October 31, 2019

Mr. Steve Gorszczyk
Public Works Director
PO Box 80
Siloam Springs, AR 72761

Subject: City of Siloam Springs, Arkansas
Wastewater Master Plan

Dear Mr. Gorszczyk:

In accordance with the February 20, 2018 Engineering Agreement, RJN Group, Inc. is pleased to submit this draft report for the above referenced project.

This report provides information on the development of the hydraulic model including flow data and the application of this information to the analysis, a brief discussion of population projections, model build and calibration, modeling results, and a projected capital improvement program.

We appreciate the opportunity to work with the City of Siloam Springs and the excellent cooperation from the City's staff throughout the project. We look forward to working with the City of Siloam Springs in the future. Should you have any questions, please call.

Respectfully Submitted,

RJN GROUP, INC.

Shannon Jones
Senior Project Manager

Enclosure

WASTEWATER COLLECTIONS SYSTEM

MASTER PLAN

October 2019

City of Siloam Springs

Siloam Springs, Arkansas

STATE OF ARKANSAS
REGISTERED PROFESSIONAL ENGINEER
No. 11053
SHANNON W. JONES

10/31/19

I hereby certify that this report was prepared under my direct supervision and that I am a duly registered Professional Engineer under the laws of the State of Arkansas.

Date: 10/31/2019 Registration No.: 11053

CERTIFICATE OF AUTHORIZATION
RJN GROUP, INC.
No. 1523
ARKANSAS

EXECUTIVE SUMMARY

Background and Objectives

In September 2017 RJN Group, INC. (RJN) was retained by the City of Siloam Springs (City), Arkansas to perform a comprehensive sanitary sewer collection system study for the city of Siloam Springs, Arkansas. The intent of the study is to provide flow monitoring, lift station evaluations, development and calibration of hydraulic model, development of future flows, hydraulic capacity analysis, and a wastewater collection system master plan.

The City of Siloam Springs owns and operates a wastewater collection system containing approximately 100 miles of gravity sewer main, 18 lift stations, and approximately 5.7 miles of force main which flows to the City of Siloam Springs Wastewater Treatment plant currently serving a population of approximately 16,991 people according to 2018 United States Census Bureau data.

Master Plan Development

Project Administration and Management

RJN coordinated with City staff on a regular basis during the study to update previous investigative work, coordinate upcoming work, and receive any input from staff. General consultation was performed with the appointed City representatives on an as-needed basis. RJN coordinated obtaining existing information from the City that included: future planned developments, pump curves, parcel and contour maps, ultimate build-out boundaries, and design storm event.

Flow Monitoring

The flow monitoring program collected both dry and wet-weather flows for a 91-day period from April 5, 2018 to July 5, 2018. A total of ten (10) flow meters and five (5) rain gauges were installed strategically throughout the City’s wastewater collection system.

The raw data collected from the flow meters and rain gauges was processed and edited into a final processed data deliverable to be utilized for hydraulic analysis. Final tabular data has been provided electronically to the City via email and will be included on the flash drive attached to this report. Depth, velocity, and flow rate hydrographs for dry and wet-weather were developed and are included in this report. Additionally, the final processed data was utilized for the hydraulic model development which determined corresponding rainfall intensities for areas tributary to all flow monitoring locations.

Lift Station Evaluation

Lift station evaluations through drawdown testing were performed to compare actual pumping rates of the lift station to the original pump curves. A total of 18 lift stations exist in the City of Siloam Springs collection system. Recommendations to upgrade, expand, or eliminate lift stations after evaluation were performed.

Hydraulic Model Software

Infoworks Integrated Catchment Modeling (ICM) was utilized as the platform for developing an accurate representation of the the City's wastewater collection system. The software gives RJN the ability to create a single dynamic hydraulic model containing the pertinent features existing in the environment with a single, unified hydrology.

Hydraulic Model Development

RJN identified 10 inch and larger sewer segments and collaborated with the City of Siloam Springs to determine the critical sewer interceptors for hydraulic capacity analysis. Rim elevation GPS points for each manhole located on the interceptor were provided by the City of Siloam Springs. A total of 297 manholes were surveyed for the development of the hydraulic model. Measure down data was collected to gather invert elevations for sewer lines for input into the hydraulic model. Additionally, routing and connectivity between sewer lines was verified during measure down inspections.

All remaining structures to aid in building the hydraulic model were estimated based on record drawings and on digital terrain model created in Infoworks ICM utilizing City provided 2-foot contours.

