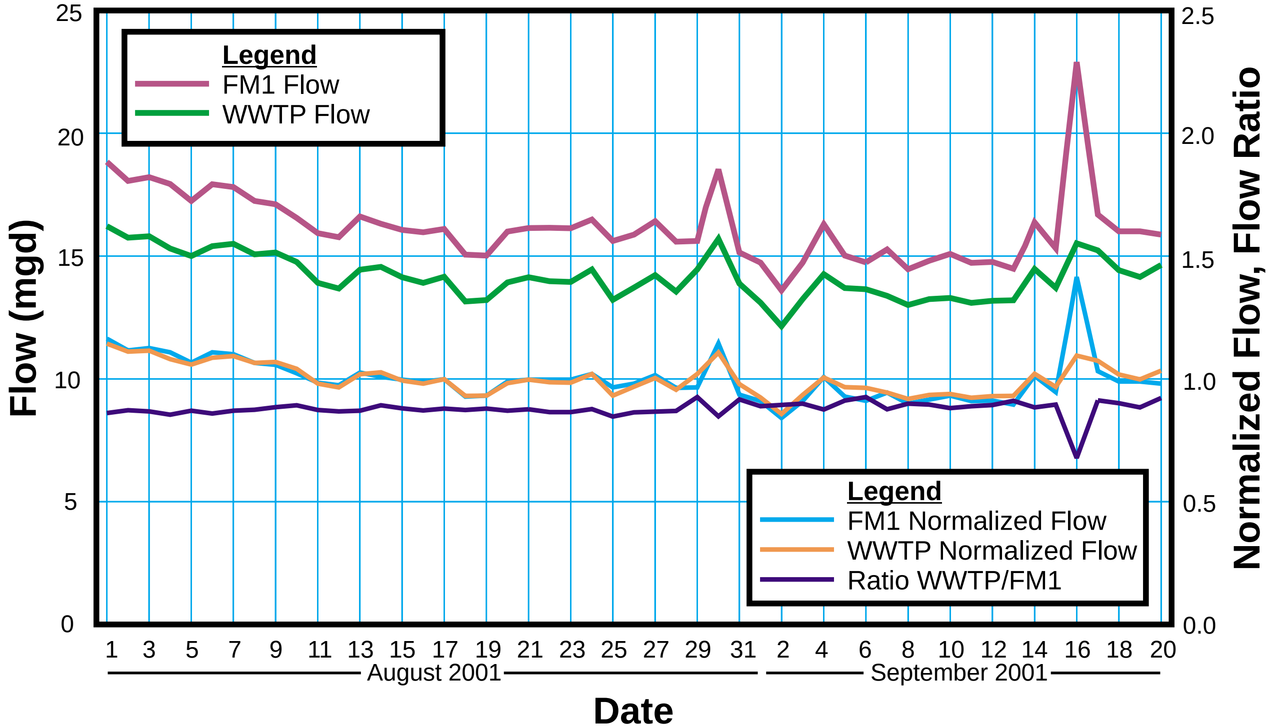
The most downstream temporary flow monitor (FM1) and the SCADA-recorded WWTP flow form the primary basis for comparison between temporary flow monitoring results and permanent, SCADA-based flow measurements. Throughout the monitoring period, FM1 and the WWTP SCADA system recorded similar trends in average daily flow and daily patterns. When average daily flow values for each monitor were normalized using the respective average flows during the monitoring period, the meters showed nearly identical results.

A calibration process was applied to all temporary flow monitors. Prior to calibration, the average flow at FM1 was 13.09 mgd. After calibration the FM1 flow average was 16.18 mgd. For comparison, the WWTP flow averaged 14.03 mgd.

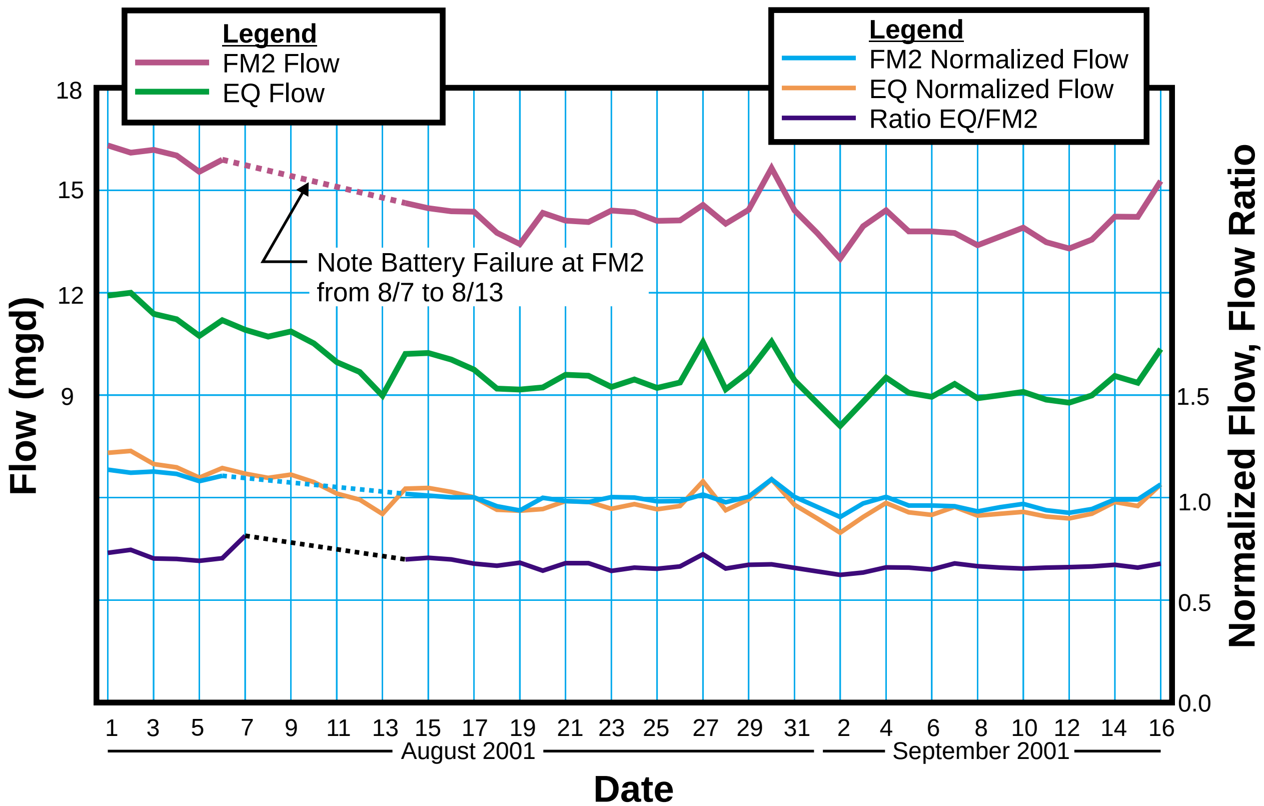
Each quarter, City personnel perform a calibration of the WWTP influent Parshall flume and SCADA equipment. The average WWTP SCADA flow values were 88 percent of the calibrated flow recorded at FM1. The difference in flow values is surprising given the thorough nature of both calibration processes. It should be noted that the accuracy of in-line flow monitoring equipment can only be guaranteed to approximately ± 10 percent. Parshall flumes are typically accurate to ± 5 percent. Additional differences may be attributable to structural conditions in the outfall sewer. The high flows and toxic (H_2S) environment prevented close inspection of the pipe for examination of possible corrosion, sediment deposition and precise diameter checks.

To further investigate the correlation between the SCADA and temporary flow monitors, the SCADA-recorded flows from the Equalization Basin facility were compared to the flows recorded by FM2. The relationship between the EQ Basin and FM2 mirrored the relationship between the WWTP and FM1. FM2 was located upstream of the EQ Basin, yet recorded consistently greater flows. Once again, the normalized trends were nearly identical. In this case, velocity profiling and calibration was performed for the temporary monitor but not for the SCADA system. The relative difference in flow magnitudes was larger between the EQ Basin and FM2 than between the WWTP and FM1. On average the EQ SCADA flow values were 68 percent of flows recorded at FM2.

Figure 3-9 shows comparisons of flows between FM1 and the WWTP SCADA system and between FM2 and the EQ Basin SCADA system. Both comparisons show actual flows, normalized flows, and the flow ratio. Data missing for FM2 between August 7 and August 13, 2001 was due to a battery failure.



WWTP vs FM1



EQ vs FM2

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Flow Monitoring Comparison
SCADA System vs. Temporary Monitors

Figure 3-9

For this project, it was concluded that permanent flow metering the WWTP's primary flow element is more accurate than the temporary flow monitors. However, the temporary monitors provide the best available data regarding incremental area flow parameters. The parameters developed in the preceeding sections are used for projecting future flows.

3.5.1.3 Review of SCADA Flows from Lift Stations

Flows from several SCADA-enabled lift stations were reviewed for correlation with data from the temporary flow monitors. Two key time-based phenomena were observed. First, several SCADA monitors recorded clear decreasing trends in average flows from the beginning of August until mid-September. This trend correlates with infiltration patterns observed at the temporary flow monitors, which resulted from heavy rain prior to the monitoring period. Second, three SCADA monitors, 203, 224, and 227, recorded clear day-of-week variations in average flow. This supports strong diurnal shape differences between days of the week observed at the temporary flow monitors.

Table 3-20 presents average weekly flows from selected lift stations. Lift stations included in Table 3-20 provided data with consistent flow patterns and average flows greater than 0.05 mgd. Also presented in Table 3-20 is the ratio of the lift station flow for each week to the 8-week average flow. This ratio illustrates the infiltration trend observed throughout the collection system. The difference between the ratio in Week 1 (1.25) and that in Week 6 (0.91) shows the relative magnitude of infiltration decline throughout the wastewater collection system.

Table 3-20
Selected SCADA-Enabled Lift Stations, Weekly Average Flows (mgd)

Lift Station	Week 1(1)	Week 2	Week 3	Week 4	Week 5(2)	Week 6(2)	Week 7	Week 8	Monitoring Period Average
203, Cherokee & "C"	N/A	N/A	0.648	0.638	0.627	0.583	0.587	0.604	0.613
204, Modern Press	0.083	0.065	0.053	0.047	0.052	0.044	0.046	0.059	0.055
224, 50th Street North	0.386	0.345	0.330	0.319	0.308	0.295	0.310	0.319	0.324
227, Highway 38A LS	0.199	0.174	0.163	0.164	0.152	0.157	0.155	0.167	0.165
233, Renner #1	0.085	0.071	0.065	0.064	0.061	0.065	0.073	0.068	0.068
Ratio, Weekly to Total Period Flows	1.25	1.10	0.98	0.96	0.95	0.91	0.95	1.03	N/A

(1) Week 1 was considered best available dry-weather high groundwater data.
(2) Days from Weeks 5 and 6 were selected to represent dry-weather low groundwater data.

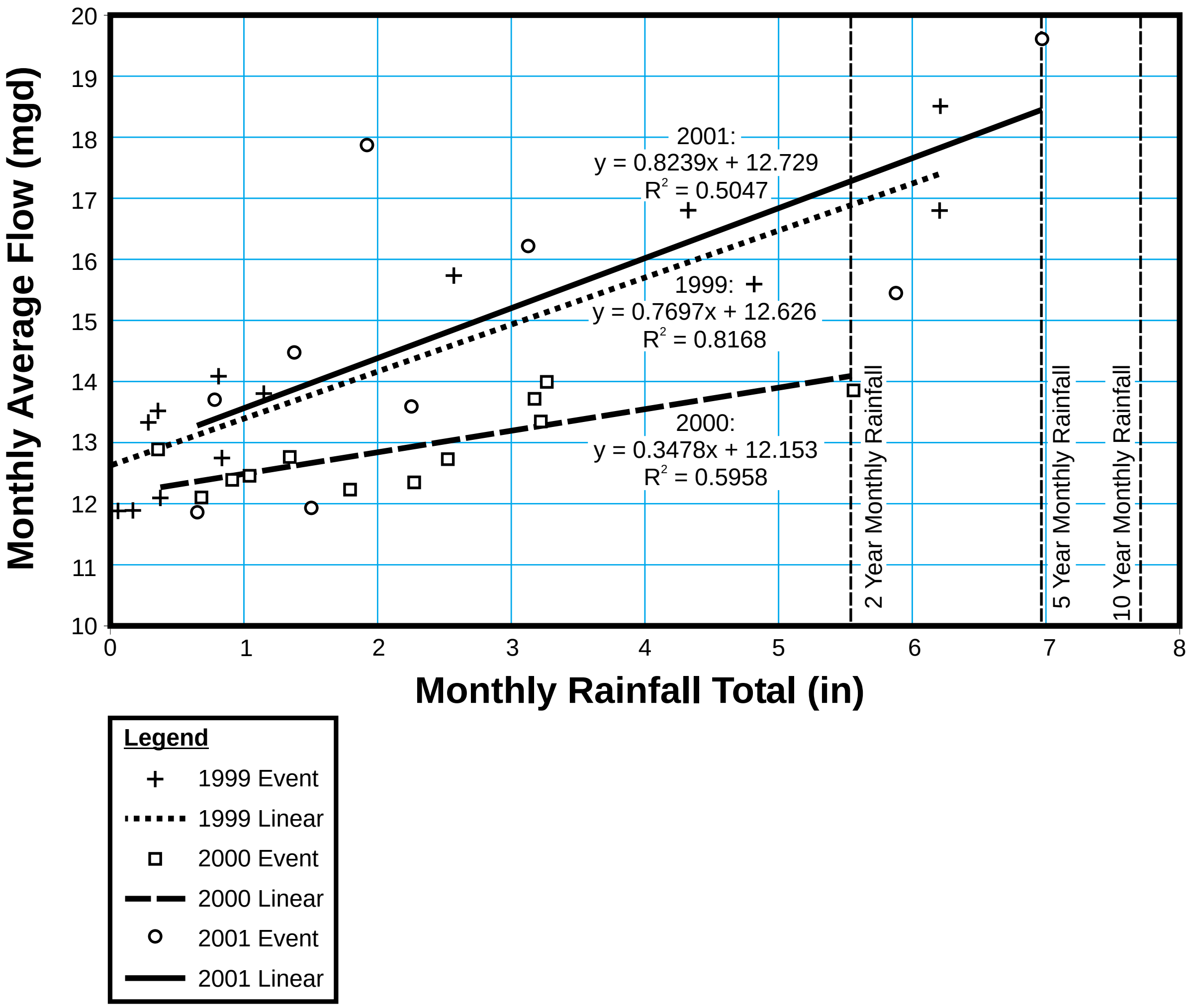
Lift stations 206 (Burnside) and 218 (Tuthill) are excluded from Table 3-20 despite qualifying on the basis of their average flow magnitudes. Data anomalies at these two lift stations were probably due either to flow monitor malfunction or data acquisition error. WWTP personnel verified the flow monitor failure at the Burnside lift station. Flow monitor error at the Tuthill station was not confirmed.

Appendix C contains graphs of daily flow averages during the monitoring period for each lift station listed in Table 3-20.

3.5.1.4 Rainfall and Monthly WWTP Flow Correlation

As an update of the 1990 Facilities Plan, the correlation of monthly WWTP flows to rainfall was developed. The monthly average flows showed a strong correlation to monthly rainfall. Figure 3-10 shows the monthly average flow to monthly rainfall relationship determined for each year. The three trends indicate an average base flow of 12.5 mgd with varying responsiveness to rainfall. The base flow is slightly smaller than that observed during in the WWTP data during the monitoring period, but that is to be expected in comparing historical flows to present flow for a community with consistent growth. The varying rainfall response may be because groundwater fluctuations, and thus infiltration, are more directly related to other variables such as river stage or river flow and less directly related to local rainfall. Analyzing the trends within specific years relates the impact of rainfall with respect to the antecedent groundwater conditions rather than attempting to frame the influence in an absolute correlation.

This analysis is relevant to temporary flow monitoring because it provides further evidence of the collection system response to rainfall observed at temporary monitors and because it provides an historical estimate of base dry-weather flow beyond the data available during the monitoring period.



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Monthly WWTP
Flow vs. Monthly Rainfall

Figure 3-10

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3.5.2 Large Industry Flows

The historical wastewater production from large water users is important in both existing system capacity analysis and projection of future wastewater flows. Table 3-21 presents average daily water consumption for the top ten water user accounts. Potable water use can be an indicator of wastewater generation, but the percentage of potablewater discharged to the wastewater collection system will vary greatly. In Sioux Falls, the ten largest users accounted for 3.8 mgd of water use in 1999 and 4.2 mgd of water use in 2000.

For the area tributary to FM3, the total ADDF was less than one quarter of the water use by the largest water user, John Morrel & Co.. This dry weather flow at FM3 is in line, however, with value reported for Drainage Basin 3 in the 1990 Facilities Plan. Since most of the large users are located in the downstream portions of the collection system, the impact of peak flows from these facilities were buffered by cumulative upstream flows. Large user water uses were considered in assigning point loads to the hydraulic model.

Table 3-21
Large Water User Data, in Order of Consumption

User Name	Address	Manhole Number	Subsystem	Average Water Use (mgd)	
				1999	2000
John Morrell & Co.	1400 N Webber Ave	03B0004	FM3	2.258	2.502
Lincoln County Rural Water	5301 S Cliff Ave	Water district(1)	Water district(1)	0.472	0.568
Sioux Valley Hospital	1100 S Euclid	10HB0010	FM2	0.189	0.190
SD Pheasantland Ind	1600 N Dr	03A0018	FM1	0.171	0.185
Norton-Froelich	1305 E 39th ST N	Sewer district (2)	N/A(2)	0.170	0.177
McKennan Hospital II	800 E 21st St	04FG004 & 04FI002	FM5	0.126	0.139
Hutchinson Technology Inc	2301 E 60th St N	17AB011	FM1	0.112	0.111
SF Stockyards	803 E Rice St	03B0013	FM3	0.112	0.110
V.A. Hospital	2501 W 22nd St	10IK001	FM2	0.099	0.109
CitiBank (SD) NA	701 E 60th St N	09D0018 & 09D0015	FM1	0.082	0.074

(1) Lincoln County Rural Water has water service only and does not return sanitary sewer flows.
(2) Norton-Froelich is a water and sewer district, it is billed as a single entity and for that reason appears on the Large Water User list. It is not sutibale for point loading.

3.6 Wastewater Flow Projections

Wastewater flow projections were developed by drainage subbasins based on the population and land uses in Chapter 2, and the unit flow rates and inflow/infiltration parameters in Chapter 3. Each wastewater flow component was projected for each planning year and then summed to determine peak storm flows. Flows were projected to the drainage subbasin level, but are presented here summarized by flow monitor area and drainage basin. Some drainage basins are split among the flow monitor areas. These partial basins are given with a decimal following the basin number to differentiate between them. Table 3-22 shows the relationship among subsystems, basins, and subbasins. Detailed projections to the subbasin level are included in Appendix D.

3.6.1 Average and Peak Daily Dry-Weather Flow Projections

ADDF was projected using per capita unit flow rates developed from the flow analysis presented in this chapter and population projections presented in Chapter 2. Peak ADDF projections were obtained by multiplying ADDF projections by the diurnal peaking factor as developed in the flow analysis.

Unit rates and peaking factors applied to drainage subbasins based on the monitored subsystem in which the subbasin was located. The relationships between drainage subbasins and monitored subsystems are shown on Figure 3-1 at the beginning of this chapter. Contributions from subbasins not included in a monitored subsystem were projected using unit rates from areas of similar development. Areas of existing development located downstream of the monitored area were assigned unit rates based on population for the cumulative monitored system. Areas of future development were assigned unit rates based on developed area for areas of recent development (FM6 and FM7).

Table 3-22 presents ADDF and Peak ADDF projections by drainage basin for the three planning years, 2001, 2015 and 2025. System totals of basin Peak ADDF projections are not presented because they do not accurately reflect the future WWTP Peak ADDF. This inaccuracy is due to peak attenuation as flows travel through the collection system.

Table 3-22
ADDF and Peak ADDF Projections (mgd)

Flow Monitoring Areas	Basin	Phasing	ADDF			Peak ADDF		
			2001	2015	2025	2001	2015	2025
Unmetered	1	Existing	0.449	0.487	0.492	0.651	0.706	0.713
FM1	2	Existing	14.882	17.016	19.459	19.138	21.883	25.024
FM1	9	Existing	0.848	0.893	0.940	1.091	1.148	1.209
FM1	17.1	Existing	0.116	0.168	0.174	0.149	0.216	0.224
FM1	18.1	Existing	0.055	0.055	0.055	0.071	0.071	0.071
FM2	10	Existing	5.378	6.756	7.598	6.572	8.256	9.285
FM2	11	Existing	1.052	1.075	1.078	1.286	1.314	1.317
FM2	12	Existing	0.437	0.495	0.502	0.534	0.605	0.613
FM2	13	Existing	0.324	0.461	0.479	0.396	0.563	0.585
FM3	3	Existing	13.767	15.826	18.259	20.637	23.723	27.370
FM4	7.1	Existing	1.598	1.871	1.906	3.210	3.759	3.829
FM4	14	Existing	0.147	0.868	1.198	0.295	1.744	2.407
FM4	15	Existing	0.043	0.207	0.656	0.086	0.416	1.318
FM5	4	Existing	12.287	14.301	16.687	15.064	17.533	20.458
FM5	8	Existing	0.965	1.075	1.089	1.183	1.318	1.335
FM6	5	Existing	4.478	5.004	6.534	5.696	6.365	8.311
FM6	6.1	Existing	2.958	3.365	3.465	3.763	4.280	4.407
FM7	6.2	Existing	1.063	1.456	1.554	1.543	2.114	2.256
FM7	7.2	Existing	0.677	0.722	0.728	0.983	1.048	1.057
FM7	16	Existing	0.102	0.238	0.303	0.148	0.346	0.440
Growth Area	17.2	2015	0.000	0.151	0.170	0.000	0.219	0.247
Growth Area	18.2	2015	0.000	0.593	0.774	0.000	0.860	1.122
Growth Area	19.1	2015	0.000	1.069	1.478	0.000	1.550	2.143
Growth Area	19.2	2015	0.000	0.051	0.086	0.000	0.074	0.125
Growth Area	20	2015	0.000	0.605	1.087	0.000	0.877	1.576
Growth Area	21	2015	0.000	0.243	0.360	0.000	0.352	0.522
Growth Area	22	2015	0.000	0.117	0.132	0.000	0.170	0.191
Growth Area	23	2025	0.000	0.000	0.300	0.000	0.000	0.435
Growth Area	25	2015	0.000	0.195	0.397	0.000	0.283	0.576
Growth Area	26	2025	0.000	0.000	0.616	0.000	0.000	0.893
Growth Area	27	2025	0.000	0.000	0.323	0.000	0.000	0.468
Growth Area	28	2025	0.000	0.000	0.371	0.000	0.000	0.538
Growth Area	29	2025	0.000	0.000	0.114	0.000	0.000	0.165
Growth Area	26EX	Existing	0.022	0.024	0.024	0.032	0.035	0.035
Facility Totals:								
Brandon LS			14.882	17.016	19.459	19.121	29.088	38.073
WWTP GLS			0.000	1.264	1.875	0.000	7.446	9.555
WWTP			14.882	18.280	21.334	19.121	36.534	47.628

3.7 Infiltration Projections

Infiltration Projections were developed in a similar manner to ADDF projections. Projections were developed by multiplying projected developed acres for each drainage subbasin, by the area unit flow rate determined from flow processing. The same associations between drainage subbasins and monitored subsystems were used to assign unit flow rates.

Table 3-23 presents projected infiltration for each planning year by drainage basin. The sum of infiltration and Peak ADDF is also presented. Infiltration-plus-Peak ADDF represents the peak flows that will occur in the collection system on a regular basis. The phenomenon of peak attenuation also applies to the sum of infiltration and Peak ADDF. For this reason, system totals for basin infiltration-plus-Peak ADDF projections are not presented.

Table 3-23
Infiltration and Infiltration-plus-Peak ADDF Projections (mgd)

Flow Monitoring Areas	Basin	Phasing	TI 2001	TI 2015	TI 2025	Peak ADDF+TI 2001	Peak ADDF+TI 2015	Peak ADDF+TI 2025
Unmetered	1	Existing	0.124	0.135	0.136	0.775	0.841	0.849
FM1	2	Existing	7.805	8.814	10.552	26.943	30.697	35.576
FM3	3	Existing	6.789	7.738	9.469	27.426	31.461	36.839
FM5	4	Existing	5.099	5.989	7.655	20.163	23.522	28.113
FM6	5	Existing	2.337	2.688	3.806	8.033	9.053	12.117
FM6	6.1	Existing	1.632	1.927	2.000	5.395	6.207	6.407
FM7	6.2	Existing	0.754	1.043	1.114	2.297	3.157	3.370
FM4	7.1	Existing	0.785	0.908	0.923	3.995	4.667	4.752
FM7	7.2	Existing	0.472	0.505	0.509	1.455	1.553	1.566
FM5	8	Existing	0.547	0.609	0.617	1.730	1.927	1.952
FM1	9	Existing	1.132	1.192	1.256	2.223	2.340	2.465
FM2	10	Existing	1.383	1.859	2.400	7.955	10.115	11.685
FM2	11	Existing	0.145	0.148	0.149	1.431	1.462	1.466
FM2	12	Existing	0.060	0.068	0.069	0.594	0.673	0.682
FM2	13	Existing	0.085	0.181	0.193	0.481	0.744	0.778
FM4	14	Existing	0.030	0.156	0.338	0.325	1.900	2.745
FM4	15	Existing	0.031	0.151	0.480	0.117	0.567	1.798
FM7	16	Existing	0.074	0.174	0.222	0.222	0.520	0.662
FM1	17.1	Existing	0.155	0.224	0.233	0.304	0.440	0.457
Growth Area	17.2	2015	0.000	0.111	0.125	0.000	0.330	0.372
FM1	18.1	Existing	0.074	0.074	0.074	0.145	0.145	0.145
Growth Area	18.2	2015	0.000	0.444	0.578	0.000	1.304	1.700
Growth Area	19.1	2015	0.000	0.794	1.095	0.000	2.344	3.238
Growth Area	19.2	2015	0.000	0.038	0.063	0.000	0.112	0.188
Growth Area	20	2015	0.000	0.453	0.808	0.000	1.330	2.384
Growth Area	21	2015	0.000	0.178	0.264	0.000	0.530	0.786
Growth Area	22	2015	0.000	0.086	0.097	0.000	0.256	0.288
Growth Area	23	2025	0.000	0.000	0.220	0.000	0.000	0.655
Growth Area	25	2015	0.000	0.143	0.291	0.000	0.426	0.867
Growth Area	26	2025	0.000	0.000	0.450	0.000	0.000	1.343
Growth Area	26EX	Existing	0.010	0.011	0.011	0.042	0.046	0.046
Growth Area	27	2025	0.000	0.000	0.237	0.000	0.000	0.705
Growth Area	28	2025	0.000	0.000	0.273	0.000	0.000	0.811
Growth Area	29	2025	0.000	0.000	0.083	0.000	0.000	0.248
Facility Totals:								
Brandon LS			7.805	8.814	10.552	26.926	37.902	48.625
WWTP GLS			0.000	0.937	1.386	0.000	8.383	10.941
WWTP			7.805	9.751	11.938	26.926	46.285	59.566

3.8 Inflow and Peak Storm Flow Projections

Inflow projections were derived from projected developed acres and inflow coefficients determined during flow analysis. The product of the inflow coefficient and developed acres were calculated at the drainage subbasin level in order to accurately allocate inflow to drainage basins that were split amongst monitored systems.

A 25-year return period storm was selected as the design rainfall event as was used in the 1990 Facility Plan. Rainfall intensities were selected from the 25-year rainfall curve by first calculating a time of concentration for each drainage basin and subbasin. The time of concentration calculation was based on an empirical equation involving developed area. Separate times of concentration were calculated for basins and subbasins to allow direct calculation of inflow at the basin level to accurately present peak cumulative flows. For this same reason, inflow and peak storm flow system totals are not presented in the Table 3-24.

Table 3-24 presents inflow projections and peak storm flow projections by drainage basin. Peak storm flow is the sum of Peak ADDF, I/I. Peak storm flow represents the combination of wastewater flows components for a given planning year and design storm. Peak storm flow is thus the design capacity at planning stage.

Subbasin flow projection details are provided in Appendix D. Appendix D is arranged by planning year and not by flow component.

Table 3-24
Infiltration and Peak ADDF Projections (mgd)

Flow Monitoring Areas	Basin	Phasing	Inflow 2001	Inflow 2015	Inflow 2025	Peak Flow 2001	Peak Flow 2015	Peak Flow 2025
Unmetered	1	Existing	4.537	4.924	4.973	5.239	5.686	5.743
FM1	2	Existing	65.344	65.876	75.953	92.292	96.578	111.535
FM3	3	Existing	59.882	60.679	70.717	87.038	91.830	107.199
FM5	4	Existing	47.139	49.086	60.075	67.316	72.624	88.207
FM6	5	Existing	20.981	22.147	29.247	29.016	31.202	41.367
FM6	6.1	Existing	16.353	17.520	18.193	21.747	23.727	24.600
FM7	6.2	Existing	9.881	12.004	12.824	12.179	15.161	16.194
FM4	7.1	Existing	9.208	10.652	10.834	13.123	15.225	15.490
FM7	7.2	Existing	7.210	6.603	6.657	8.703	8.197	8.264
FM5	8	Existing	6.907	6.586	6.670	8.637	8.513	8.622
FM1	9	Existing	9.177	9.663	8.954	11.400	12.004	11.419
FM2	10	Existing	25.886	30.398	32.634	33.839	40.511	44.316
FM2	11	Existing	8.062	8.242	8.264	9.493	9.704	9.731
FM2	12	Existing	4.757	4.429	4.494	5.351	5.102	5.177
FM2	13	Existing	3.708	4.567	4.758	4.238	5.381	5.609
FM4	14	Existing	2.040	6.769	9.732	2.258	8.036	11.604
FM4	15	Existing	0.853	2.809	6.271	0.946	3.261	7.704
FM7	16	Existing	1.745	3.253	4.139	1.967	3.772	4.801
FM1	17.1	Existing	2.213	2.581	2.682	2.517	3.021	3.139
Growth Area	17.2	2015	0.000	2.557	2.319	0.000	2.887	2.691
FM1	18.1	Existing	1.054	1.054	1.054	1.199	1.199	1.199
Growth Area	18.2	2015	0.000	6.669	7.430	0.000	7.973	9.131
Growth Area	19.1	2015	0.000	9.034	12.471	0.000	11.381	15.713
Growth Area	19.2	2015	0.000	1.034	1.461	0.000	1.146	1.649
Growth Area	20	2015	0.000	6.807	9.178	0.000	8.140	11.567
Growth Area	21	2015	0.000	3.312	4.027	0.000	3.843	4.814
Growth Area	22	2015	0.000	1.972	2.219	0.000	2.228	2.508
Growth Area	23	2025	0.000	0.000	4.078	0.000	0.000	4.734
Growth Area	25	2015	0.000	2.658	4.445	0.000	3.084	5.312
Growth Area	26	2025	0.000	0.000	6.857	0.000	0.000	8.202
Growth Area	26EX	Existing	0.366	0.287	0.290	0.404	0.329	0.332
Growth Area	27	2025	0.000	0.000	4.392	0.000	0.000	5.098
Growth Area	28	2025	0.000	0.000	4.154	0.000	0.000	4.966
Growth Area	29	2025	0.000	0.000	1.916	0.000	0.000	2.165
Facility Totals:								
Brandon LS			65.344	65.876	75.953	92.292	96.578	111.535
WWTP GLS			0.000	10.679	14.196	0.000	13.451	18.305
WWTP			65.344	71.2	83.84	92.74	105.016	123.863

4.0 Existing Wastewater System Facilities

4.0 Existing Wastewater System Facilities

This chapter describes the existing sanitary sewer system facilities and the inventory of facilities created for the purpose of computer modeling. This includes basis of data used and lists significant components of the system.

4.1 Collection System

4.1.1 Basis of Developing Data for Model

4.1.1.1 Drainage Basins

Drainage basins or watersheds define the areas within which flows can be collected and conveyed by gravity. The 1990 Wastewater Collection System Facilities Plan identified 13 major drainage basins within the existing sanitary sewer system. These 13 drainage basins were divided into subbasins and modified to account for current land uses and sanitary system characteristics. Since the 1990 Plan, 16 additional basins have been defined to accommodate recent growth areas and projected future growth areas around the periphery of the City. The 29 basins defined for existing and future development have been further divided into 184 subbasins. Subbasin 24 which was comprised of four subbasins was not included in the hydraulic analyses.

The topography of the study area has considerable change in elevation. The elevation difference between the highest sewered ground and the WWTP is approximately 300 feet. The highest elevations are located on the west side of the planning area. The steepest slopes are along the embankments surrounding the Big Sioux River. The total area currently sewered is approximately 42,000 acres.

4.1.1.2 Sanitary Sewer Inventory

An inventory and definition of the existing sewer system was compiled as part of this project. The facility inventory considered previous reports, City records, the City's GIS system, and as-built drawings for new sewers. Additional data at selected locations was requested from the City to complete the inventory.

4.1.1.2.1 City Geographic Information System (GIS) Source Information

The primary source of information for the sanitary sewers was the City's GIS system. The City provided Black & Veatch with GIS files in Shapefile format containing the complete sanitary sewer collection system. The City prepared the files to include information from the ArcINFO mapping system and RJN maintenance management system. The City's information was converted to the files listed in Table 4-1 in preparation for model construction.

Table 4-1
Geographical Information System Files

File Name	Description
smain.shp	Sanitary sewers
future.shp	Future sewers
sanstruc.shp	structures (pump stations, manholes, etc.)
forcemain.shp	Force mains
ps.shp	Pump stations
sbasin.shp	drainage basin and subbasin boundaries

The City provided Black & Veatch with supplemental GIS files, such as topographical data and streets, which provided further supporting information and background maps for study exhibits.

4.1.1.2.2 Manhole and Sewer Information

The City of Sioux Falls GIS provides a comprehensive manhole and sewer inventory. Most manholes are identified using a 7-character string (e.g. 06G0001). The first two characters of the string identify the major basin number (e.g. 06), while the third character designates the subbasin within the major basin (e.g. G). The remaining four characters are the manhole number. Some manhole identifiers are up to nine characters due to subbasins that have been added since the original subbasin numbering.

The XP-SWMM model allows up to 10 characters for pipe names. The pipes are named within the model using the convention of the upstream manhole plus the characters ".1" or ".2". Therefore, the model limited some manhole names to eight characters.

The original GIS network file, obtained from the City, contained approximately 12,500 sanitary sewer segments. GIS pre-processing work was performed to provide a network that was continuous and that excluded unnecessary data. GIS records that were missing manhole numbers or were defined as stub-outs were not included. GIS records that consisted of several segments in series were merged into one sewer segment record. GIS records for future sewers were not included in the existing sewer system. After GIS processing, there are 11,777 total gravity sewer records and 27 force mains.

The complete system inventory was imported into Black & Veatch's Sanitary Sewer Management System (SSMS). Black & Veatch used SSMS to maintain the collection system inventory, to develop the model, and to aid in analysis of the model results. The presence of the complete inventory in SSMS will facilitate future analyses by the City of additional sewers beyond the trunk lines.

The lines to be modeled are tagged within SSMS files so that models can be constructed by query. A total of 2,559 sewer lines were identified for the trunk sewer model, comprised of the pipes 10 inches or greater in diameter, force mains, and smaller lines where necessary to provide a continuous model.

A significant number of pipe records did not have essential information (e.g. diameter or invert elevations) when provided by the City. Black & Veatch identified missing data for 768 modeled pipes and manholes. The City provided data at 530 of these locations. A total of 95 new pipes were digitized in basins 14A-D. For the remaining modeled pipes and manholes, values were interpolated using existing data. For example, a missing pipe diameter was estimated to be the same diameter as that of the upstream pipe. Similarly, invert elevations were calculated using known elevations, lengths and estimated slopes. Missing rim elevations were assumed to be 15 feet above the pipe invert. For the remainder of the system, missing data research is beyond the scope of this project. It will be possible to update diameter and invert information in SSMS when more information becomes available in the future.

All elevations were converted to the USGS datum prior to their inclusion in SSMS. The existing information was in the City datum to which 1309.18 feet was added to convert to the USGS datum.

4.1.2 System Component Data

4.1.2.1 Flow Diversions

Locations in the collection system where the flow splits into two downstream sewers are modeled in the SSMS/XP-SWMM system as diversion structures. During a dynamic analysis, XP-SWMM determines the flow split between the sewers based on head differentials in each direction. Diversions located on the modeled trunk sewer lines are listed in Table 4-2.

Table 4-2
Modeled Diversions

Number	Upstream MH	Downstream MH	Diameter (in)
1	04E0008	BV-4704	10
		04E0007	10
4	08C0005	08C0004	18
		06AB004A	10
5	08E0007	06DA007	15
		BV-8608	21
6	08G0003	06EA014	12
		08G0002	21
7	10H0001	10E0016	20
		10D0016	30
8	10HA006	10HA005	12
		10HA007	12
9	20A0007F	20A0007E	8
		05EG011C	10
10	BV-1223	03C0004A	15
		BV-1225	15
11	BV-1225	03CA002 BV-1175	15 15
12	BV-4614	11AK001 11E0010	12 42
13	16AB001	06A0007 06AA005	42 15
14	BV-8787	05EG008 05EH005	10 12
15	BV-9687	07FB008 07B0008	15 15
16	03A0013	0340012 EQBASIN	66 36

4.1.2.2 Wastewater Pumping Stations and Force Mains

There are 27 active pump stations that were considered during modeling. Available pump station information is presented in Tables 4-3 and 4-4. This data was obtained through queries of existing data from previous reports and surveyed data gathered by the City.

Table 4-3
Modeled Pump Station Inventory

Number (ft.)	Name	Address	Number of Pumps	Head	Test Flow (mgd)	Firm Capacity (mgd)	Status
Brandon	Brandon	3300 E. Rice Street	4	132	13.54	40.61	Modeled
TEMP_PS1	LaMesa	LaMesa and 12th Street					Calibration Only
PS201	2nd & Brookings	1000 Blk N. 2nd	2	50	0.24	0.24	Not Modeled
PS202	Air Terminal	South End of Costello Terminal	2		0.46	0.46	Modeled
PS203	Cherokee & "C"	Cherokee and C Avenue	3	56	1.01	2.02	Modeled
PS204	Modern Press	806 N. West Avenue	2	30	1.07	1.07	Modeled
PS205	6th & Hawthorne	6th & Hawthorne, 300 Blk N.	2	19	0.41	0.41	Modeled
PS206	Burnside	1800 Burnside	2	23	0.84	0.84	Modeled
PS207	Ramada Inn	2902 W. Russell	2	45	1.17	1.17	Abandoned 2015
PS208	Rice & Kiwanis	1400 N. Kiwanis	2	21	0.20	0.20	Modeled
PS209	9th & Kiwanis	101 N. Kiwanis	3	80	2.05	4.11	Modeled
PS210	Skunk Creek	6700 Block W. 12th St.	2	15	0.33	0.33	Calibration Only
PS212	Westward Ho	3100 – 3110 Sherman Park	2		0.46	0.46	Not Modeled
PS213	23rd & Kiwanis	1421 S. Kiwanis	2	31	0.24	0.24	Not Modeled
PS214	River Run	616 S. Lyons	2	46	0.68	0.68	Calibration Only
PS215	Sioux River North	3301 W. 12th St.	4	66	4.80	14.40	Modeled
PS216	Summerhill South	4813 S. Sycamore	1	100	0.95	0.95	Abandoned 2025
PS217	26th & Dubuque	5211 E. 26th St.	2	78	0.46	0.46	Abandoned 2015
PS218	Tuthill Park	3500 S. Blauvelt	4	30	5.04	15.12	Modeled
PS219	Haley& Bailey	1231 N. Haley Ave.					Not Modeled
PS220	Rock Island	1260 S. Blauvelt	2	70	0.56	0.56	Not Modeled
PS221	Madison & Vail	1116 N. Sycamore	2	45	0.14	0.14	Not Modeled
PS222	Rice St. LS	2800 Block of Rice St.					Not Modeled
PS224	50th Street North	50th Street North	2	27	1.09	1.09	Modeled
PS225	40th Street North	210 E. 40th Street North	2	25	0.17	0.17	Not Modeled
PS227	Highway 38A LS	201 Powderhouse Road	2	130	1.08	1.08	Abandoned 2015

Table 4-3
Modeled Pump Station Inventory

Number (ft.)	Name	Address	Number of Pumps	Head	Test Flow (mgd)	Firm Capacity (mgd)	Status
PS228	Arena LS	1201 Northwest Ave.					Not Modeled
PS233	Renner #1	N. of 72nd St.	2		0.52	0.52	Modeled
PS234	Renner #2	N. of 72nd St.	2		0.21	0.21	Modeled
PS235	Renner #3	47492 Berry Lane	2		0.21	0.21	Modeled
PS236	Renner #4	25775 Lindburg Ave.	2		0.12	0.12	Modeled
PS237	Renner #5	47419 258th St.	2		0.08	0.08	Modeled
GLS_WWTP		E. of Big Sioux R. near WWTP	3				Modeled
G20_LS		41st St. E. of Six Mile Rd.					Proposed
G26_LS		57th St. E. of Six Mile Rd.					Proposed
G27_LS		S. of 85th St. at Six Mile					Proposed
G28_LS		S. of 85th and E. of Southeastern Ave.					Proposed
G29_LS		69th St. E. of Six Mile Rd.					Proposed

Table 4-4 shows the modeled pump controls. A detailed definition of pump stations, including the pump curves, control settings, wet wells dimensions, and down stream force mains, needed for the dynamic model. The following default values were used when information was unavailable.