Development of Future Flows

The master plan, as shown later in this report, will calculate the new service area population growth and evaluate ultimate build-out based on zoning and other land uses. The City aided in the development of future flows by supplying RJN Group with future planning land use of undeveloped areas and known, planned development. This hydraulic analysis will utilize processed wastewater flows attained during the study period mentioned in this report and allowed the City's system to be modeled, calibrated, projected, and alternatives evaluated for ultimate build-out conditions.

Hydraulic Capacity Analysis

City of Siloam Springs wastewater collection system was analyzed for existing and future dry and wet weather conditions. Normal wastewater production and dry weather infiltration flow components versus pipe capacity was analyzed to determine if any potential dry weather hydraulic capacity problems existed. RJN consulted with the City and it was decided moving forward to use a design storm of 2-year frequency/24-hour duration with a total rainfall depth of 4.10 inches. Design scenarios for the Siloam Springs sewer network were to convey all flows to the WWTP with a secondary consideration to maintain a level of surcharge not to exceed three feet below manhole rim elevations.

Table ES.1 WASTEWATER TREATMENT PLANT FLOWS				
Descriptions	ADWF (MGD)	Peak Hour ADWF (MGD)	Peak Hour 2-yr/24-hr (MGD)	Peak Hour 2-yr/24-hr (MGD)
Existing System	1.77	2.38	12.13	24.34*
Buildout System	3.72	4.88	15.80	36.93

**Value does not account for 0.57 MG overflow in the system*

Capital Improvement Plan

SSES Recommendations

Excessive I/I was identified in four (4) basins. SSES projects may identify sources of I/I and be a cost effective measure to reduce wastewater flows during wet-weather. Reducing I/I will reduce the recommended capacity upgrades. The basins recommended for future SSES are identified in Table ES.1 below.

Table ES.2 RECOMMENDED SSES PRIORITY BY BASIN		
Priority	Basins	Total Length (lf)
1	SS-08	41,763
	SS-01	21,737
	SS-02	96,578
	SS-07	62,904
Total		**222,982**

Budgetary pricing for full SSES is approximately $3.50 per linear foot for a total of $780,437.

Sewer Line Recommendations

A summary of phased sewer line improvements is shown in Table ES.3 below.

Table ES.3
CAPITAL IMPROVEMENT PLAN SUMMARY

Category	Estimated Cost ($)
Phase 1 gravity	$7,067,763
Phase 2 force main	$508,496
Phase 2 gravity	$1,346,108
Phase 3 gravity extensions	$2,383,935
Total gravity/force main	**$11,306,302**

Lift Station Recommendations

Lift station pump upgrades will be site specific. Various factors to consider depending on the pump manufacturer, power requirements, site conditions, etc. Estimated costs for pump station upgrades are listed in Table ES.4 below.

Table ES.4
LIFT STATION IMPROVEMENT SUMMARY

Category	Estimated Cost ($)
Dogwood LS	$30,000
Mt. Olive LS	$60,000
Stone Crest LS	$45,000
Villa View LS	$75,000
Maloree LS	$50,000
Total	**$260,000**

Capital Improvement Plan Schedule

Timing for the capital improvement plan is directly dependent on sewer rates to support the expenditures. It is recommended that the capital costs be utilized in the next sewer rate analysis and a 10-year schedule be developed.

Cleaning and Televising

As part of the future collection system maintenance recommendations, cleaning & televising 20% of the wastewater collection system is recommended annually. Cleaning and/or televising every five years is a common guideline used by the EPA and other agencies. The benefits include dry weather overflow (blockage) prevention and allowing continual observation of the system. The operators will also be able to keep a plan in place in regard to age and condition of the system. Cleaning and televising also helps keep mapping activities up to date with measured distances between manholes.

Hydraulic Model Maintenance and Updating

The City has made an investment to build and calibrate the wastewater collection system model. The hydraulic model will need to be kept up to date periodically by including infrastructure improvements and performance checks with the recommended permanent flow meters. Ideally, the model would be kept up to date and used as a planning tool for analyzing each major development for both existing and ultimate build-out scenarios at the design storm. Also, the model should be updated with all constructed sewer improvements on an annual or bi-annual basis with record drawings. This also serves a dual purpose to keep the wastewater collection system GIS map up to date.

It is recommended the wastewater collection system model be re-calibrated approximately every 5-7 years depending on the results of the annual or bi-annual updates. The re-calibration of the model will include installation of temporary flow meters and rain gauges to supplement the City's permanent flow meter network. Sub-basins and catchments will be reviewed and updated. Population and land use data will also be evaluated and updated to reflect current conditions.