- Number of pumps = 2
- Wet well diameter: estimated from capacity
- Wet well depth = 20 feet
- Pump 1 start point = influent invert elevation or 3 feet from the wet well bottom
- Pump 2 start point = 1 foot above pump 1 on elevation
- Pump stop point = 1 foot above wet well bottom elevation
- System flow/head point: enough to pump estimated peak flow (will vary)

Table 4-4
Wet Well Summary by Pump Station

Station Number	WW Width or Diameter (ft)	WW Length (ft)	WW Depth (ft)	Pump 1 On Elevation (ft)	Pump 1 Off Elevation (ft)	Pump 2 On Elevation (ft)	Pump 2 Off Elevation (ft)
201	6.0	NA	20.0	1,311.94	1,310.94	1,312.94	1,310.94
202	4.5	7.5	18.7	1,408.38	1,407.08	1,408.88	1,407.08
203	9.5	20.0	21.3	1,399.68	1,398.58	1,400.08	1,398.58
204	3.0	11.0	16.6	1,401.01	1,398.21	1,401.41	1,398.61
205	4.0	10.0	13.5	1,408.92	1,408.02	1,409.62	1,408.02
206	5.3	12.0	19.4	1,406.18	1,404.58	1,407.08	1,404.58
207	6.0	9.0	16.3	1,408.74	1,407.44	1,409.07	1,407.44
208	5.5	8.5	13.3	1,408.04	1,406.84	1,408.44	1,407.54
209	6.0	NA	20.0	1,394.43	1,393.43	1,395.43	1,394.93
210	6.0	NA	19.4	1,417.65	1,415.65	1,418.15	1,415.65
212	6.0	NA	20.0	1,421.10	1,420.10	1,422.10	1,420.10
213	4.0	8.0	13.1	1,446.02	1,445.02	1,446.52	1,445.52
214	10.0	NA	28.5	1,391.18	1,389.18	1,392.18	1,389.18
215	26.0	30.0	43.5	1,382.50	1,381.50	1,383.50	1,381.50
216	10.0	NA	19.3	1,429.28	1,427.18	1,429.58	1,427.18
217	6.0	NA	22.6	1,488.36	1,492.86	1,489.06	1,492.86
218	7.7	38.0	11.8	1,373.43	1,372.43	1,374.43	1,372.43
220	5.0	14.0	20.5	1,381.20	1,379.10	1,382.00	1,379.10
221	7.0	NA	14.7	1,471.18	1,470.08	1,473.68	1,470.38
224	8.0	NA	29.0	1,403.18	1,400.38	1,404.08	1,400.38
225	8.0	NA	19.1	1,408.42	1,407.42	1,409.13	1,407.62
227	10.0	NA	28.0	1,409.30	1,407.50	1,410.00	1,408.00
233	6.0	NA	25.7	1,413.24	1,412.24	1,414.24	1,412.24
234	6.0	NA	27.7	429.68	1428.68	1431.68	1428.68
235	6.0	NA	20.0	1,423.00	1,422.00	1,424.00	1,422.00
236	6.0	NA	20.0	1,427.62	1,426.62	1,428.62	1,426.62
237	6.0	NA	20.0	1,426.00	1,425.00	1,427.00	1,425.00
Brandon	12.0	53.3	39.0	VFD pumps op. to maintain ww level at 5 feet			

Table 4-5 presents force main information associated with each pump station.

Table 4-5
Modeled Force Main Inventory

Number	Name	Diameter (in)	Length (ft)
201	2nd & Brookings	4	398
202	Air Terminal	6	2,795
203	Cherokee & "C"	12	4,970
204	Modern Press	6	658
205	6th & Hawthorne	6	364
206	Burnside	8	437
207	Ramada Inn	12	4,710
208	Rice & Kiwanis	8	715
209	9th & Kiwanis	8	3,336
210	Skunk Creek	6	66
212	Westward Ho	4	1,354
213	23rd & Kiwanis	8	483
214	River Run	8	2,646
215	Sioux River North	36	18,975
216	Summerhill South	8	3,804
217	26th & Dubuque	8	2,642
218	Tuthill Park	36	589
219	Haley and Bailey	4	370
220	Rock Island	6	1,491
221	Madison & Vail	4	428
222	Rice & Cleveland	8	1,064
224	50th Street North	10	1,103
225	40th Street North	4	587
227	Highway 38A LS	8	4,391
233	Renner #1	8	3,968
234	Renner #2	6	2,732
235	Renner #3	8	4,921
236	Renner #4	8	1,787
237	Renner #5	8	2,632
Brandon	Brandon	36	12,257
Total			**83,941**

4.1.2.3 Siphons

There are nine siphons in the collection system for which the City provided as-built drawings. Six of these are included in the modeled inventory and were modeled in XP-SWMM. Table 4-6 presents the size, length and the number of barrels for modeled siphons.

Table 4-6
Siphon Inventory

Number	Upstream MH	Downstream MH	Pipe 1 (in)	Pipe 2 (in)	Pipe 3 (in)	Length (ft)	Modeled
1	03A0005	BV-1001	36	30	24	686	Y
2	04G0001	04H0005	8			225	Y
3	04JA001	04J0001	24			220	Y
4	06AA008	06AA007	6	6		191	N
5	06B0005	06B0004	8	8		240	Y
6	06CB002	06CB001	12			345	N
7	06EB003	06EB001	8			325	N
8	08A0004	08A0003	14			275	Y
9	17A0001A	BV-963	10	8	4	375	Y

4.2 Flow Equalization Facility

The wet-weather flow equalization facility recommended in the 1993 Report on Wastewater Flow Equalization was constructed. The facility is located upstream of the Brandon pumping station and is used to handle peak flows in excess of the Brandon station capacity. The equalization facility consists of two cells providing a total volume of about 12 million gallons. There is a 1 million gallon clarifier located in one of the cells. Flows are directed to and from the facility by gravity flow, which is based on valve adjustments at a manhole outside the facility. This facility was modeled as a storage node in the XP-SWMM model. The model's real time control (RTC) options can be utilized in optimizing the storage in this facility in handling peak flows under alternatives analyses.

4.3 Wastewater Treatment Facility

The WWTP is located on the north side of the Big Sioux River, east of Sycamore Street and south of East 60th Street. At the time of the 1990 and 1993 reports, the plant's design capacities were 13.43 mgd for average daily flow and 27.0 mgd peak flow. Facility improvements since 1993 have increased the plant's average treatment capacity to 18.7 mgd.

Hydraulic profiles were developed for the 1993 report to determine the peak hydraulic capacity of the treatment facilities, which include the Brandon pumping station, the WWTP, and the Transfer pumping station. With the pumping station modifications recommended in that report, the current capacities of the treatment facilities are as follows: maximum monthly flow of 25 mgd, a maximum daily flow of 27.5 to 30 mgd, and peak hourly flow of 35 mgd.

5.0 Trunk Sewer Model Development

5.0 Trunk Sewer Model Development

5.1 Hydraulic Model Software Basis

5.1.1 Sanitary Sewer Management System

SSMS is Black & Veatch's standard proprietary system that is routinely used in master planning studies of wastewater collections systems. SSMS is a fully functional relational database management system (RDBMS) that is programmed in FoxPro and runs in standard windows environment. The SSMS database can incorporate inspection, scheduling, and maintenance information in addition to the collection system inventory. It has a complete pre- and post-processing model interface with three different models including XP-SWMM. SSMS supports traditional database functions through its ability to query the underlying database tables and generate custom reports as well as import and export database files in standard formats (e.g. *.xls, *.dbf, *.txt). Additionally, SSMS allows the user to view network information in a graphical format, similar to ArcView. Users can view shape files in conjunction with the network inventory and color code by fields of interest.

The SSMS database will be used for all subsequent modifications in the inventory. Black & Veatch will maintain a list of network modifications that will be provided to the City's GIS staff for corresponding updates in their system (See Appendix F).

5.1.2 XP-SWMM

This section provides an overview of the modeling software, XP-SWMM, selected for this project. XP-SWMM is a fully dynamic model developed by XP Software. A dynamic hydraulic model is a mathematical representation of the sewer system depicted by a series of nodes and links. Nodes represent manholes, storage basins, wet wells, junction boxes, and outfalls. Links, as the name implies, represent any hydraulic structure connecting two nodes. Pipes, pump stations, weirs, and gates are all represented by links in a model.

Hydraulic heads are computed at the nodes and flows through the links, conserving mass and momentum. These hydraulic calculations enable the user to query hydraulic grades at nodes and velocities/flows in links to evaluate the hydraulic capacity of a sewer system under various hydraulic and hydrologic scenarios. Hydraulic scenarios may include flow diversions, parallel pipes, replacement pipes, storage basins, and various other RTC or operational changes. Hydrologic scenarios include different design storm conditions and/or rainfall distribution types, for example, a 10-year recurrence interval design storm following an SCS Type II distribution. Hydrographs can be generated externally and directly loaded into the model or can be generated by the model given the appropriate data. Data required by the model for hydrograph operations include: basin area broken down by percent of impervious surface, time of concentration, and various infiltration/inflow factors depending upon the method used, for example, curve numbers and shape factors are required when using the SCS Hydrology method.

5.2 Trunk Sewer Model Inventory

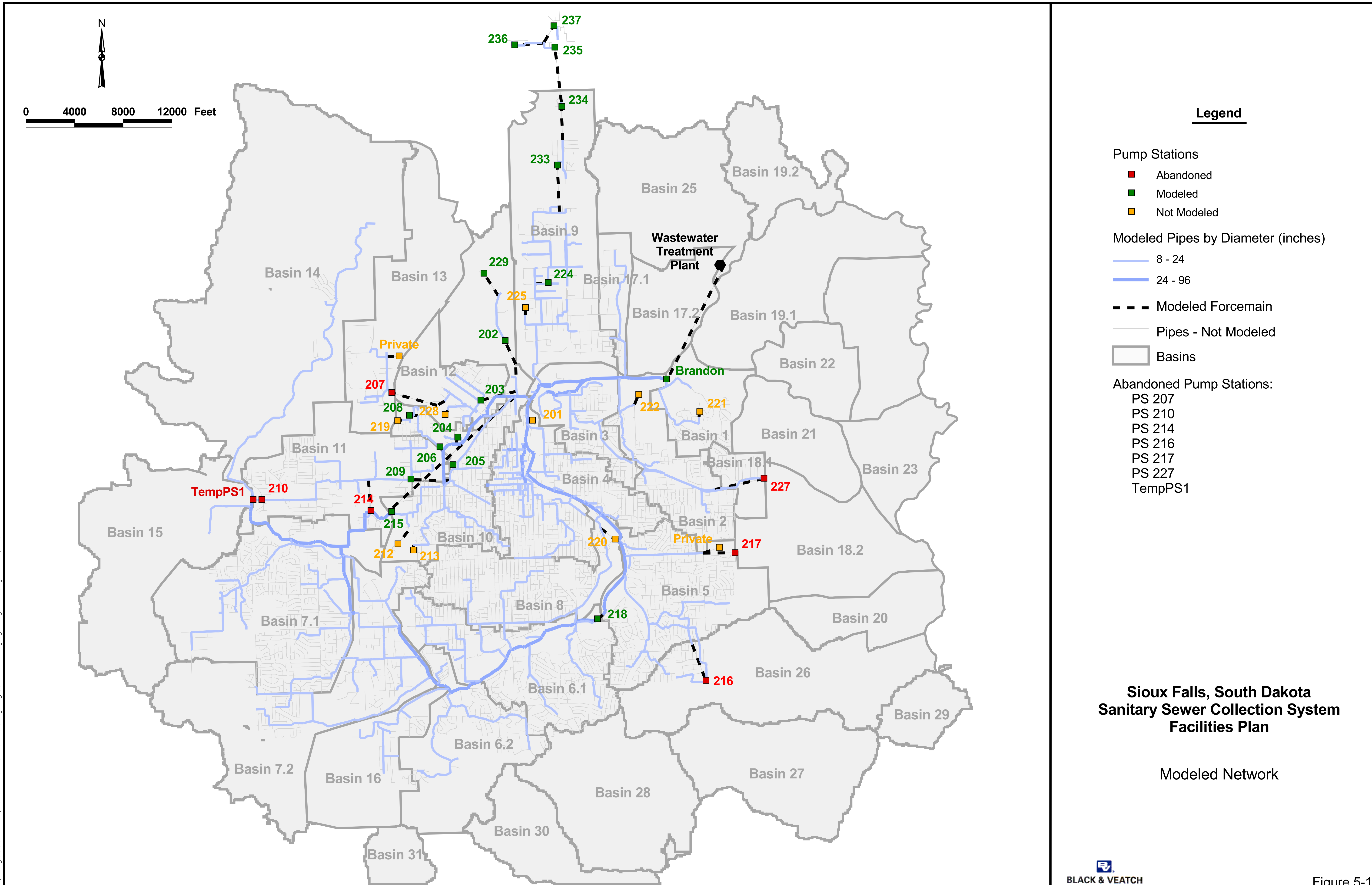
The network inventory used for the XP-SWMM model is a subset of the overall network inventory. In general, all pipes greater than 10 inches in diameter were selected. However, additional pipes were also selected to preserve the connectivity of the system. Modeled gravity pipe inventory data is presented in Table 5-1. Table 5-2 lists the modeled force main and siphon lengths by pipe size. Overall, 2,559 pipes were selected. The total modeled sewer length is approximately 144 miles, not including the force mains. Figure 5-1 shows the modeled pipe network. See Appendix K for a map showing project locations and manhole references.

Table 5-1
Modeled Gravity Pipe Inventory

Pipe Diameter (in)	Total Length (ft)	Un-modeled Length (ft)	Modeled Length (ft)	No. of Modeled Pipes
N.A.	463,543	463,543		
< 6	245	245	0	0
6	44,891	43,936	955	4
8	1,935,930	1,863,245	72,685	272
10	170,581	241	170,340	615
12	110,944	300	110,644	384
14	676	0	676	2
15	116,656	1,051	115,605	357
16	1,543	0	1,543	9
18	64,116	350	63,766	222
20	4,338	0	4,338	14
21	51,595	0	51,595	162
24	65,826	0	65,826	207
30	17,729	0	17,729	52
32	414	0	414	2
36	30,370	0	30,370	90
40	1,965	0	1,965	11
42	30,878	553	30,325	89
48	10,545	0	10,545	34
60	3,182	0	3,182	12
66	10,077	0	10,077	21
Total	**3,136,044**	**2,373,464**	**762,580**	**2,559**

Table 5-2
Modeled Force Main and Siphon Inventory

Pipe Diameters (in.)	Force Main Length (ft.)	Siphon Length (ft.)	Total Length (ft.)
< 6	30		30
6	1,076		1,816
8	2,754	465	107,535
10	25	375	169,459
12	604		112,841
14		275	684
15	5		103,677
18	5		61,110
24	5	220	65,227
36	5	691	26,832
42	5		34,892
66	5		9,641
Total	**4,519**	**2,026**	**786,626**


N
0 4000 8000 12000 Feet
Legend
Pump Stations
Abandoned
Modeled
Not Modeled
Modeled Pipes by Diameter (inches)
8 - 24
24 - 96
Modeled Forcemain
Pipes - Not Modeled
Basins
Abandoned Pump Stations:
PS 207
PS 210
PS 214
PS 216
PS 217
PS 227
TempPS1
Wastewater Treatment Plant
Brandon
Private
TempPS1
201
202
203
204
205
206
207
208
209
210
212
213
214
215
216
217
218
219
220
221
222
224
225
227
228
229
233
234
235
236
237
Basin 1
Basin 2
Basin 3
Basin 4
Basin 5
Basin 6.1
Basin 6.2
Basin 7.1
Basin 7.2
Basin 8
Basin 9
Basin 10
Basin 11
Basin 12
Basin 13
Basin 14
Basin 15
Basin 16
Basin 17.1
Basin 17.2
Basin 18.1
Basin 18.2
Basin 19.1
Basin 19.2
Basin 20
Basin 21
Basin 22
Basin 23
Basin 25
Basin 26
Basin 27
Basin 28
Basin 29
Basin 30
Basin 31
Sioux Falls, South Dakota
Sanitary Sewer Collection System
Facilities Plan
Modeled Network
BLACK & VEATCH Corporation
Figure 5-1

5.3 Design Criteria Review

Design criteria provide standards for evaluating an existing system or designing improvements. The City's design criteria generally conform to the 10-States Standards, and are documented in Chapter 9 of the "Engineering Design Standards for Public Improvements for the City of Sioux Falls". These design criteria are summarized for use in the planning effort.

5.3.1 Sanitary Sewer Design Criteria

- Manning's roughness coefficient = 0.013;
- maximum and minimum velocities = 14 fps and 2 fps, respectively;
- minimum gravity sanitary sewer size = 8 inch (not including laterals);
- depth of sewer = 7 feet (where practical);
- minimum manhole diameter = 48 inch;
- minimum manhole spacing = 400 feet (8" to 15" sewers), or 500 feet (18" to 30" sewers), or 650 feet (30" and larger sewers);
- minimum grade = sufficient to maintain 2 fps velocity;
- minimum manhole drop = 0.10 (where there is no change in pipe size), or where there is a change in pipe size, match 0.8 depth point of all lines as a minimum, and match pipe crowns whenever possible; and
- maximum manhole drop = 1.5 feet as measured from invert to invert.

5.3.2 Pumping Station Design Criteria

- Minimum number of pumps = 2, each with capacity to pump peak design flow;
- No submersible pumps allowed;
- Variable Frequency Drives (VFD) for all motors greater than 30 hp;
- Minimum pump idle time = 30 minutes;
- Effective wet well volume = 2.5 times the pump discharge rate, based on an operating volume to maintain 6 starts per hour (0-25 hp motor), or 5 starts per hour (26-35 hp motor), or 4 starts per hour (36-60 hp motor); and
- Maximum Pump Speed = 1800 rpm.

5.3.3 Design Wastewater Flow Criteria

- Average Daily Flow (Trunk Sewer) = Area x Area Density x Rate, OR
  = Number of Units x Unit Density x Rate
- Peak Trunk Flow = Average Daily Flow x 2.5

Table 5-3
Design Density

Land Use	Area Density[1]	Unit Density	Rate
Low Density Residential	6 units/ac	3 people / unit	100 gpcd
Medium Density Residential	12 units/ac	2 people / unit	100 gpcd
High Density Residential	25 units/ac	2 people / unit	100 gpcd
Office and Institutional		Dependent on Water Use	
Commercial		Dependent on Water Use	
Industrial		Dependent on Water Use	

[1] Area = Gross Area including streets and alleys but excluding parks, school grounds, and similar dedicated open space.

5.4 Model Calibration

5.4.1 Allocate Wastewater Flows

The analysis of flow and rainfall monitoring data and the development of the hydraulic model converge in the wastewater flow allocation process. In order for flows to be applied to the model accurately, the hydraulic network is sub-divided into flow allocation units known as subsystems. The subsystem definitions chosen for this model match with the drainage basins defined by the City of Sioux Falls. Exceptions to this occur where basins were split by flow monitoring areas. In this event, a basin was split into multiple subsystems which correspond to the different flow parameters derived for each area. Appendix D includes tables that define the subsystems in terms of constituent subbasins. The subsystems and basins are shown on Figure 2-1. Within subsystems, flows were distributed evenly to manholes receiving incremental flow. This included all manholes except those assumed to have no service connections on the immediate upstream pipes.