Maintaining a model will involve adding new sewer lines to the model as a result of development, sewer capacity enhancements, adding storage, updating WWTP inlet capacity, and lift station capacity improvements as they are built.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF EXHIBITS

LIST OF APPENDICES

CHAPTER 1 – INTRODUCTION

Background and Objectives

In September 2017 RJN Group, INC. (RJN) was retained by the City of Siloam Springs (City), Arkansas to perform a comprehensive sanitary sewer collection system study for the city of Siloam Springs, Arkansas. The intent of the study is to provide flow monitoring, lift station evaluations, development and calibration of hydraulic model, development of future flows, hydraulic capacity analysis, and a wastewater collection system master plan.

The City of Siloam Springs owns and operates a wastewater collection system containing approximately 100 miles of gravity sewer main, 18 lift stations, and approximately 5.7 miles of force main. The flow conveys to the City of Siloam Springs Wastewater Treatment Plant (WWTP) that currently serves a population of approximately 16,991 people according to 2018 United States Census Bureau data. The sanitary sewer collection system for Siloam Springs is displayed in Figure 1.1 on page 1-5.

This plan should be viewed as a dynamic working document, reviewed annually, and updated as conditions in the City's service area change. The Capital Improvement Plan contained herein will assist the organization in prioritizing projects and developing annual budgets. Recommendations identified in this plan should be considered as conceptual only. Additional details and potential alternatives should be further investigated and analyzed in the preliminary engineering phase of each project design.

Scope of Master Plan Development

Project Administration and Management

RJN coordinated with City staff on a regular basis during the study to update previous investigative work, coordinate upcoming work, and receive any input from staff. General consultation was performed with the appointed City representatives on an as-needed basis. RJN coordinated obtaining existing information from the City that included: future planned developments, pump curves, parcel and contour maps, ultimate build-out boundaries, and design storm event.

Flow Monitoring

The flow monitoring program collected both dry and wet-weather flows for a 91-day period from April 5, 2018 to July 5, 2018. A total of ten (10) flow meters and five (5) rain gauges were installed strategically throughout the City's wastewater collection system.

The raw data collected from the flow meters and rain gauges was processed and edited into a final processed data deliverable to be utilized for hydraulic analysis. Final tabular data has been provided electronically to the City via email and will be included on the flash drive attached to this report. Depth, velocity, and flow rate hydrographs for dry and wet-weather were developed and are included in this report. Additionally, the final processed data was utilized for the hydraulic model development which determined corresponding rainfall intensities for areas tributary to all flow monitoring locations.

Flow monitoring is discussed further in Chapter 2 – Flow Monitoring.

Lift Station Evaluation

Lift station evaluations through drawdown testing were performed to compare actual pumping rates of the lift station to the original pump curves. A total of 18 lift stations exist in the City's collection system. Recommendations to upgrade, expand, or eliminate lift stations after evaluation were performed. A full discussion on lift station evaluation is located in Chapter 3.

Hydraulic Model Software

Infoworks Integrated Catchment Modeling (ICM) was utilized as the platform for developing an accurate representation of the the City's wastewater collection system. The software gives RJN the ability to create a single dynamic hydraulic model containing the pertinent features existing in the environment with a single, unified hydrology.

Hydraulic Model Development

Construction of the hydraulic model began in September 2017. RJN identified 10 inch and larger sewer segments and collaborated with the City to determine the critical sewer interceptors for hydraulic capacity analysis. Rim elevation GPS points for each manhole located on the interceptor were provided by the City. A total of 297 manholes were surveyed for the development of the hydraulic model. Measure down data was collected to gather invert elevations for sewer lines for input into the hydraulic model. Additionally, routing and connectivity between sewer lines was verified during measure down inspections.

All remaining structures to aid in building the hydraulic model were estimated based on record drawings and on digital terrain model created in Infoworks ICM utilizing City provided 2-foot contours.

Hydraulic Model Calibration

Pumping capacities of the lift stations were obtained from City provided record drawings and pump curves. Projected flows, along with flow and survey data collected during the flow monitoring period were utilized to calibrate the model for both dry and wet-weather conditions. Residential flows based on gallons per day per capita were projected and commercial/industrial flows were predicted from water usage records supplied by the City. In addition, the following tasks were included in this phase of the project:

1. Select dry-weather period from flow data
2. Develop dry-weather flow unit equivalents
3. Calibrate model to dry-weather diurnal weekday and weekend conditions
4. Select storm events with varying rainfall intensities
5. Calibrate model based on a selected storm event

RDII Analysis

The hydraulic model utilizes the dry and wet-weather calibration to perform RDII analysis for the collection system. Dry-Weather Flow (DWF) is the flow during dry weather and is considered to consist of sanitary sewer flow and permanent groundwater infiltration. Wet-Weather Flow (WWF) is sanitary sewer flow recorded immediately after a storm event and includes inflow and infiltration (I/I). Wet-weather flow can be compared to dry-weather flow to calculate peaking factors resulting from I/I. Additionally, Rain-Derived Inflow and Infiltration (RDII) analysis is be performed to calculate the I/I effect modeled in the collection system and determine a prioritization of basins requiring additional invesetigation to reduce I/I.

Development of Future Flows

The master plan, as shown later in this report, will calculate the new service area population growth and evaluate ultimate build-out based on zoning and other land uses. The City aided in the development of future flows by supplying RJN with future planning of land use for undeveloped areas and known, planned development. This hydraulic analysis will utilize processed wastewater flows attained during the study period mentioned in this report and allowed the City’s system to be modeled, calibrated, projected, and alternatives evaluated for ultimate build-out conditions.

Hydraulic Capacity Analysis

City of Siloam Springs wastewater collection system was analyzed for existing and future dry and wet weather conditions. Normal wastewater production and dry weather infiltration flow components versus pipe capacity was analyzed to determine if any potential dry weather hydraulic capacity problems existed. RJN consulted with the City and it was decided moving forward to use a design storm of 2-year frequency/24-hour duration with a total rainfall depth of 4.10 inches. Utilizing the selected design storm, the following tasks were included in this phase:

1. System hydraulic analysis under existing population conditions

2. System hydraulic analysis under ultimate build out
3. Estimate average daily dry flow, peak dry flow and peak wet-weather flow rates for the WWTP under both existing and future population conditions

Collection System Master Plan and Report

The Collection System Master Plan consisted of several tasks and evaluation for the existing collection system performed and proposing necessary collection system improvement alternatives. The following tasks were included in this phase:

1. Prioritization of each monitored area for future SSES and rehabilitation efforts based on RDII analysis
2. Necessary capacity improvements for full conveyance to WWTP
3. Peak flow management
4. Peak wastewater flow to WWTP
5. Develop a Capital Improvement Plan (CIP)
6. Delivery of InfoWorks ICM model

Reporting included the creation of draft and final master plan reports summarizing the results of all previous tasks. The reporting describes work performed during various tasks, procedures, and methodologies used, alternatives evaluated, required improvement plan, and suggested maintenance plan as well as engineer's opinion of probable costs.

A flash drive is enclosed with this report and following items are included on it:

1. Master Plan Report
2. Exhibits
3. Appendicies
4. Infoworks ICM Hydraulic Model files
5. ArcGIS shape files with updated wastewater collection system map