The XP-SWMM model simulates flows from a variety of sources and is capable of modeling all wastewater flow components. ADDF is a constant value for projecting flows in the existing system. ADDF was estimated for each basin based on rates of wastewater production per capita. For projecting flows in growth areas, ADDF was estimated based on rates per developed acre. See Chapter 3 for a detailed description of flow calculation. Infiltration is modeled as a constant value. Direct inflow from stormwater is modeled using the SCS hydrology method. This method simulates stormwater runoff resulting from an input hyetograph. Several parameters are necessary for the runoff simulation. The following list shows the parameters used in XP-SWMM in generating the wastewater flow contributions at each manhole:

- Area in Acres
- ADDF in Gallons Per Day
- Diurnal Curve Pattern for ADDF
- Infiltration Rates in Gallons Per Day
- Percent Impervious
- SCS Curve Number
- Time of concentration
- SCS Curve Number
- Initial Abstraction (losses to groundwater)

Each SCS hydrology parameter serves a different function in determining the output hydrograph from the input hyetograph. SCS curve number in the SCS hydrology methodology the percent impervious, and initial abstraction values control the runoff volume. The time of concentration and SCS shape factor parameters help in calibrating the hydrograph shape, peak flow, and time to peak. Tables showing the flow allocation values used for each parameter are included in Appendix F, titled Flow Loadings for XP-SWMM Model, Calibration Conditions.

5.4.2 Calibration Model

Calibration analyses are performed to establish confidence in the results generated by the hydraulic model. Model calibration consists of adjusting model parameters so that predicted flows match those observed in flow monitoring. Once parameters have been adjusted within reasonable levels, the difference between predicted and observed flows is determined. The closer the agreement between predicted and observed flows, the better the calibration.

Calibration was performed for both dry-weather and wet-weather conditions. Dry-weather conditions included peak flow, peak ADDF, and infiltration. A review of historical WWTP flows indicated seasonal patterns in infiltration. Spring infiltration in Sioux Falls tends to be greater than fall infiltration. Due to the seasonal nature of infiltration variation, the Fall 2001 Flow Monitoring Program did not provide the opportunity to observe peak infiltration conditions. Infiltration values derived based on the Fall 2001 Flow Monitoring Program data were modified to reflect the magnitude of infiltration shown in WWTP flow records.

If flow metering areas (subsystem) are identical to the basins, ADDF and peak infiltration need no adjustments. For this project, each flow monitoring area was composed of several basins (see Appendix D).

ADDF and infiltration values were allocated at the basin level with adjustments to the timing of the diurnal curve to obtain model results at the flow meter locations that agreed within 10 percent with measured flows. Final dry weather calibration results are summarized in Table 5-4.

Table 5-4
Dry Weather Calibration Results

	Peak Flow (cfs)				Average Flow (cfs)			
Monitor	Observed	Modeled	Difference	Percent Difference	Observed	Modeled	Difference	Percent Difference
FM1	34.331	32.900	-1.431	-4.2	28.642	28.839	0.197	0.7
FM2	27.719	27.421	-0.298	-1.1	23.700	23.898	0.198	0.8
FM3	1.492	1.433	-0.059	-4.0	1.166	1.147	-0.019	-1.6
FM4	3.264	3.032	-0.232	-7.1	2.173	2.190	0.017	0.8
FM5	20.436	20.846	0.410	2.0	17.633	17.757	0.124	0.7
FM6	14.000	14.176	0.176	1.3	6.679	6.741	0.062	0.9
FM7	6.679	6.741	0.062	0.9	5.278	5.317	0.039	0.7

Wet-weather calibration simulated flows combining the direct inflow from stormwater runoff as a result of the August 29, 2001 storm with the ADDF and infiltration flows used in dry-weather calibration. In wet-weather calibration initial values for the SCS hydrology parameters used are not calculated directly as a part of flow and rainfall analysis, but are inferred from values determined while using the inflow coefficient method for inflow analysis.

Wet weather calibration was achieved by making a series of model runs. Initially the input area for inflow calibration was set to twice the developed area times the inflow coefficient and the percent impervious was set to 50 percent. In subsequent model runs the percent impervious was adjusted until the model produced a peak flow estimate at each flow monitoring point that was close to the observed peak. Table 5-5 shows the final results of the calibration runs.

Table 5-5
8/29/2001 Storm Event Peak 15-minute Flows (cfs)

Monitor	Observed	Modeled	Difference	Percent Difference (%)
FM1	43.105	43.921	0.816	1.9
FM2	33.040	32.486	-0.554	-1.7
FM3	7.949	8.065	0.116	1.5
FM5	25.873	26.147	0.274	1.1

Percent impervious was not permitted to drop below one percent nor exceed 100 percent for any basins. If the model at any flow monitoring point could not match the observed flow within 10 percent with percent impervious 100 percent or less, then adjustment was made to the developed area. Basins 3 and 4 required a value of 100 percent impervious and area adjustments to achieve calibration.

Final parameter values determined during calibration are carried forward for use in capacity analysis modeling. Final parameter values for this project are included in the Calibration Conditions table in the Flow Loading for XP-SWMM Model section of Appendix F. Hydrographs plotting predicted versus observed flow are included in Appendix F.

5.4.3 Additional/Design Storm Calibration

The storm used for calibration was estimated to be about 60 percent of a 1 year storm. The selected design storm was the 25 year storm event, which for Sioux Falls, has a peak intensity 202 times greater than the 1 year storm. Unless percent impervious is set to 100 percent the SCS method of inflow calculation does not reliably predict inflow for storms of magnitude greater than the storm used for calibration. An additional calibration for the design storm event was required and is documented in Appendix F.

The design storm calibration was achieved by comparing model peak flow results using the developed areas and percent impervious values obtained in the wet weather calibration to peak flow values estimated as described in Chapter 3, Section 3.8, Inflow and Peak Storm Flow Projections.

Input areas were adjusted on a basin basis until model peak flow results matched projected values. Final calibration results were summarized in Table 5-6.

Table 5-6
Model Peak Flow Results For Design Year 2025
and the 25-year Storm Event

Basin Number	Manhole Number	Equalization Basin Flow (mgd)	25-yr Storm Flow (mgd)	Peak Flow Projections (mgd)
1.0	01A0001		12.236	5.743
2.0	02A0002	23.761	87.585	111.535
3.0	03A0001	23.761	84.684	107.199
4.0	04A0001		92.491	88.207
5.0	05A0007		38.453	41.367
6.1	06A0003		23.464	24.600
6.2	06H0004		17.010	16.194
7.1	11E0026D		15.644	15.490
7.2	BV-10288		9.127	8.264
8.0	08A0002		7.997	8.622
9.0	09A0001		6.385	11.419
10.0	10A0002		42.836	44.316
11.0	11A0001		8.217	9.731
12.0	12B0001		4.281	5.177
13.0	13A0002		5.612	5.609
14.0	14A0001		9.037	11.604
15.0	15A0001		6.657	7.704
16.0	16A0002		4.748	4.801
17.1	17A0001		0.965	3.139
17.2	G17C0001		0.683	2.691
18.1	18A0001		1.051	1.199
18.2	G18B0001		8.170	9.131
19.1	G19B0001		9.348	15.713
19.2	G19F0001		0.574	1.649
20.0	G20_LS		8.557	11.567
21.0	G21A0001		2.338	4.814
22.0	G22A0001		1.348	2.508
23.0	G23A0001		2.144	4.734
25.0	G25A0001		2.744	5.312
26.0	G26A_LS		5.952	8.202
27.0	G27A_LS		2.900	5.098
28.0	G28D_LS		2.548	4.966
29.0	G29A_LS		1.196	2.165

5.5 Hydraulic Capacity Analyses

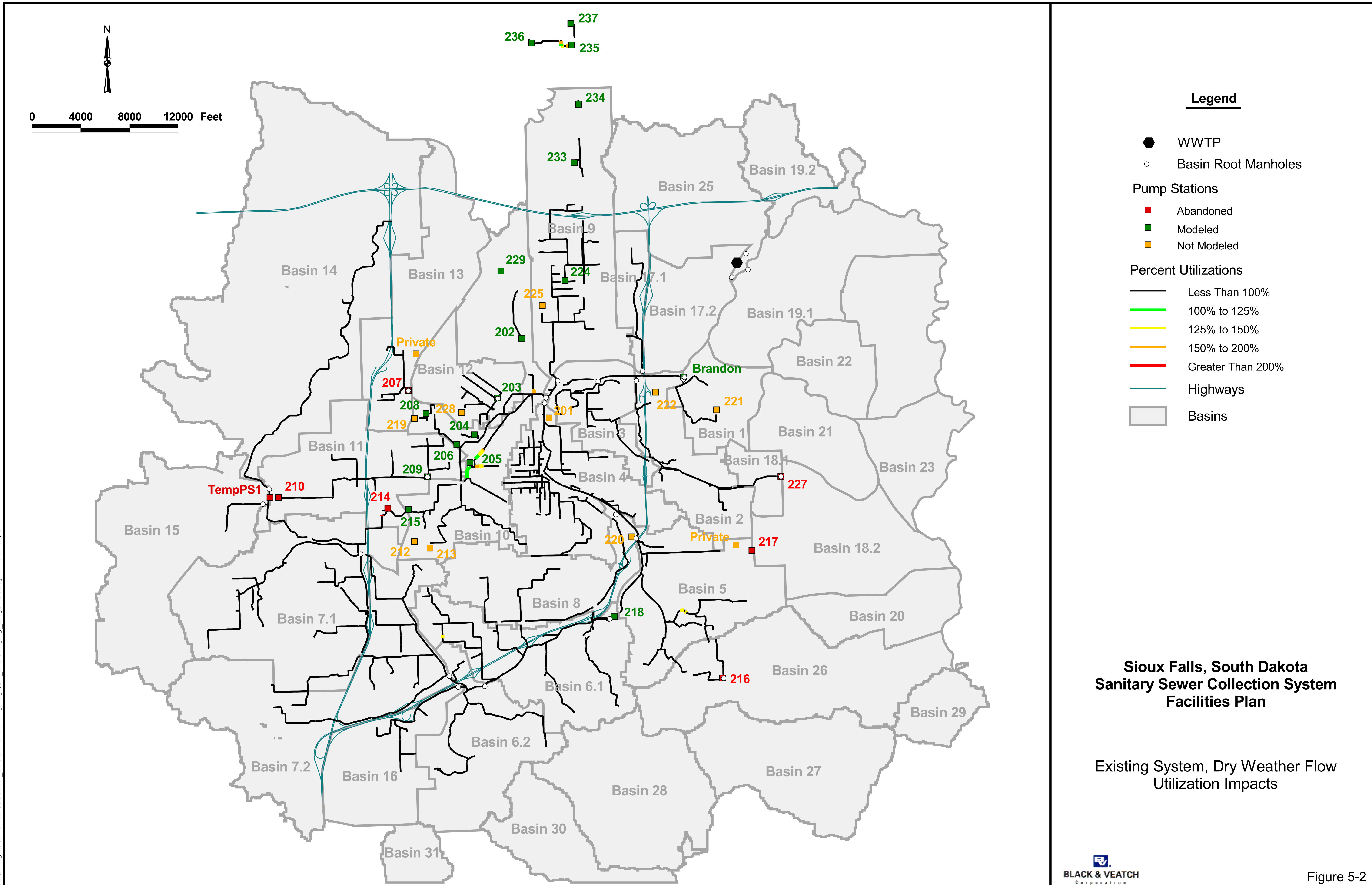
Hydraulic capacity analyses were performed for four development conditions and three flow conditions. The four development conditions included Existing, 2015, 2025 and Build-Out. The three flow conditions included peak dry-weather flow (ADDF and infiltration) and two wet-weather events (direct inflow from stormwater and dry-weather flow). The selected storms were 1-year and 25-year frequency events. The 25-year frequency storm event was considered the Design Storm.

The existing flow equalization basin was taken into account in all wet-weather hydraulic analyses. Modeling of the equalization basin operation was simplified and used the RTC capabilities of XP-SWMM. The simplified hydraulic representation of the inlet pipe and outfall sluice gate indicated hydraulic constraints in diverting peak flows to the basin. The model results indicate that improvements to the equalization basin inlet pipe and sluice gate will reduce wet weather peaks downstream.

For the purpose of determining peak potential flows, the XP-SWMM model was allowed to dynamically resize modeled conduits to ensure that the entire storm flow would be transmitted through the system. Major lift stations were also modeled as static-head pumps to ensure that peak flows were transmitted through the hydraulic network. These two steps allowed calibration of peak potential flows. These flows do not represent the actual capacity of the system to convey these flows, but rather the theoretical maximum potential flows provided no downstream hydraulic constraints. Peak flow values were returned to SSMS where they were compared with calculated existing pipe capacities to determine percentage capacity utilization under peak conditions was represented as a percentage of capacity. Thus 100 percent utilization means the pipe is flowing full at peak. Utilization of 100 percent to 125 percent is considered moderately overloaded. Utilization of 125 percent or more is considered a candidate for relief.

Figures 5-2 through 5-5 graphically illustrate the peak flow modeling results as percentage utilization. The dry weather flow conditions produced virtually no overloaded pipes for the existing system, year 2015, and year 2025. The 1-year storm event overloaded some segments but the differences between the existing system, year 2015, and 2025 are limited to magnitude. Virtually the same number of pipes were affected regardless of development conditions. A few more differences in the number of affected pipes was evident between the development conditions operating during the 25 year storm. Comparisons of the model results are found in Appendix G.

- Figure 5-2 illustrates the impacts of the dry weather flow on the utilization of the existing system.
- Figure 5-3 illustrates the impacts of the dry weather flow on the utilization of the year 2025 system.
- Figure 5-4 illustrates the impacts of the 1-year storm event on the utilization of the 2025 system.
- Figure 5-5 illustrates the impacts of the 25-year storm event on the utilization of the year 2025 system.



Sioux Falls, South Dakota
Sanitary Sewer Collection System
Facilities Plan

Existing System, Dry Weather Flow Utilization Impacts

BLACK & VEATCH Corporation

Figure 5-2

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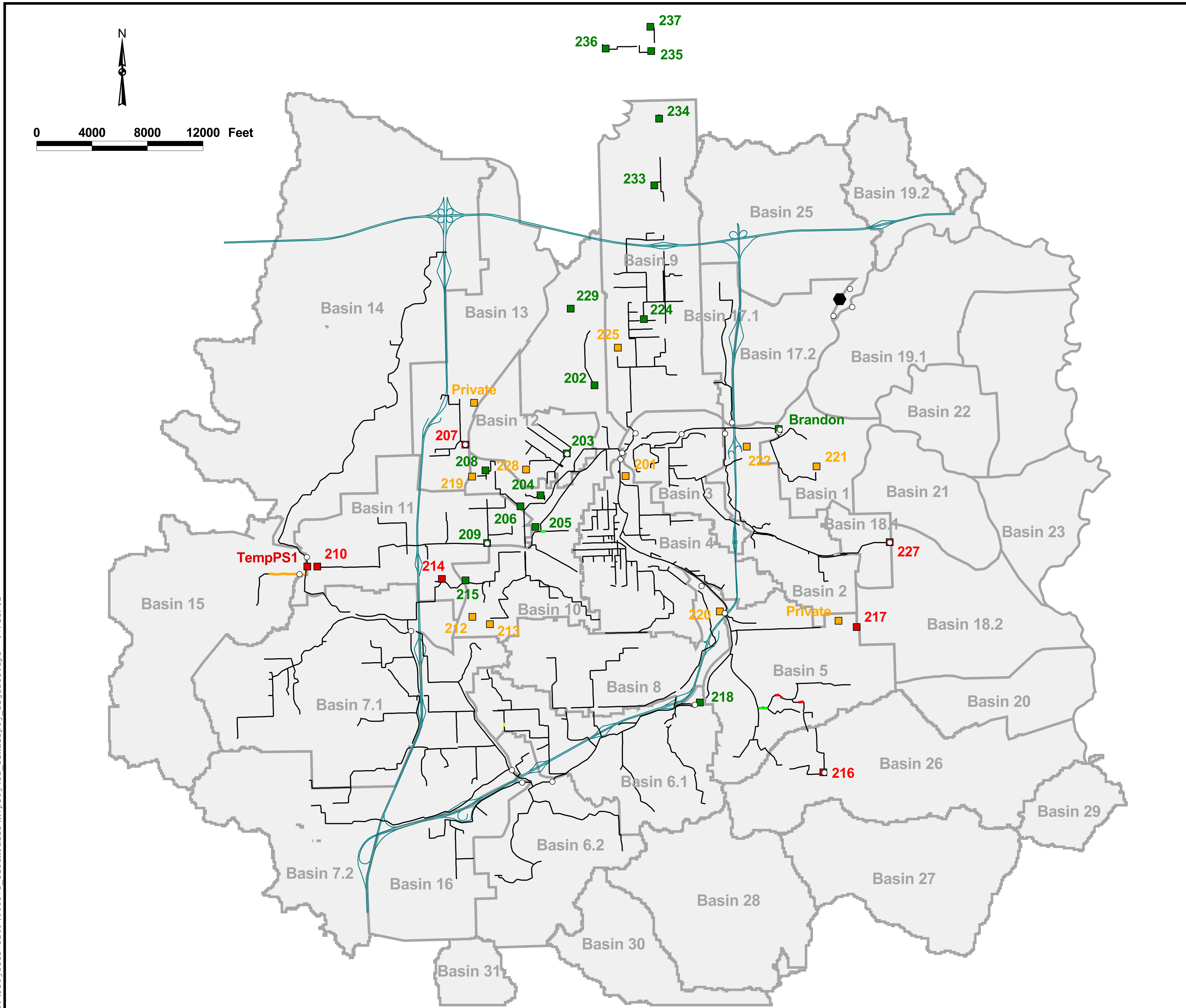

Legend

- WWTP
- Basin Root Manholes

Pump Stations

- Abandoned
- Modeled
- Not Modeled

Percent Utilizations

- Less Than 100%
- 100% to 125%
- 125% to 150%
- 150% to 200%
- Greater Than 200%
- Highways
- Basins

Sioux Falls, South Dakota
Sanitary Sewer Collection System
Facilities Plan

Dry Weather Flow, Year 2025
Utilization Impacts

BLACK & VEATCH Corporation

Figure 5-3

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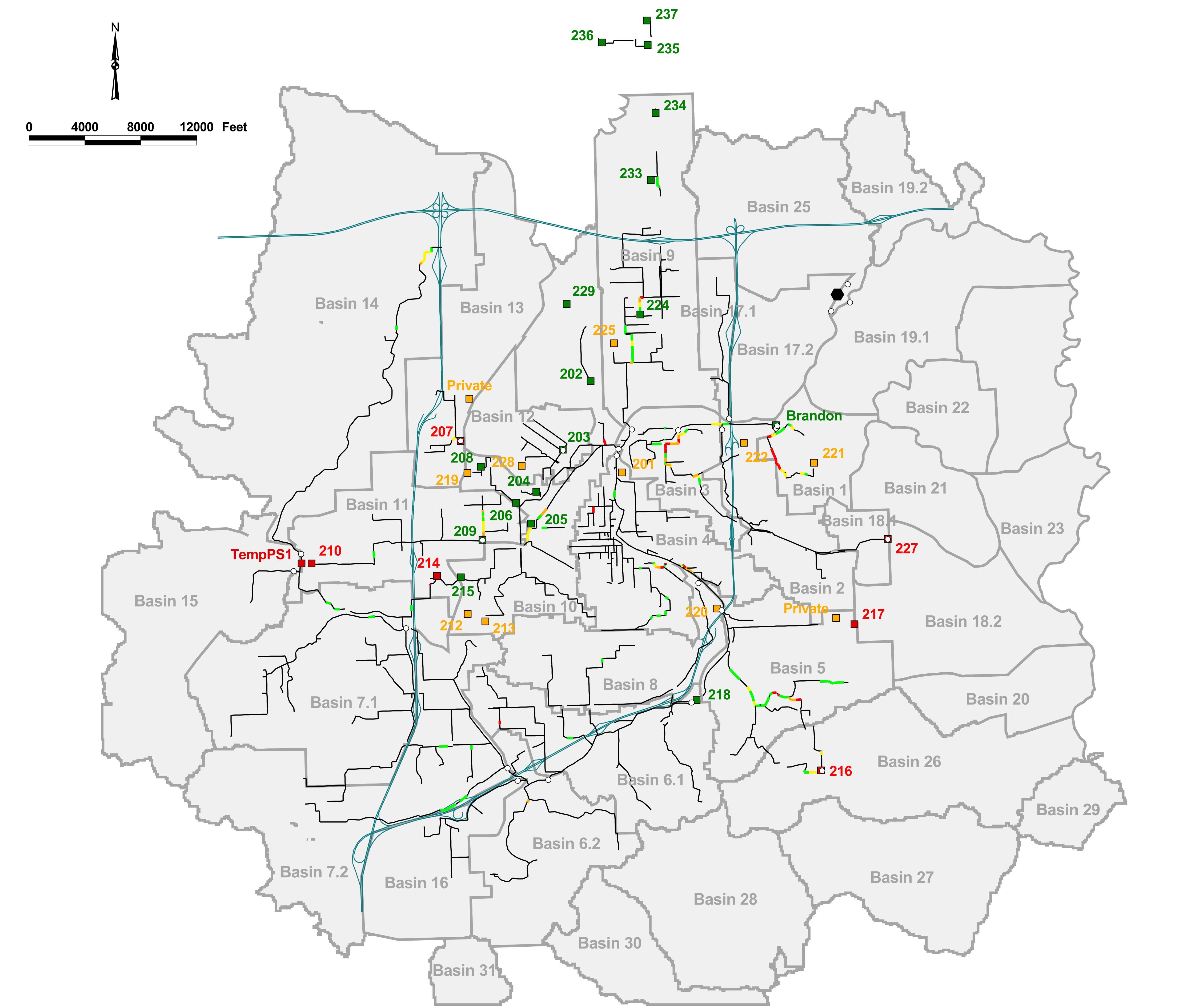