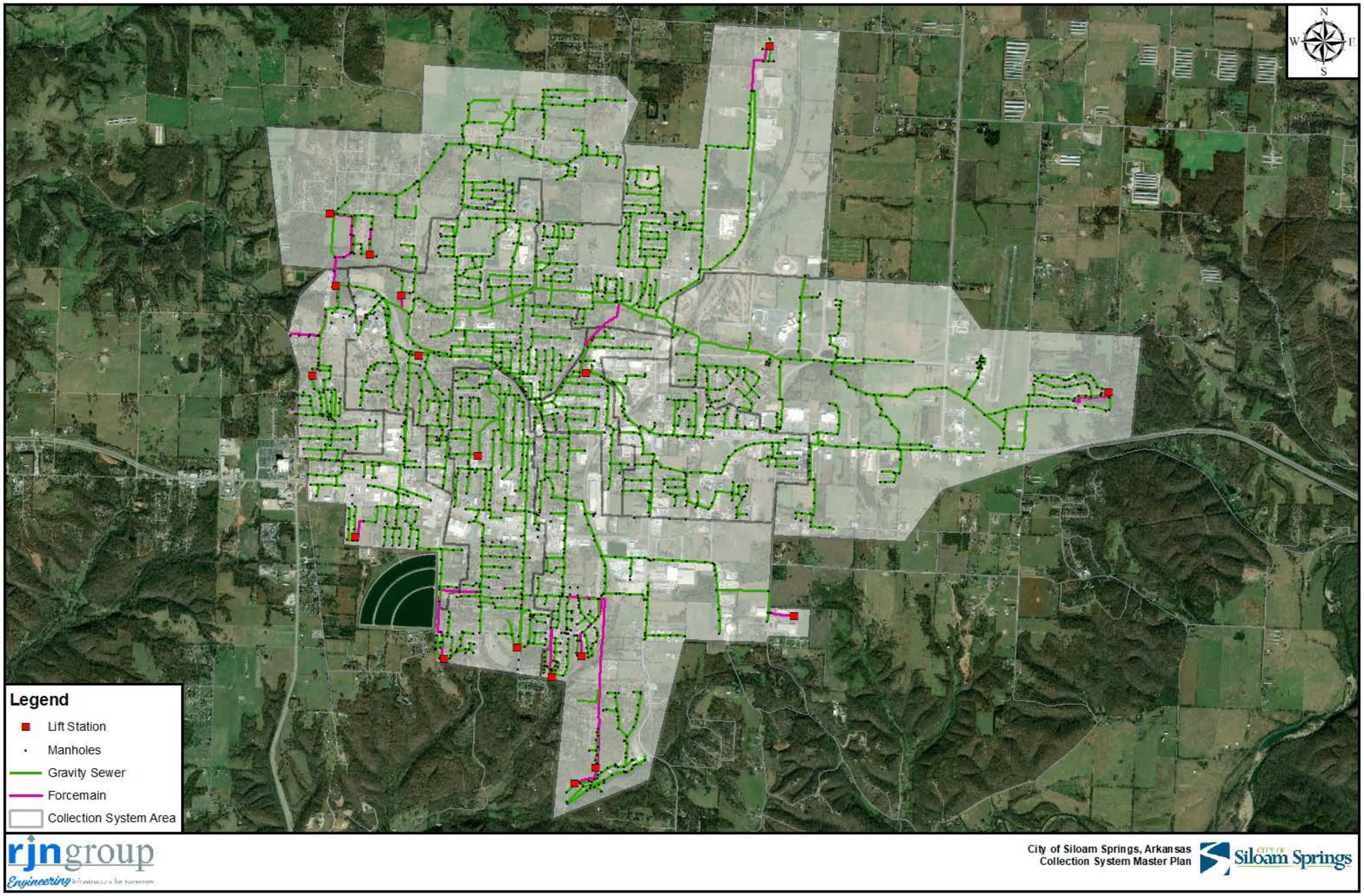


Figure 1.1
Siloam Springs Sanitary Sewer Collection System

Terminology/Definitions/Acronyms

Average Dry-Weather Flow (ADWF) – Flow during a period of extended dry weather (7 to 14 days) and seasonally high groundwater. Flow includes sanitary sewer flow and permanent groundwater infiltration and excludes significant industrial and commercial flows (assumes no inflow during dry-weather conditions).

Capital Improvements Plan (CIP) – A financial and scheduling plan which identifies needed wastewater collection system capital projects with regards to this project.

Dry-Weather Flow (DWF) – All flow in a sewer (includes sanitary flow and permanent groundwater infiltration) minus flow caused directly by rainfall. Measuring during a period of extended dry weather (7 to 14 days) and seasonally high groundwater.

Environmental Protection Agency (EPA) – An agency of the United States Federal Government which was created for the purpose of protecting human health and the environment by writing and enforcing regulations based on laws passed by Congress.

Head – Pressure head of a fluid due to the pressure exerted on a pipe and measured in units of feet (1-foot head = 0.4335 lb/in2 or psi).

Hydrograph – A graph showing depth, velocity, flow, and rain measurements with respect to time.

Infiltration – Water other than sanitary sewer wastewater entering a sewer system due to rain, from sources such as pipe joints, sewer line defects, defective manhole walls, benches, and pipe seals.

Inflow – Water other than sanitary sewer wastewater entering a sewer system from sources such as roof downspouts, foundation drains, yard drains, area drains, manhole covers, defective services, and cross connections between storm and sanitary sewers.

Infoworks Integrated Catchment Modeling (ICM) **–** Is an integrated hydraulic modeling platform created by Innovyze used for this project.

Lift Station (LS) – Wastewater lift station used for pumping wastewater from a lower to higher elevation.

Million Gallons (MG) – Unit of volume.

Million Gallons per Day (MGD) – Unit of measure for flow rate.

National Oceanic and Atmospheric Administration (NOAA) – Is an American government scientific agency within the United States Department of Commerce focused on conditions of the oceans and the atmosphere.

Peak Hourly Wet-Weather Flow (PWWF) – The highest one-hour flow during a significant rain event.

Peaking Factor – The ratio of peak hourly flow to average dry-weather flow.

Permanent Groundwater Infiltration (GWI) – Measured during average dry-weather flow period. The average of the low nighttime flows (usually midnight to 6 A.M.) per day for the same time period, minus significant industrial or commercial nighttime flows.

Rain-Derived Inflow and Infiltration (RDII) – A portion of rainfall entering into a sewer system in the form of inflow and infiltration and becomes excess flow.

Sanitary Sewer Overflow (SSO) – An event when surcharged sanitary sewer flow reaches a height above the top of a sanitary sewer structure and excess sanitary sewer flow is discharged out of the sanitary sewer system as bypass.

Wet-Weather Flow (WWF) – The highest daily flow during and immediately after a significant storm event. It includes sanitary flow, infiltration, and inflow.

Wastewater Treatment Plant (WWTP) – Biological, chemical, and physical process facility for removing contaminants from wastewater.

CHAPTER 2 – FLOW MONITORING

Description of the Study Area

RJN Group, Inc. performed the flow monitoring program from April 5, 2018 to July 5, 2018. The City of Siloam Springs' sanitary sewer system was divided into ten (10) basins for the purpose of evaluating flow characteristics in each basin area.

It was observed that residential and industrial development had spread beyond the original 1994 sewer basin boundaries, and it was determined that an update to the City's basin boundaries was necessary for 2018 Collection System Master Plan. The sewer flow network was analyzed, and new basins were developed prior to Flow Monitoring beginning.

The collection system contains approximately 527,000 linear feet of sanitary sewer pipe and approximately 2,100 manholes. Table 2.1 provides a summary of each basin and associated footages that are contained within each sewershed.

Table 2.1
SUMMARY OF METER BASINS

Meter Basins	Linear Footage[1/]
SS-01	21,737
SS-02	96,578
SS-03	37,871
SS-04	28,444
SS-05	68,992
SS-06	30,181
SS-07	62,904
SS-08	41,763
SS-09	67,110
SS-10	71,174
Total	**526,754**

1/ Linear footage is approximate and based upon City maps

An overall map of the Siloam Springs sanitary sewer collection system can be found in Figure 2.1 on page 2-2 and in Exhibit 1. The resulting sewer flow network is displayed in Figure 2.2 on page 2-3.