Legend

- WWTP
- Basin Root Manholes

Pump Stations

- Abandoned
- Modeled
- Not Modeled

Percent Utilizations

- Less Than 100%
- 100% to 125%
- 125% to 150%
- 150% to 200%
- Greater Than 200%
- Highways
- Basins

Sioux Falls, South Dakota
Sanitary Sewer Collection System
Facilities Plan

1-year Storm Flow, Year 2025
Utilization Impacts

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Corporation

Figure 5-4

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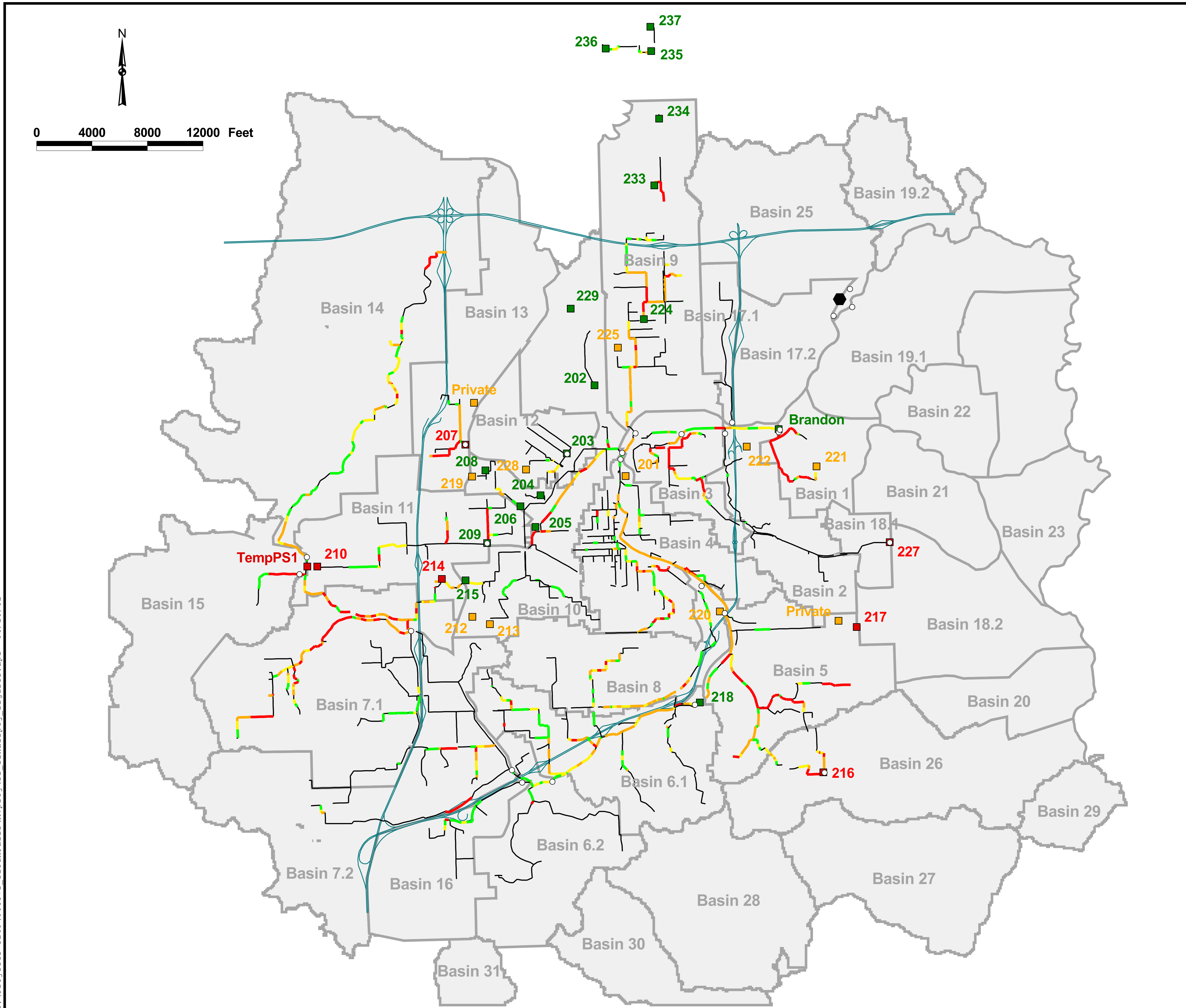

Legend

- WWTP
- Basin Root Manholes

Pump Stations

- Abandoned
- Modeled
- Not Modeled

Percent Utilizations

- Less Than 100%
- 100% to 125%
- 125% to 150%
- 150% to 200%
- Greater Than 200%
- Highways
- Basins

Sioux Falls, South Dakota
Sanitary Sewer Collection System
Facilities Plan

25-year Storm Flow, Year 2025
Utilization Impacts

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Figure 5-5

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6.0 Growth Area Analyses

6.0 Growth Area Analyses

This Chapter describes alternative plans and associated costs for managing future wastewater flows, based on the drainage basins and growth sewers defined in the February 8, 2002 Memorandum on Growth Sewers CIP Development. As directed by the City, the scope of this project presumes flows will be treated at the existing WWTP location. However, since the eastern and southeastern growth area outlets are downstream of the WWTP, this Chapter also considers a satellite WWTP.

6.1 Major Basin Flow Summary

Previously submitted technical memoranda include:

- Population and Land Use Projections, November 5, 2001.
- Wastewater Flows, November 9, 2001.
- Growth Sewer CIP Development, February 8, 2002.

The February 8, 2002 submittal included revised population and developed acres by subbasin (Appendix H) based on revised development staging. Each growth area subbasin was assigned to year 2015 or 2025 development. Since the growth acres determined by the planning department are about three times the acreage needed to support the projected population, an estimate of the actual acres needed to support projected population growth is provided in Table 6-1. The maximum month average day flow (MMAD) is 1.2 times the ADDF.

Table 6-1
Projected vs. Buildout Study Area Characteristics

Item	2000/2001	2015	2025	Buildout
Population	123,975	156,000	185,000	309,000
Basin Full Development Acres	20,583	39,525	53,172	-
Implied Population Density	6.022	3.947	3.479	-
Developed Acres	20,583	25,905	30,717	53,172
Percent of Full Development in Growth Areas	-	27.6 %	31.1 %	100 %
ADDF (mgd)	14.545	17.600	20.274	37.1
MMAD (mgd)	17.45	21.12	24.33	44.5

The existing WWTP has a permitted capacity of 19.7 mgd ADDF. Projected flows reach 19.7 mgd ADDF in year 2023. Sioux Falls should be able to handle flows with the existing WWTP treatment capacity until near the end of the 25-year design period. The costs for constructing a new satellite WWTP are, therefore, not offset by construction costs for an expansion of the existing plant. A complete plant capacity evaluation considering wastewater loads and other factors is beyond the scope of this report.

The City provided a list of the basins expected to begin development during the 5-year CIP period of 2003-2007. The staging of development, as provided by the city, is summarized in Table 6-2. Exhibit 2-1 shows the relationship between basins and growth area phases.

The Cities of Brandon and Sioux Falls agreed that the City of Brandon would develop Basin 24 and Subbasins 22A and 22B.

Table 6-2
East and Southeast Growth Areas Development Staging

Year	Area	Basins
5-year CIP (2003 - 2007)	East	18B, 18C, 18D, 18F, 19D, 21A, 21C, 21B
2007 – 2015	East	17, 18, 19 (all but 19G), 20, 21, upper 22, 25
2007 – 2015	East	Lower 22, 23, 19G
2015 – 2025	Southeast	26,27,28,29

Note: No development is projected for Basin 24 since this will be developed by the City of Brandon.

The total projected flows from existing development plus growth areas were grouped into major areas as listed in Table 6-3. Flow details by subbasin are provided in Appendix H. As shown, if flows for the southeastern basins were to be treated at a satellite WWTP, projected ADDF flows to that facility would be 1.5 mgd by 2025, and 4.6 mgd with buildout of these drainage basins.

Table 6-3 Major Area ADDF Flows						
Major Areas		Included Drainage Basins	Total ADDF (mgd)			
			2001(1)	2015	2025	Build-Out
A:	Central, Tributary to Sioux River Central Interceptor	2, 3, 4, 8, 10, 11, 12	7.102	7.184	7.196	7.403
B:	West, Tributary to Sioux River North Lift Station (No. 215)	7.1, 13, 14, 15	1.971	3.386	4.336	9.576
C:	South, Tributary to Central Interceptor	5, 6.1, 6.2, 7.2, 16	4.602	5.138	5.291	6.817
D:	North, Tributary to Central Interceptor	1, 9, 17.1	1.270	1.383	1.440	1.814
E:	Northeast, Tributary to Future WWTP Lift Station	17.2, 19.1, 19.2, 25	0.553	1.069	1.294	2.937
F:	East, Tributary to Sioux River	18.1, 18.2, 20, 21, 22, 23	0.119	0.944	1.344	4.056
G:	Southeast, Tributary to South Sioux River	26, 27, 28, 29	0.100	0.230	1.502	4.606
	Totals		**15.717**	**19.334**	**22.403**	**37.209**

(1) Flows projected based on unit rates and developed areas. Some developed areas are not sewered at present.

Peak wet weather flows at key locations in the collection system that are pertinent to the alternative analysis are summarized in Table 6-4. The flows in Table 6-4 are cumulative flows that include multiple basins. Peak flows were determined in the computer model.

Table 6-4 Future Key Location Flows (mgd)				
Location	2025		Buildout	
	ADDF	PWF	ADDF	PWF
WWTP	25.37	136.08	44.73	191.85
Brandon	22.91	123.68	37.08	158.93
PS218	4.72	22.16	6.42	35.31
PS215	4.75	32.70	10.71	60.39
GLS_WWTP	2.47	12.51	7.64	41.53
G20_LS	1.49	9.02	4.54	28.59
G26A_LS	1.03	5.31	3.10	16.85
G27A_LS	0.43	2.97	1.38	11.01
G28D_LS	0.40	2.62	1.28	9.84
G29A_LS	0.14	1.22	0.47	4.25

The layout, sizing, and probable costs for growth area sewers includes the following features:

- Lift Station No. 227 may be retired by construction of a gravity sewer to subbasin 18C, which is planned for development by year 2015.
- Lift Station No. 217 may be retired by construction of a gravity sewer to subbasin 18E, which is planned for development by year 2015.

- Lift Station No. 216 may be retired by construction of a gravity sewer to subbasin 26C, which is planned for development by year 2025.
- An area in subbasin 26A, planned for development by year 2025, could be served earlier by constructing a temporary pumping station near growth sewer manhole G26A0029, and force main to basin 20. The pumping station would be retired before year 2025 with construction of subbasin 26 sewers. Since basin 20 growth sewers are sized for buildout, no upsizing of the basin 20 sewers would be required.
- Flows from the area in subbasin 9 upstream of Lift Station No. 233 (Renner Sewer District) could be redirected through a new force main to subbasin 25D to relieve subbasin 9 sewer overloads. Subbasin 25 growth sewer sizing would need to consider the additional flow.

6.2 Alternative Plans Basis

Additional development within the existing service area and growth areas to the north, west, and southwest of the City can be served by gravity flow to connections with existing sewers. The evaluation of any improvements within the existing system to convey these flows are part of the computer model analyses. Alternative plans for these areas are not required.

Because flows from the growth areas east and southeast of the City cannot be delivered to the existing sewers without pumping, definition and evaluation of alternative plans for these areas is required. The selected alternative is used in the computer model to determine impacts on the existing facilities.

Four alternative conceptual plans to manage flows from the eastern and southern basins that cannot drain by gravity to the existing WWTP or to existing pumping stations are as follows:

- Plan 1: Force mains to existing collection system.
- Plan 2: Force mains to existing WWTP.
- Plan 3: Eastern basins force main to existing WWTP, and Southeastern basin force mains to existing collection system.
- Plan 4: Construction of a new Southeast WWTP. Plan 4 is evaluated as two sub-alternatives, Plan 4A and Plan 4B, to consider whether flows should be conveyed to the WWTP by pumping and force mains or by gravity sewers to a final pumping station.

Within a drainage basin, the collection system is generally the same regardless of the alternative plan. The collection systems are designed to drain to the low point of the basin. At that location, a pumping station would discharge to a force main conveying flow under pressure to either a WWTP, or to a point in an adjoining basin where gravity flow could resume. The addition of flows to existing basins will impact required sizes for certain major sewers in receiving basins.

Each alternative plan is described in the following sections and summarized with probable costs in the attached tables. Pumping station and force main sizing was determined both for year 2025 and buildout development. Since build-out significantly exceeds the development that will be supported by the 2025 populations, it is cost-effective to construct these facilities for year 2025 flows. Additional growth beyond the year 2025 projections will require pumping station expansions and parallel force mains.

The basis for the cost analysis is as follows:

- Costs for gravity sewers in the growth areas do not vary by alternative plan and are not included.
- Unit construction costs for sewers, pumping stations, and force mains are listed in the Appendix J.
- Operation and energy costs for pumping stations are estimated based on year 2025 ADDF, pumping head, 70 percent wire-to-water efficiency, and $0.08 per kilowatt hour.
- Satellite WWTP construction costs are estimated based on a planning level unit cost of $ 4.50 per gallon per day average capacity. This cost level was selected based on available cost estimates for comparable facilities.
- Satellite WWTP operation and maintenance costs area estimated based on an assumed annual rate of 3.5 percent of the capital costs.
- Capacity impacts on the existing system facilities are based on computer model results.
- A present worth factor of 14.094 times annual costs is used, based on 5 percent interest rate and a 25-year period.
- Probable capital costs are construction costs plus an allowance of 50 percent for contingencies and engineering, legal, and administrative costs.

Future pump station G29C_LS (February 8, 2002 memorandum) was eliminated for this analysis with flows from subbasin 29C conveyed by gravity flow to future pump station G29A_LS.

6.2.1 Plan 1: Force Mains to Existing Collection System

The growth area sewers in the east and southeast growth areas terminate in pumping stations as identified in the memorandum on Growth Sewers CIP Development. Each pumping station would discharge to force mains. Under Plan 1, the force mains are directed to the nearest existing trunk sewer. Since additional flows could exceed the capacity of existing sewers, expansion or relief sewers are evaluated in the computer model. Plan 1 facilities are shown on Figure 6-1.

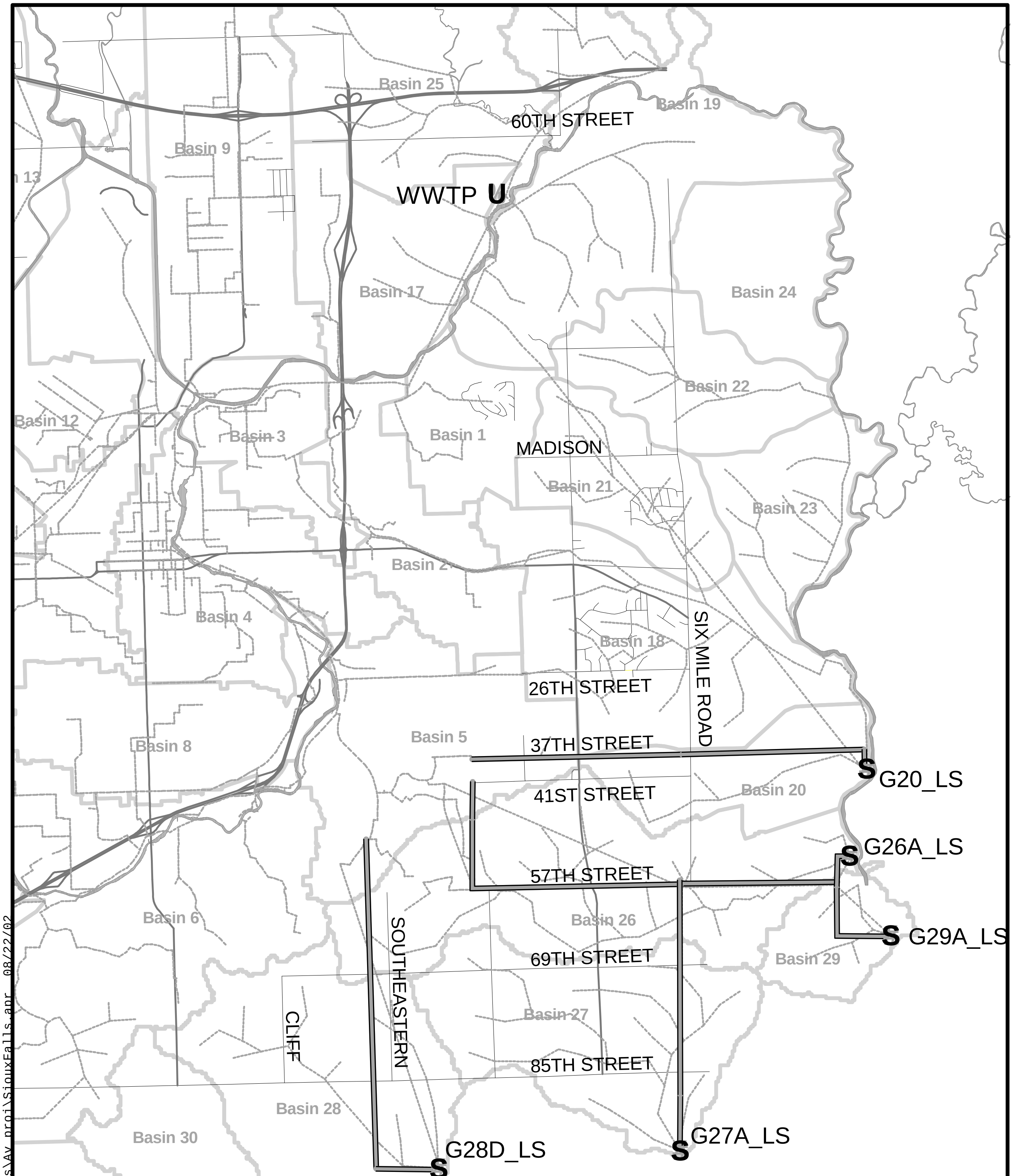
By year 2007, flows from eastern Basins 18, and 21 will be conveyed to pump station G20_LS. By year 2015, development would include Basin 20. By year 2025, Basins 22 and 23 would be added to this system. Flows from Station G20_LS would be pumped through a force main to existing trunk sewers. The shortest alignment would extend north to 37th Street, then west on 37th Street to connect to an existing 15-inch gravity sewer at manhole 05EA022 near the intersection of 37th Street and Sycamore Avenue. This force main would be 20,415 feet in length.

The highest ground elevation on the force main alignment is at about 1,510 feet, and with a pumping station wetwell elevation of about 1,260 feet, the static lift is about 250 feet (108 psi) before considering headlosses. Generally, high capacity wastewater pumps with rated heads above about 200 to 230 feet are difficult to obtain. For this installation at G20_LS, two pumping units installed in series will probably be required. An alternative would be to provide an intermediate station at a suitable location along the force main. Since all of the future pumping stations are at similar elevations, and all of the potential force mains cross similar ridge line elevations, similar pumping heads apply to all alternative plans.

The southeastern basins are planned for development in the 2015 – 2025 period. Flows from these basins would be handled as follows:

- Basin 28 growth sewers terminate at pump station G28D_LS. The flows would be carried through a force main extended east to Southeastern Avenue, and then north along Southeastern Avenue to an existing 10-inch sewer at manhole 05F010, near the intersection of Southeastern Avenue and 49th Street.

- Flows from Basin 27 are collected at pump station G27A_LS. These flows would be conveyed by a force main north along Six Mile Road to 57th Street, and west on 57th Street to connect to a 10-inch sewer at manhole 05H0010. This manhole is on Judy Avenue just south of the intersection of Judy Avenue and Marson Drive. Lift Station No. 216 could be retired when the gravity sewers are constructed.