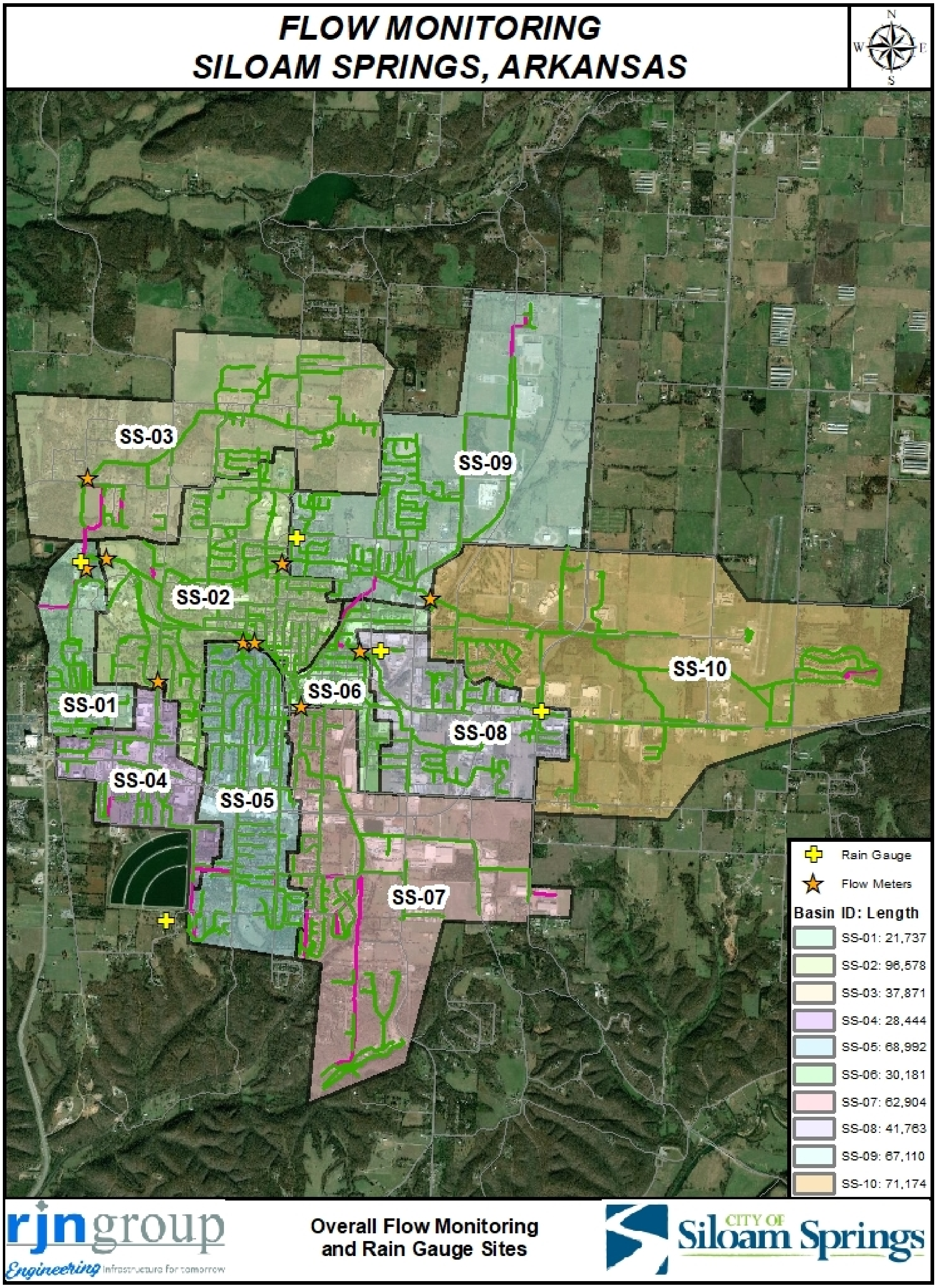


Figure 2.1
Flow Meter and Rain Gauge Site Locations

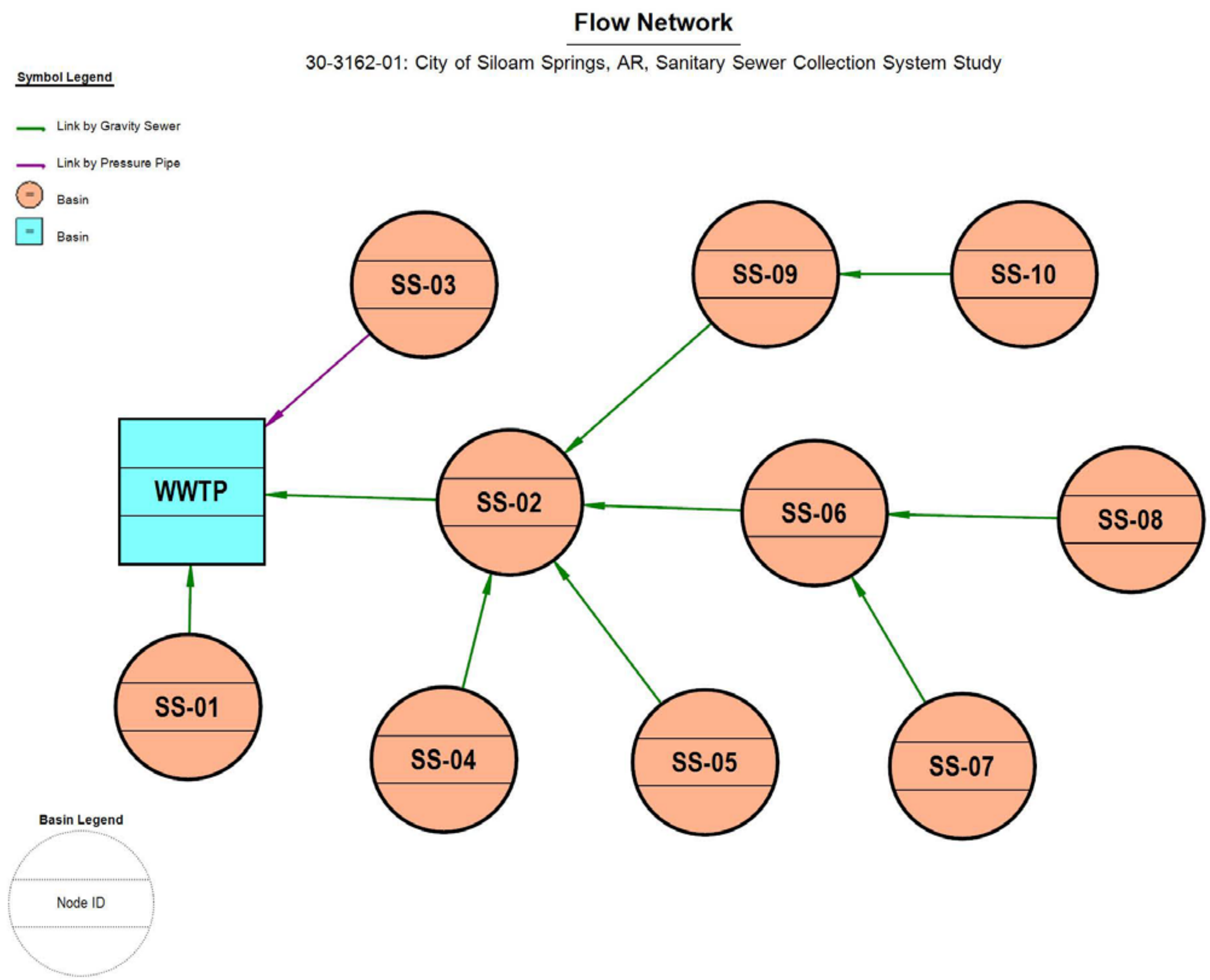


Figure 2.2
Flow Network

Meter Site Selection

Ten (10) flow meter sites were selected. Once the target manhole was identified for a meter site the manhole was investigated to determine if the site had good hydraulics, which is a key for collecting high quality and accurate flow data. In general, it is preferred to have a straight channel where flow is free of turbulence or backwater effects. In some sites it was necessary to install meters where a slight bend in pipe direction existed or the optimum flow characteristics were not prevalent. This was because the location was strategic to the monitoring program. Listed below are other considerations that were investigated before the sites were installed:

- Installation constraints
- Pipe dimensions
- Site accessibility
- Site specific concerns
- Manhole accessibility
- Manhole configuration
- Flow depth range
- Velocity range
- Employee safety
- Surcharging evidence
- Silt Deposition
- Maintainability
- Telemetry constraints
- Sensor survival

Where necessary, the City assisted RJN in locating manholes and providing accessibility.

A copy of the meter site installation forms complete with photos for each meter location, are provided in Appendix A. The photos of each meter site include the site location, interior manhole down shot, and pipe before and after sensor installation. Depth, velocity, and flow rate hydrographs were developed and are included in this report in Appendix B.

Flow Meter Equipment Information

The ADS Triton Plus meter was selected as the gravity flow meter to complete the flow monitoring program. The average accuracy of the meters for the depth measurement is ±0.2 inches and the average accuracy for velocity is ±0.25 fps.

Figure 2.3
Triton Plus Meter and Sensors

Triton Plus Meter Principles