Legend

- U WWTP
- S Pumping Stations
- Force Mains
- Trunk Sewers
- Highways
- Rural Streets
- Big Siouux River
- Drainage Basins

Sioux Falls, SD
Sanitary Sewer Collection System
Facilities Plan

Plan 1 Facilities

Figure 6-1

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- Flows from Basins 26 and 29 are collected to pump stations G26A_LS and G29A_LS respectively. Force mains for these stations would merge at 57th Street, and then extend west to join with the Basin 27 force main at Six Mile Road. The force mains are intended for this analysis to be common force mains where they are on the same alignments, rather than parallel force mains.

6.2.2 Plan 2: Force Mains to Existing WWTP

- Plan 2 facilities are shown on Figure 6-2. Under Plan 2, impacts on the existing collection system are eliminated by extending a long force main directly to connect to the existing WWTP. The proposed alignment is on Six Mile Road. The force main would extend from pump station G27A_LS north to Timberline Avenue, thence westward to cross the Big Sioux River, terminating at the existing WWTP. Flows from Basin 28 are conveyed by force main to the Six Mile Road force main along 69th Street. Flows from Basins 26 and 29 are conveyed by force main along 57th Street to join the Six Mile Road force main. Flows from Basins 18, 21, 22, and 23 are all conveyed by force mains along 37th Street to join the Six Mile Road force main. The force mains are intended for this analysis to be common force mains where they are on the same alignments, rather than parallel force mains.

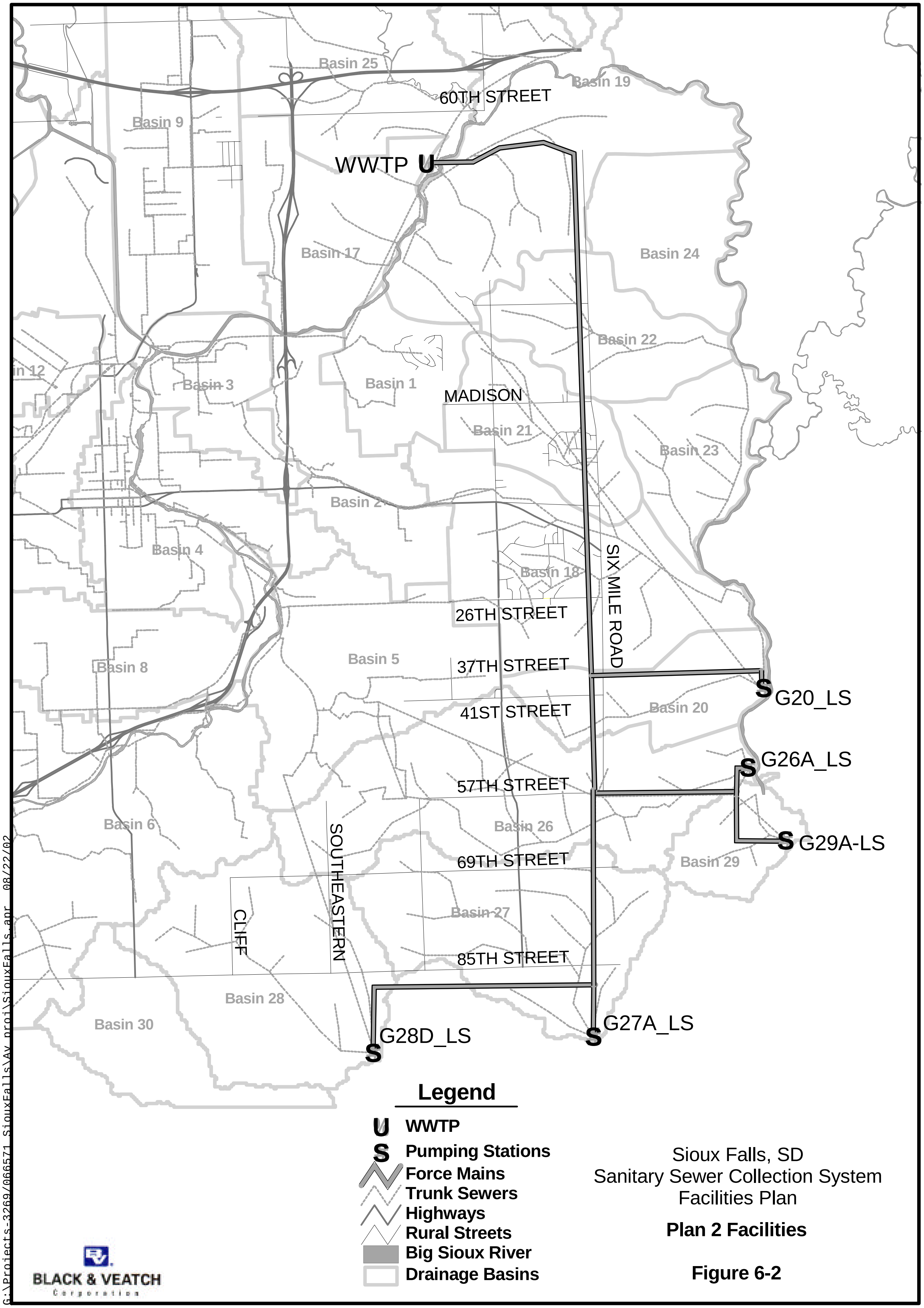
Since flows would bypass the Brandon pumping station, screening facilities may be required for these flows at the WWTP.

6.2.3 Plan 3: Eastern Basins Force Main to Existing WWTP, and Southeastern Basin Force Mains to Existing Collection System

Under Plan 3, the eastern basin flows collected at pumping station G20_LS would be pumped via force main to the WWTP. Flows collected from southeastern Basins 26, 27, 28, and 29 would be conveyed to existing manholes as described under Plan 1. Plan 3 facilities are shown on Figure 6-3.

Three alternative alignments for the station G20_LS force main are considered:

1. On 37th Street, Six Mile Road, and Timberline Avenue as described under Plan 2.
2. North from the pump station along the Sioux River to E. 60th Street, thence east to cross the Sioux River, and south to the WWTP. The alignment is to locate the sewer in ground not exceeding 1360 feet elevation, which is the high water level at the WWTP.



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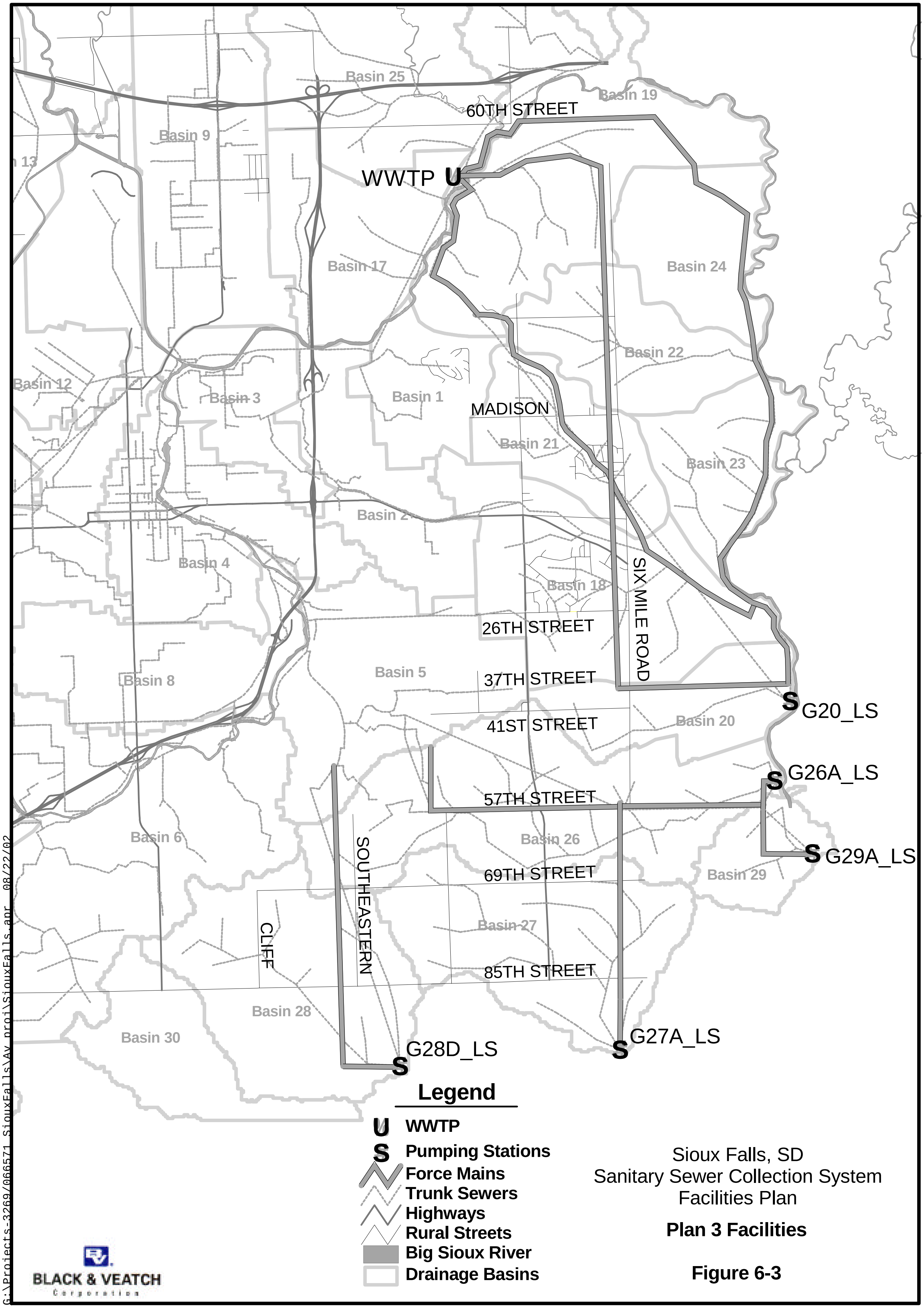

Sioux Falls, SD
Sanitary Sewer Collection System
Facilities Plan

Plan 3 Facilities

Figure 6-3

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3. Following the principal trunk sewer planned for basin 18 from the pump station to near planned manhole G21B009, thence over the ridge to the vicinity of planned manhole G19A009. From that location, the force main would parallel the proposed growth area sewers to terminate at the WWTP.

Table 6-5 shows a comparison of the potential force main alignments for Plan 3. The force main is 24-inch diameter, to provide 9.0 mgd at less than 3.5 fps velocity. Energy costs are based on 1.5 mgd ADF flow on an annual basis. As shown, alignment 3 following the drainage course has the lowest capital cost and lowest present worth. Therefore, alignment 3 is recommended.

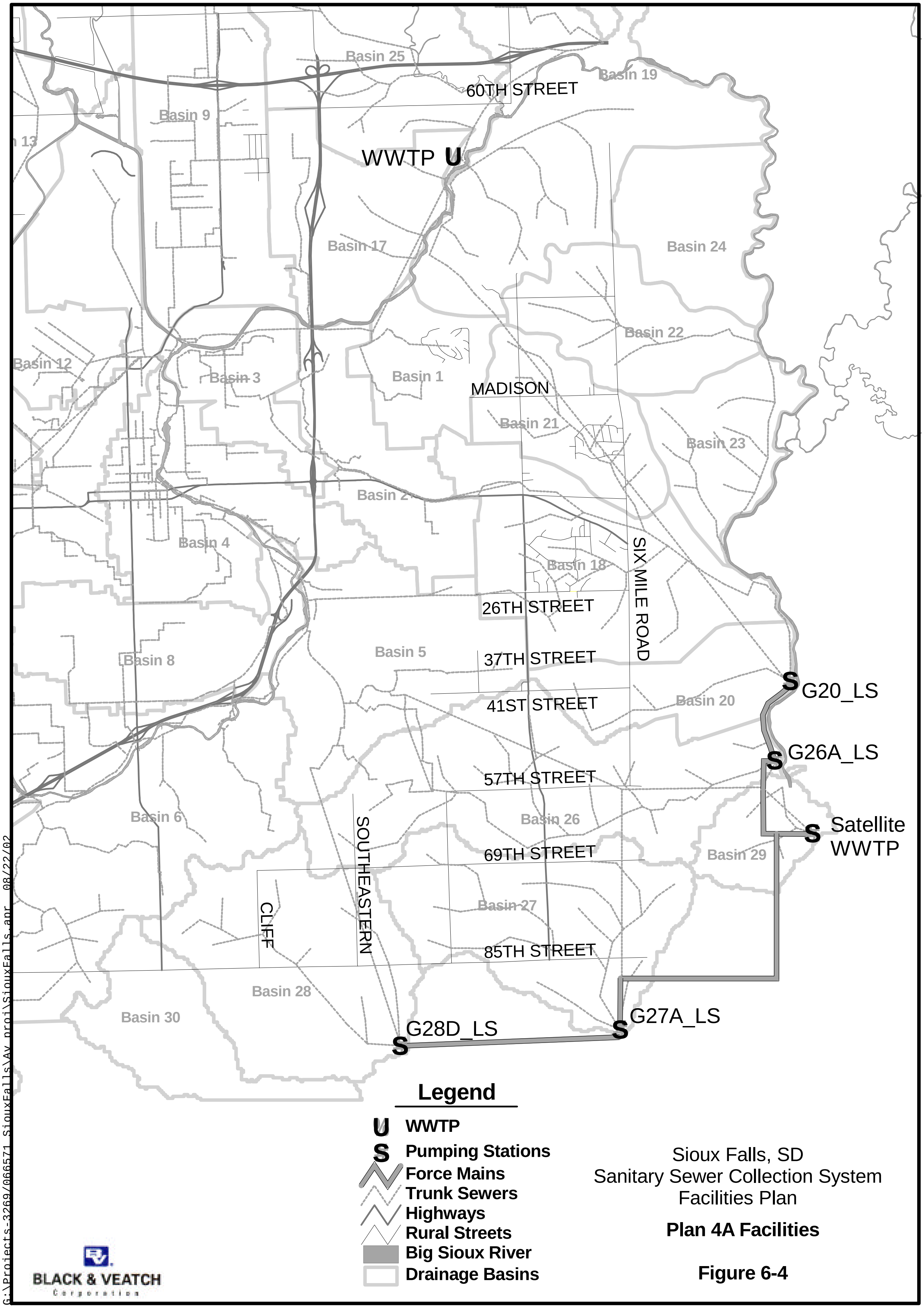
Table 6-5
East Basin Force Main Alternatives

Alternative	Length (ft)	Capital Cost (Mil. $)	High Ground Elevation (ft)	Annual Pumping Cost ($)	Present Worth (Mil. $)
1. Six Mile Road and Timberlane Avenue	46,710	4.974	1500	24,361	5.571
2. Sioux River and E. 60th Street	49,400	5.261	1360	841	5.522
3. NW along drainage course	37,400	3.982	1530	29,416	4.650

6.2.4 Plan 4: Construction of a New Satellite WWTP

6.2.4.1 Plan 4A: Pumping Stations and Force Mains to Southeast WWTP

Plan 4A facilities are shown on Figure 6-4. Plan 4A includes a new satellite WWTP to serve all eastern and southeastern basins. A location south of Basin 29 is assumed. This plan would allow for future growth beyond the year 2025, and lessen the likelihood of overloading the existing WWTP. Flows from Basin 28 would be pumped to the Basin 27 pump station location, and the combined flows would be pumped via a force main along Six Mile Road and 69th Street to the new plant. Flows collected at pumping stations G20_LS, G26A_LS, and G20A_LS will be pumped through a series of force mains to the new satellite WWTP. The force mains are intended to be a single force main where they follow the same alignment. The force mains are sized for projected 2025 development, and may need to be paralleled or replaced after year 2025 if development proceeds to buildout.



Sioux Falls, SD
Sanitary Sewer Collection System
Facilities Plan

Plan 4A Facilities

Figure 6-4

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From Table 6-3, the design ADDF flow to the satellite plant is 2.846 mgd for year 2025, and 8.662 mgd at buildout. Based on $4.50 per gallon per day for treatment, the capital cost for a 2.846 mgd plant is about $12,800,000. The annual operating cost, estimated at 3.5 percent of the capital cost, is about $450,000. The present worth of capital and annual operating costs for the plant is $19,142,000.

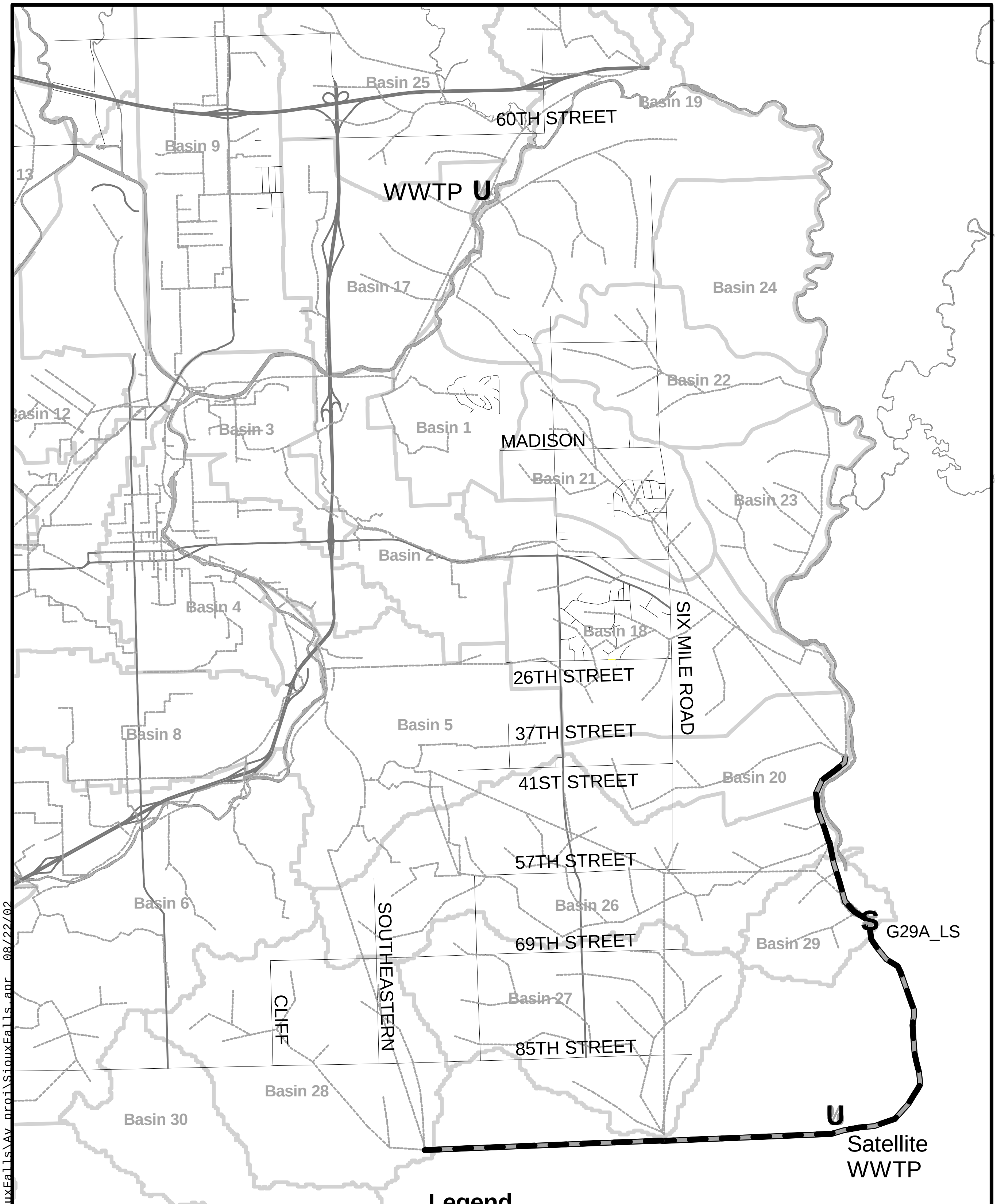
Subbasin 24 is not included in the projected flows because it is intended to be served by Brandon. However, the potential for future flows from Brandon should be considered when the downstream gravity sewers, pumping stations and force mains are designed.

6.2.4.2 Plan 4B: Gravity Sewers to Satellite WWTP

Plan 4B facilities are shown on Figure 6-5. This alternative Plan 4B includes the new satellite WWTP to serve all eastern and southeastern basins. A location south near the confluence of Nine Mile Creek and the Big Sioux River is assumed. This plan would allow for future growth beyond the year 2025, and lessen the likelihood of overloading the existing WWTP.

The growth area pumping stations are eliminated by constructing major gravity trunk sewers to the satellite WWTP. The growth area pumping stations discussed under the preceeding alternative plans are not required. A raw water pumping station would be required at the proposed plant. Flows from Basins 28 would be pumped to the Basin 27 pump station location, and the combined flows would be pumped via a force main along Six Mile Road and 69th Street to the new plant. Flows from Basins 18, 20, 21, 22, 23, 26 and 29 travel by gravity sewer to a pump station G29A_LS or its equivalent, and are pumped into the new satellite WWTP. The force mains are sized for projected 2025 development, and may need to be paralleled or replaced after year 2025 if development proceeds to buildout.

From Table 6-3, the design ADDF flow to the satellite plant is 2.846 mgd for year 2025, and 8.662 mgd at buildout. Based on $4.50 per gallon per day for treatment, the capital cost for a 2.846 mgd plant is about $12,800,000. The annual operating cost, estimated at 3.5 percent of the capital cost, is about $450,000. The present worth of capital and annual operating costs for the plant is $19,142,000.



Legend

- WWTP
- Gravity Mains
- Trunk Sewers
- Highways
- Rural Streets
- Big Sioux River
- Drainage Basins

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Sioux Falls, SD
Sanitary Sewer Collection System
Facilities Plan

Plan 4B Facilities

Figure 6-5

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6.3 Modeled Facility Demands and Existing Interceptor Relief Needs under Plan 2

Computer simulation of the existing and proposed collection system was performed using XP-SWMM hydraulic modeling software. The growth sewers defined in the February 8, 2002 memorandum are included. The pumping station and force main facilities defined previously for Plan 2 are included. Additional improvements in the model include:

- recently completed sewers that direct the northwestern growth areas and northern portion of Basin 7 to pumping station 215, and
- a planned trunk sewer to eliminate pumping station 207 by conveying its flows to pumping station 215.

Modeling results determined the 2025 peak flows at existing and proposed facilities, determined relief needs for existing sanitary sewer interceptors under 2025 and Build-Out conditions. For the design year 2025 analysis, the growth area developed acres in the model were adjusted to match the projected population growth.

In the Relief CIP Table 6-6, relief pipe diameters are selected based on an assumption of replacement of existing pipe, and providing design capacity equal to 2025 peak wet weather flows. Wet weather flow is wastewater production plus total infiltration plus the inflow from the 25-year frequency storm event.

Table 6-6
Relief Sewer Projects
Central Trunk Sewer Alignment

Existing Collection System Relief CIP Project Name	Map Label	Upstream Manhole	Downstream Manhole	Length (ft)	Average Slope (ft/ft)	Average CIP Diameter (in)	Average CIP Capacity (cfs)	2025 Peak Flow (cfs)	Relief Cost (M$)
Outfall Sewer	02_03	04A0001	02A0002	11,453	0.0030	87	385.7	191.6	5.275
Sioux River Central Interceptor, Project 1	SRCI_03B	04H0012	04A0009	10,873	0.0008	60	100.0	81.0	3.580
Sioux River Central Interceptor, Project 2	SRCI_03C	05B0007	04H0012	5,917	0.0012	54	72.0	66.0	1.761
Sioux River Central Interceptor, Project 3	SRCI_03D	05C0009	05B0007	11,364	0.0006	54	55.0	43.0	1.163
Sioux River Central Interceptor, Project 4	SRCI_03E	(BV-10264)	06A0001A	5,587	0.0005	54	50.0	42.0	4.157
Southeast Growth Tie-In, Project 1	05_01	(BV-8646)	05B0007	4,735	0.0078	42	88.6	21.0	0.514
Southeast Growth Tie-In, Project 2	05_01	(BV-8787)	(BV-8646)	5,301	0.0060	33	40.8	14.6	0.543
Southeast Growth Tie-In, Project 3	05A_03	05F0011	(BV-8646)	4,712	0.0082	17	8.1	5.9	0.332
Basin 10 Interceptor Relief, Project 1	BAS10_01		10A0004	10,005	0.0026	39	41.6	76.2	1.623
Totals				**69,947**					**18.948**

6.4 Comparison of Alternative Plans

Table 6-7 provides a summary comparison of the alternative Plans. Plans 4A and 4B include the potential Southeast WWTP, are shown to be essentially equivalent in present worth to each other, and are the most economical long-term plans.

Table 6-7
Summary of Alternative Plans

	Construction Costs						
	Force Mains $	Pumping Stations $	Relief or Trunk Sewers $	Screening Facilities $	Treatment Facilities $	Total Capital Costs $	Total Present Worth $
Plan 1	4,350,000	5,760,000	20,954,166	-	-	46,596,249	48,880,618
Plan 2	6,788,000	9,297,000	18,711,918	350,000	-	52,720,377	56,304,177
Plan 3	4,881,000	7,772,000	20,513,058	250,000	-	50,124,087	52,581,292
Plan 4A	2,276,000	9,179,000	18,711,918	-	12,807,000	36,393,000	44,758,850
Plan 4B	-	3,513,000	27,769,918	-	12,807,000	38,067,000	44,494,574

However, the distinguishing feature of Plan 4, the satellite treatment facility, need not be constructed for 10 years. The decision is safely postponed until development begins in Basin 20 and southwards. At that time, alternatives should be re-evaluated based on the most current information.

Of the Plans that exclude a satellite plant, Plan 1 shows the lowest present worth. Plan 3 has the next lowest present worth. Plan 3 provides the greatest flexibility. In consultation with the City the Capital Improvements Program for this report was based on Plan 3

6.5 Recovery of Development Costs

The East Side Growth Area was divided into four planning areas, per discussions with the City, for the purpose of estimating the recoverable capital costs per developed acre at Build-out. Table 6-8 and Figure 6-6 define the Planning Areas by Subbasin.

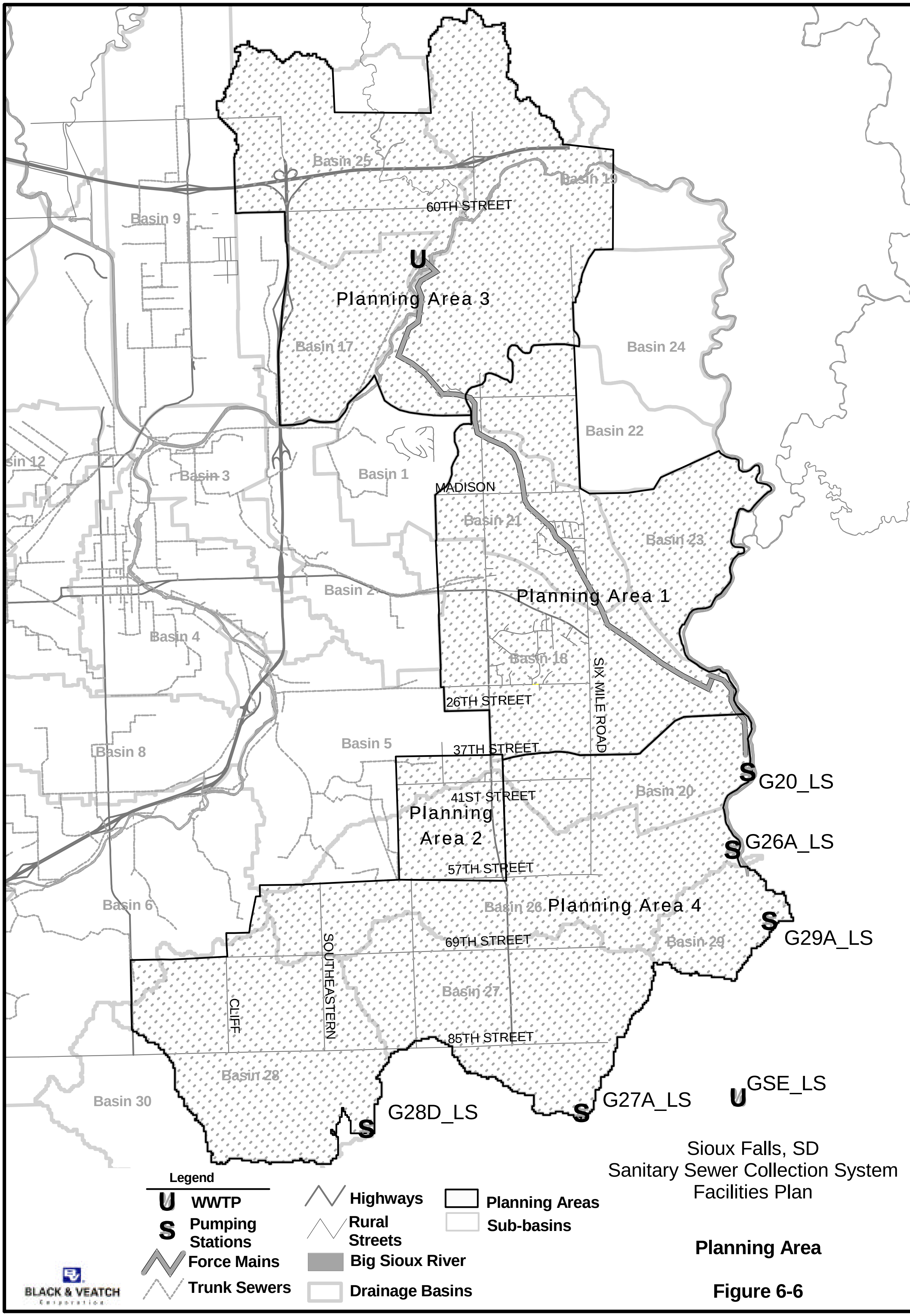
The costs, detailed in Table 6-8, Recoverable Capital Costs Per Acre, were based on Plan 3.

Table 6-8
Areas Brought Into Development Summary

East Side Planning Area No.	Subbasins Served	Build-out Developed Acres	Component	Sizing Basis	Unit Costs Basis ($)	Total Const. Costs ($)	Total Capital Costs ($)	Total Capital Cost to City (Relief Sewer Capital Costs) ($)	Capital Cost w/o Relief Sewer ($)	Recoverable Capital Costs per Total (Build-out) Acre Developed ($/ac)
Area 1	18,20, 21,23,22 C, 22D	5262	Growth Sewers	Technical Memo		13,094,000	19,641,000			
			Pump Station "E"	11.11 cfs (7.2 mgd)	199,200 / mgd	1,434,240	2,151,360			
			Force Main, G20_LS to WWTP	37,400 ft, 24"	71 / ft	2,655,400	3,983,100			
			Pump Station at G22C001 for subbasins 22C, 22D	9.63 cfs	223,100 / mgd	2,148,000	3,222,000			
			Force Main, G22C001 to N. of G21B001	7100 ft, 8 inch	24 /ft	170,400	255,600			
			Screening Facilities	LS		250,000	375,000			
					Total Area 1	19,752,040	29,628,060	0	29,628,060	5,631
Area 2	26B (65%)	505	Growth Sewers	Technical Memo		784,000	1,176,000			
			Relief Sewers	Interim Alternatives		288,000	432,000			
			Pump Station at G26A0029	0.780 cfs	462,100 per mgd	360,360	540,540			
			Force Main, G26A0029 to 05EH003	8 inch, 12,000 ft	24 per foot	288,000	432,000			
					Total Area 2	1,720,360	2,580,540	432,000	2,148,540	4,255
Area 3	17, 19 (Except part 19E & 19H), 25 (Except 25F)	5412	Growth Sewers	Technical Memo		10,386,000	15,579,000			
			Pump Station at WWTP	13.96 cfs		2,012,000	3,018,000			
			Screening Facilities	LS		250,000	375,000			
					Total Area 3	12,398,000	18,597,000	0	18,597,000	3,436

Table 6-8
Areas Brought Into Development Summary

East Side Planning Area No.	Subbasins Served	Build-out Developed Acres	Component	Sizing Basis	Unit Costs Basis ($)	Total Const. Costs ($)	Total Capital Costs ($)	Total Capital Cost to City (Relief Sewer Capital Costs) ($)	Capital Cost w/o Relief Sewer ($)	Recoverable Capital Costs per Total (Build-out) Acre Developed ($/ac)
Area 4	26 (Except part 26B), 27, 28, 29	6755	Growth Sewers	Technical Memo		11,414,000	17,121,000			
			Relief Sewers	Table 6-6						
			SRC1-03B	60", 10,873 ft		3,580,000	5,370,000			
			SRC1-03C	54", 5,917 ft		1,761,000	2,641,500			
			05-01	15 - 36", 10,036 ft		1,363,000	2,044,500			
			05-03	18", 4,712 ft		405,000	607,500			
			Pump Stations	Chapter 7						
			G28D_LS	4.05 MGD		835,000	1,252,500			
			G27A_LS	4.6 mgd		947,000	1,420,500			
			G29A_LS	1.88 mgd		486,000	729,000			
			G26A_LS	8.22 mgd		1,480,000	2,220,000			
			Force Mains	Chapter 7						
			G28D_LS	2.62 cfs	18670 ft	670,000	1,005,000			
			G27A_LS	2.97cfs	12870 ft	463,000	694,500			
			G29A_LS	1.22cfs	5545 ft	133,000	199,500			
			G26A_LS	5.31cfs	1750 ft	82,000	123,000			
			G29A_LS + G26A_LS	6.53 cfs	7585 ft	455,000	682,500			
			G27A_LS + G29A_LS + G26A_LS	9.5 cfs	15520 ft	1,102,000	1,653,000			
					Total Area 4	25,176,000	37,764,000	10,663,500	27,100,500	4,012



Sioux Falls, SD
Sanitary Sewer Collection System
Facilities Plan

Planning Area

Figure 6-6

7.0 Recommended Capital Improvements Plan

7.0 Recommended Capital Improvements Plan

7.1 Introduction

This section presents the Recommended Capital Improvements Plan, which includes recommendations for an adequately sized trunk sanitary sewer system. The Capital Improvements Plan was prepared using information from flow monitoring, sewer system inventory, growth and development projections from the City, direction from City Staff, and computer modeling described in this report. The plan addresses requirements for trunk sewers that are 10 inches in diameter or larger. The planning area excludes Basin 24 which is in the City of Brandon growth area. Included were subbasins 22A and 22B which were later found to be in the City of Brandon growth area. The plan includes the following components:

- Constructing relief sewers.
- Upgrading pumping stations and force mains.
- Constructing sewer extensions to serve Growth Areas.

The capital improvements recommended in the Plan are based on the following criteria:

- Sewer capacity and flow containment for 25-year storm event.
- Sanitary sewer flow projections for years 2015 and 2025.
- Replacement pipes were sized based on estimated flow installed at the same slope of the existing pipes.

The actual extended sewer and relief sewer sizes should be based on detailed design including slopes and using the estimated projected flow for year 2025 during the 25-year storm event.

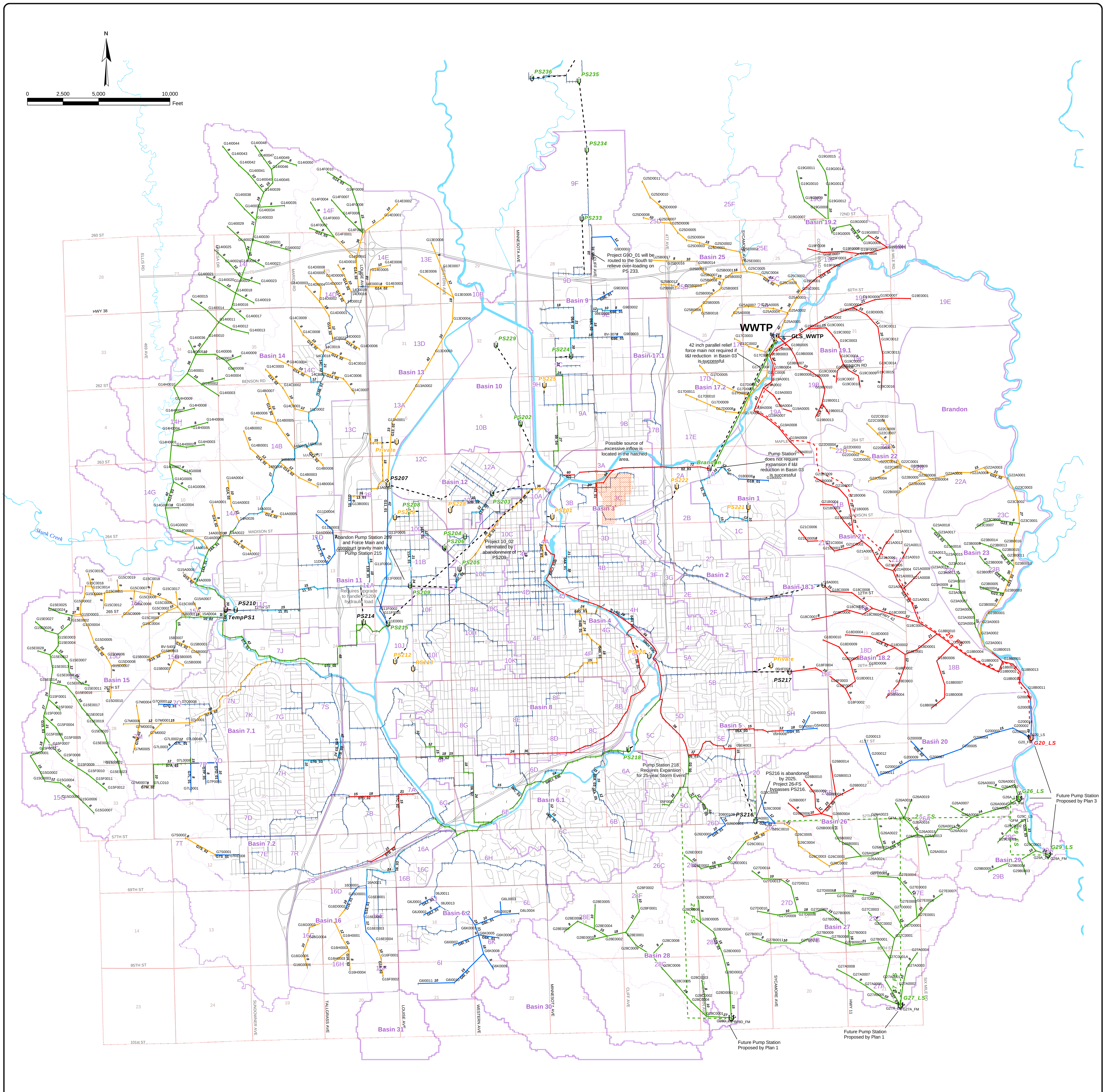
Figure 7-1 shows the facilities included in the implementation plan. The recommended improvements are grouped into three priorities and a watch list. Priority 1 improvements are needed to address immediate or near term deficiencies and can be implemented and placed into service within the next 5 years. Priority 2 improvements are additional facilities needed by year 2015. Priority 2 improvements should be reviewed before implementation, based on the actual growth that occurs. Priority 3 improvements are facilities that are needed by year 2025. Priority 3 improvements should be reviewed at the same time as Priority 2 improvements as changes in growth patterns alter priorities. Watch list projects include pipes that are marginally surcharged during large storm events or used assumed data in the absence of directly measured data. Depth of surcharge should be checked during storm events at the downstream manholes on watch list pipes.

7.2 Relief Sewers

For projected year 2025 development, a total of 307,249 feet of sewer lines are surcharged by the 25-year storm event and 230,534 feet of sewer lines are surcharged by the 5-year storm event. A total of 5,114 feet of sewer lines are surcharged during dry weather for projected 2025 development. The overloaded sewer lines are listed in Appendix J, Modeled Results and Comparisons. Recommended relief sewers, shown on Figure 7-1 and listed in Table 7-1, were identified based on the 25-year storm event flows and were sized to handle year 2025 development flows. The relief sewer and existing sewer capital improvement projects are shown on Figure 7-1. No priorities are associated with the Capital Improvement Projects numbering system. The projected cost of relief sewers required to alleviate priority 1 hydraulic deficiencies is $22.3 million, as shown in Table 7-1.

The principal reasons that relief sewers are required are:

- Existing sewer lines are undersized for large storm events.
- Basin 3, which includes the Stockyards and the Morrell plant, includes inflow sources such as area drains. If the inflows observed during flow monitoring are proportional to rainfall, these sources would contribute an estimated inflow of 65 mgd during the 25-year storm, overloading the existing 66 inch trunk sewer and the Brandon Pump Station (CIP 02_03).
- The Sioux River Central Interceptor sewer (CIP SRCI_03B) is significantly overloaded during the 5-year and 25-year storm events.



Legend

Note:

Sizing of pipes was based on the preliminary alignments, modeled flows, existing slopes for sewer relief or the ground surface slope for proposed extensions.

Pipe location and sizing will be finalized under the sewer design contracts.

WWTP

Pump Stations
- Abandoned
- Modeled
- Not Modeled

Proposed Pump Stations
- Priority 1
- Priority 3

Proposed Extensions
- Priority 1
- Priority 2
- Priority 3
- As Needed

Relief Priority
- Priority 1
- Priority 2
- Priority 3
- Watch List
- No Relief Required

Existing Force Mains

Growth Area Force Mains - Plan 1
- Priority 1
- Priority 3

Interstate
State Road
Rural Road
City Road

Big Sioux River
Section Lines

Sub-Basins By Phase
- 2000 Phase Basin
- 2015 Phase Basin
- 2025 Phase Basin

Primary Basin

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Recommended Facility Reliefs
Sioux Falls, South Dakota
2002

Figure 7-1

Table 7-1
Recommended Relief Sewers-Priority 1

CIP Name	Upstream Elevation	Downstream Elevation	Range of Existing Diameters (in.)		Range of Relief Diameters (in.)		Total Length (ft.)	Construction Cost $	Capital Costs $
02_03	1,311.25	1,292.00	36	66	72	72	8,904	3,384,000	5,076,000
03_01[(1)]	1,333.95	1,300.14	12	36	30	54	4,754	925,000	1,387,500
05_01	1,458.13	1,391.78	12	24	15	36	10,036	1,363,000	2,044,500
05A_03	1,475.55	1,455.35	8	15	12	18	4,033	325,000	487,500
07B_02	1,426.18	1,413.68	10	10	15	15	2,264	180,000	270,000
07C_02	1,403.78	1,389.78	8	15	15	21	4,233	357,000	535,500
07D_02	1,409.38	1,403.88	12	12	15	18	2,148	178,000	267,000
08_01[(1)]	1,407.56	1,382.10	14	24	24	36	16,518	2,563,000	3,844,500
EQ	1,325.37	1,325.17	36	36	60	60	850	280,000	420,000
SRCI_03B	1,382.10	1,372.89	48	48	60	60	10,873	3,580,000	5,370,000
SRCI_03C	1,389.93	1,382.10	42	42	54	54	5,917	1,761,000	2,641,500
						Priority 1 Totals	**70,530**	**14,896,000**	**22,344,000**

[(1)] Project included to show cost, see text for alternative recommended.

For planning purposes, relief sewers are sized to replace existing sewers based on the existing slopes. Slopes for extension sewers assumed construction paralleling the ground surface. The alignments and pipe diameters indicated in this report are preliminary and can be used as a guide in planning. The precise alignments and pipe sizes and slopes would be determined during design based on the projected year 2025 flows for the 25-year storm event.

Project 03-01 is included to show the cost of replacement to convey 25-year storm flows. An infiltration and inflow reduction program in the area of the stockyards and Morrell plant would likely eliminate this project and reduce costs of upgrading the Brandon Pump Station and force main.

Project 08-01 is included to show the cost of replacement but this entire project would be relieved by increasing the capacities of the diversion at 41st and Duluth (manhole 08E0007) and the diversion at Cliff Avenue and Pam Road (manhole 08C0005).

Project 10_02 is included to show the cost of replacement but this entire project cost could be saved by abandoning Pump Station 209 at 9th and Kiwanis and conveying the projected flows of 13 cfs by gravity to Pump Station 215. Pump Station 215 will require upgrade to accommodate the additional flows.

Priority 2 relief sewers required for development are listed in Table 7-2. The reliefs were sized to convey the additional flow from the projected growth area basins. Project 04B-01 includes the cost of replacing the surcharged sewer upstream of manhole 04F0006. In manhole 04F0006 a proposed diversion to manhole 04GB007 would relieve the surcharge on all downstream pipes but increase the surcharging on project 04G-01.

The capital cost shown in Table 7-2 for project 04G-01 includes the additional capacity required to convey 4 cfs of diverted storm flow from project 04B-01.

Table 7-2
Recommended Relief Sewers-Priority 2

CIP Name	Upstream Elevation	Downstream Elevation	Range of Existing Diameters (in.)		Range of Relief Diameters (in.)		Total Length (ft.)	Construction Cost $	Capital Costs $
04B_01	1,461.48	1,401.66	8	21	15	27	8,057	754,000	1,131,000
04C_01	1,379.18	1,373.04	36	40	48	48	1,400	342,000	513,000
04G_01	1,393.53	1,384.35	8	12	24	24	2,256	211,000	316,500
07A_02	1,464.83	1,405.78	15	18	24	24	7,970	828,000	1,242,000
10_02(1)	1,417.52	1,329.38	24	36	36	36	8,120	1,310,000	1,965,000
13_01	1,422.95	1,409.18	8	12	15	21	7,300	608,000	912,000
						Priority 2 Totals	**35,103**	**4,053,000**	**6,079,500**

(1) Project included to show cost, see text for alternative recommended.

Project 07A-02 includes the cost of replacing the surcharged sewer upstream of manhole 07J0013. In manhole 07J0013 a proposed diversion to manhole 11E0007 on project 07J-02 would relieve the surcharge on all downstream pipes but increase the surcharging on project 07J-02. The capital cost shown in Table 7-3 for project 07J-02 includes the additional capacity to convey 2.5 cfs of diverted storm flow from project 07A-02. The city should monitor project 07J-02 after completion of project 07A-02 and consider upgrading 07J-02 to a priority 2 project.

Priority 3 relief sewers required for the projected 2025 development are listed in Table 7-3.

Table 7-3
Recommended Relief Sewers-Priority 3

CIP Name	Upstream Elevation	Downstream Elevation	Range of Existing Diameters (in.)		Range of Relief Diameters (in.)		Total Length (ft.)	Construction Cost $	Capital Costs $
05_03	1,466.28	1,426.42	10	15	18	18	4,712	405,000	607,500
05B_03	1,448.99	1,441.78	8	12	10	15	3,039	221,000	331,500
06_02	1,446.91	1,394.65	8	18	10	24	9,731	869,000	1,303,500
07A_03	1,503.75	1,485.41	8	14	10	15	4,585	316,000	474,000
07J_01	1,404.67	1,398.50	30	30	42	42	5,292	1,070,000	1,605,000
07J_02	1,399.19	1,380.40	8	42	48	48	12,856	3,182,000	4,773,000
09_04	1,416.17	1,407.71	18	24	21	27	8,839	1,025,000	1,537,500
14A_01	1,418.04	1,403.69	24	30	30	30	8,689	1,222,000	1,833,000
15_02	1,421.74	1,404.67	15	15	15	18	4,268	356,000	534,000
SRCI_03D	1,393.71	1,389.93	36	42	54	54	4,176	1,163,000	1,744,500
SRCI_03E	1,386.84	1,377.21	36	42	54	54	17,111	5,094,000	7,641,000
						Priority 3 Totals	**83,298**	**14,923,000**	**22,384,500**

Projects that are marginally surcharged or may be affected by incomplete information are listed in Table 7-4. This list includes sewers that the model shows are overloaded by a storm event, but are not recommended for improvements without further investigations, such as localized flow monitoring. The depth of surcharging of sewer lines listed on the Watch List should be measured periodically.

Table 7-4
Recommended Relief Sewers – Watch List

CIP Name	Upstream Elevation	Downstream Elevation	Range of Existing Diameters (in.)		Range of Relief Diameters (in.)		Total Length (ft.)	Construction Cost $	Capital Costs $
01_02	1,490.68	1,295.50	8	15	10	21	8,756	690,000	1,035,000
04A_01	1,390.18	1,380.63	12	18	15	21	1,865	154,000	231,000
04H_01	1,394.07	1,384.68	8	12	18	18	2,212	190,000	285,000
06A_03	1,419.88	1,388.88	8	10	12	15	1,870	121,000	181,500
06B_03	1,427.28	1,394.40	8	18	10	15	1,473	99,000	148,500
06C_02	1,405.08	1,403.18	12	12	18	18	993	85,000	127,500
06C_03	1,401.98	1,390.78	8	8	15	15	2,788	221,000	331,500
07B_03	1,461.88	1,404.27	8	30	10	42	5,280	498,000	747,000
09A_02	1,426.17	1,406.76	8	18	10	30	10,202	965,000	1,447,500
10A_01	1,415.28	1,405.83	8	8	12	18	1,073	75,000	112,500
10C_02	1,419.24	1,418.35	20	20	30	30	850	120,000	180,000
11_01	1,421.42	1,406.60	10	12	15	15	5,368	426,000	639,000
11A_01	1,406.50	1,399.40	10	10	21	24	2,554	246,000	369,000
11B_01	1,410.21	1,404.38	10	10	12	18	2,039	171,000	256,500
12A_02	1,412.56	1,409.46	10	10	12	12	1,123	78,000	117,000
12B_02	1,410.43	1,406.13	8	18	10	10	653	41,000	61,500
14B_01	1,444.73	1,420.49	21	21	27	27	13,746	1,750,000	2,625,000
14C_01	1,448.88	1,444.73	8	8	12	12	1,038	72,000	108,000
14C_02	1,447.73	1,445.32	18	18	21	21	1,891	178,000	267,000
16_03	1,439.07	1,431.44	21	21	24	24	4,572	475,000	712,500
						Watch List Totals	**70,346**	**6,655,000**	**9,982,500**

7.3 Existing Pump Station and Force Main Improvements

Pump station capacities were evaluated based on existing and projected peak flow conditions, no I/I removal in the drainage system, and the existing firm pumping capacity. Recommendations for expansion or replacement of a pump station are based on whether the flow/capacity ratio of the station equals or exceeds 2.0. Expansion is suggested when flow/capacity ratio is between 1.5 and 2.0. Replacement is recommended when flow/capacity ratio is higher than 2.0. When flow/capacity ratio is less than 1.5, no improvement is recommended. Proposed force mains are assumed to be paralleled; however, whether the force mains are paralleled or replaced will be determined during detailed design. This study includes no consideration of the present physical configuration or the condition of the pumping station; therefore, detailed review of whether to expand or replace each pumping station should be carried out as part of detailed design.

Six pump stations are listed in Table 7-5. The Brandon pump station is impacted by the Basin 3 inflow estimated at 65 mgd. If the peak 25-year storm flow of 89 mgd could be reduced by 65 mgd (requires 100 percent reduction of inflow from Basin 3 or equivalent), then upgrading the pump station firm capacity is unnecessary.

The existing capacities, the design flow, and the recommended firm capacities of pumping station improvements are listed in Table 7-5. The total expansion or replacement cost is projected to be $3.77 million without inflow reduction in Basin 3 or $1.72 million with Basin 3 inflow eliminated. Table 7-5 shows the cost of upgrading PS 215 excluding flow from PS 209 which was assumed to continue service through 2025. However, if PS 209 is abandoned and the flow diverted to PS 215, the additional capital cost of upgrading PS 215 is $661,000 compared to the total capital cost of upgrading PS 209 and relief project 10_02 of $2,691,000.

Table 7-6 shows the recommended force main improvements. The 36 inch force main from the Brandon pump station has a capacity of 55 mgd at 12 feet per second. If the peak 25-year storm flow of 89 mgd could be reduced by 65 mgd (requires 100 percent reduction of inflow from Basin 3 or equivalent), then the upgrade is unnecessary. The total force main improvement cost is projected to be $2.31 million without reducing the inflow in Basin 3 or $5,000 with Basin 3 inflow eliminated.

Table 7-5
Recommended Pump Station Improvements

Manhole	Structure Name	Location	Firm Capacity (mgd)	Design Flow (mgd)	Pumping Capacity Improvement (mgd)	Project Type	Probable Construction Cost $	Probable Capital Cost $
PS236	Renner #4	25775 Lindburg Ave.	0.12	0.61	0.49	Replace	134,000	201,000
BRANDON	Brandon(1)	3300 E. Rice Street	40.61	88.83	48.22	Expand	2,516,000	3,774,000
PS206	Burnside	1800 Burnside	0.84	1.29	0.45	Expand	93,000	140,000
PS209	9th & Kiwanis(1)	101 N. Kiwanis	4.11	8.19	4.08	Expand	484,000	726,000
PS215	Sioux River North	3301 W. 12th St.	14.4	23.91	9.51	Expand	837,000	1,256,000
PS218	Tuthill Park	3500 S. Blauvelt	15.2	27.44	12.32	Expand	1,013,000	1,520,000
						Total	**5,077,000**	**7,617,000**

(1) Project included to show cost, see text for alternative recommended.

Table 7-6
Recommended Force Main Improvements

Station Name	Existing Diameter	Capacity Based on 12 ft. per sec. (mgd)	Design Flow Year 2025, 25-Year Storm Event (in)	Type of Relief	Diameter Based on 6 ft. per sec. (in)	Length	Probable Construction Cost $	Probable Capital Cost $
BRANDON(1)	36	54.82	88.83	Parallel	42	12,257	1,540,000	2,310,000
PS209(1)	8	2.71	8.19	Parallel	16	64	3,000	5,000
					Totals	**12,321**	**1,543,000**	**2,315,000**

(1) Project included to show cost, see text for alternative recommended.

7.4 Growth Areas Sewers

7.4.1. Growth Area Trunk Sewers

New interceptor sewers and pumping stations will be required to serve portions of the study area that are currently not developed. The sewers are indicated as Growth Area sewers on Figure 7-1. These gravity sewers are the same for each of the Plan alternatives discussed in Chapter 6. The Plan alternatives differ by the pump station sizes and forcemain lengths. The preliminary layouts of recommended interceptor sewers are sized to serve ultimate development. The indicated locations are preliminary, and should only be used as a guide for planning purposes. More precise alignments can only be defined following a detailed alignment survey performed under a design contract.

The construction of new interceptor sewers is dependent on development within the study area. For this study, the City provided Growth Area development phasing which is shown on Figure 7-1 and listed by project in Table 7-7. For a complete listing of proposed extensions, see Appendix I. Priority "0" in Table 7-7 means that these sewers are to be built on an as needed basis to provide expansion for near term growth.

Table 7-7
Growth Area Extensions List

CIP Name	Downstream Node	Range of Sizes (in)	CIP Length	Construction Cost $	Priority	Development Phase
G11_01	G11F0005	8 - 42	19,612	3,191,000	0	2003 - 2007
G13_01	G13C0001	8 - 8	655	38,000	0	2003 - 2007
G16_01	G16D0001	12 - 18	5,903	473,000	0	2003 - 2007
G1B_01	G19G0001	8 - 8	2,110	122,000	0	2003 - 2007
G20_02	G200015	8 - 48	18,741	1,934,000	0	2003 - 2007
G26_01	G26C0007	8 - 15	4,413	282,000	0	2003 - 2007
G5H_01	05HI008	12 - 18	3,452	286,000	0	2003 - 2007
G6I_01	G6K0008	8 - 15	7,351	566,000	0	2003 - 2007
G6J_01	06J0013	8 - 8	2,497	160,000	0	2003 - 2007
G6K_01	06J0019	8 - 24	5,920	493,000	0	2003 - 2007
G6L_01	G6K0001	8 - 10	4,118	260,000	0	2003 - 2007
G7L_01	07L0008	8 - 12	7,805	513,000	0	2003 - 2007
G7Q_01	07Q0008	12 - 12	1,580	120,000	0	2003 - 2007
G7S_01	07R0026	8 - 8	3,383	201,000	0	2003 - 2007
G9D_01	09FC003B	12 - 18	2,262	194,000	0	2003 - 2007
G9E_01	09ED006	8 - 8	2,798	171,000	0	2003 - 2007
G18_02	G18B0013	8 - 48	53,293	5,048,000	1	2003 - 2007
G19_02	G19A0001	8 - 36	57,515	4,545,000	1	2003 - 2007
G21_02	G21A0001	8 - 18	27,841	1,898,000	1	2003 - 2007
G26_00	G26B0005	8 - 21	11,522	744,000	1	2003 - 2007
G13_02	G13A0001	8 - 42	21,420	2,837,000	2	2008 - 2015
G14_02	G14A0006	8 - 42	67,027	5,201,000	2	2008 - 2015
G15_02	G15C0001	8 - 42	36,621	4,012,000	2	2008 - 2015
G16_02	G16G0001	8 - 18	13,274	964,000	2	2008 - 2015
G17_02	G17C0001	8 - 21	17,131	1,277,000	2	2008 - 2015
G22_02	G22A0001	8 - 27	25,174	1,814,000	2	2008 - 2015
G23_02	G23A0001	27 - 36	12,482	1,917,000	2	2008 - 2015
G25_02	GLS_WWTP	8 - 42	40,376	3,629,000	2	2008 - 2015
G26_02	G26C0001	15 - 24	10,321	942,000	2	2008 - 2015
G7M_02	07L0001	8 - 15	9,981	683,000	2	2008 - 2015
G14_03	G14G0002	8 - 42	94,905	10,121,000	3	2016 - 2025
G15_03	G15E0002	8 - 30	43,332	3,819,000	3	2016 - 2025
G19_03	G19G0002	8 - 15	14,815	939,000	3	2016 - 2025
G23_03	G23A0004	8 - 15	23,250	1,511,000	3	2016 - 2025
G26_03	GFM_JCT1	8 - 36	35,057	2,980,000	3	2016 - 2025
G27_03	GFM_JCT2	8 - 27	56,248	4,504,000	3	2016 - 2025
G28_03	05F0012	8 - 27	34,905	3,051,000	3	2016 - 2025
G29_03	G29A_LS	8 - 33	10,504	962,000	3	2016 - 2025
		Totals	**799,090**	**72,402,000**		

7.4.2. Growth Area Pumping Stations and Force Mains

Pumping stations and force mains will be required to transport wastewater for the eastern and southeastern basins to the WWTF. Plan alternatives discussed in Chapter 6 differ by the sizes of pump stations and forcemains. Plan 3 facilities, as described in Chapter 6, are listed in Tables 7-8 and 7-9.

Table 7-8
Recommended Pump Station Improvements

Structure Name	Design Flow (mgd)	Design Flow (cfs)	Probable Construction Cost[(1)] $	Probable Capital Cost $
G28D_LS	2.62	4.05	835,000	1,252,500
G27A_LS	2.97	4.6	947,000	1,420,500
G29A_LS	1.22	1.88	486,000	729,000
G26A_LS	5.31	8.22	1,480,000	2,220,000
G20_LS	9.02	13.96	2,012,000	3,018,000
GLS_WWTP	13.30	20.59	2,199,000	3,298,500
Total			**7,959,000**	**11,938,500**

[(1)]Based on interpolation of Pump Station Cost Curve, Appendix J

Table 7-9
Recommended Force Main Improvements

Structure Name	2025 Design Flow (mgd)	2025 Design Flow (in)	Force Main Unit Cost ($/ft.)	Length (ft.)	Probable Construction Cost ($)	Probable Capital Cost ($)
G28D_LS	2.62	12	36	18,670	673,000	1,010,000
G27A_LS	2.97	12	36	12,870	464,000	696,000
G29A_LS	1.22	8	24	5,545	131,000	197,000
G26A_LS	5.31	16	47	1,750	83,000	124,000
G20_LS	9.02	24	71	27,800	1,976,000	2,964,000
G29A_LS +G26A_LS	6.53	20	60	7,585	453,000	680,000
G27A_LS + G29A_LS + G26A_LS	9.50	24	71	15,520	1,103,000	1,655,000
GLS_WWTP	13.30	36	108	200	22,000	32,000
Totals				**89,940**	**4,905,000**	**7,358,000**

[(1)]Based on Force Main Cost Basis, Appendix J

7.5 Summary of Costs

The summary of the costs includes the cost of implementing the Growth Area Wastewater Management Plan, the cost of implementing growth area sewer extensions, and the relief sewer projects.

Table 7-10
Implementation Plan Project Cost Summary

Priority	Probable Construction Cost $	Probable Capital Costs $
Priority 1, 2003-2007		
Relief Sewers	11,408,000	17,112,000
Pumping stations	4,211,000	6,316,500
Force Mains	1,997,000	2,996,000
Basin 3 Inflow Reduction[1]	67,000	100,000
Growth Area Extensions	21,239,000	31,858,500
Total Priority 1	38,922,000	58,383,000
Priority 2, 2008-2015		
Relief Sewers	2,743,000	4,114,500
Pumping stations	837,000	1,256,000
Force Mains	0	0
Growth Area Extensions	23,276,000	34,914,000
Total Priority 2	26,856,000	40,284,500
Priority 3, 2016-2025		
Relief Sewers	14,923,000	22,384,500
Pumping stations	4,854,000	7,281,000
Force Mains	2,907,000	4,360,500
Growth Area Extensions	27,887,000	41,830,500
Total Priority 3	50,571,000	75,856,500
Grand Total	**116,349,000**	**174,524,000**

[1]Assumed City cost for private sector inflow source removal program.

7.6 Sewer System Management Plan

To improve the performance of the sewer system and to develop a database for analysis of the sewer system, a sewer system management plan could be developed. This plan should include the following components:

- Installation of a network of rain gauges and flow meters.
- Annual evaluation of flow and rainfall data collected. Flow data collected from the permanent meters should be analyzed in conjunction with the rain gauge data for post-rehabilitation evaluation of I/I rates and for subsequent modeling and planning.
- An annual program of cleaning and televising sewer lines, and other system inspections as needed.
- System inspections to identify areas in need of rehabilitation.