ADS Triton Plus is an open channel flow meter that utilizes Doppler ultrasound technology to sense the peak flow velocity, as well as a redundant depth sensor. The ADS Triton Plus has four primary components: a logger unit that stores the collected data and houses the RTU, the Ultrasonic Depth Sensor, Peak Combo Sensor, and the integrated wireless modem. The Peak Combo Sensor typically mounts at the bottom of the pipe and measures an upward-looking ultrasonic depth, pressure depth, and peak velocity using three independent sensors. One of the three sensors in the Peak Combo Sensor utilizes a pressure transducer to measure pressure depth which will record water level including full pipe and surcharge level. The Ultrasonic sensor is a non-intrusive quad redundant transceiver sensor which will accurately record the depth to within one inch of the sensor's face. The sensors emit signals and receive returned signals from the flow to capture the characteristics of the flow velocity and depth.

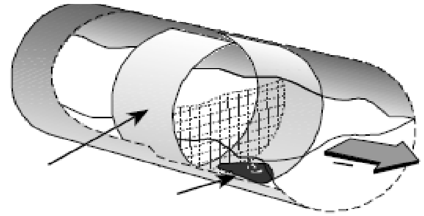


Figure 2.4
Flow is calculated using the Continuity Equation

During surcharged conditions the ultrasonic depth sensors no longer functions, and the level is recorded through the Peak Combo Sensor that contains the pressure sensor. Flows are calculated using the continuity equation.

Flows are calculated using the continuity equation, which is expressed as $Q = AV$, where Q is flow, A is the cross sectional area, and V is the average velocity of the flow. The meter records velocity and depth at five-minute intervals.

Triton Plus Calibration and Installation:

The ADS Triton Plus must be calibrated on site by entering physical offsets of the sensors and their positions in the meter as well as comparing the depth and velocity measurements recorded by the meter to manual measurements. Depth and velocity adjustments for the ADS Triton Plus are made directly to the meter upon initial installation, and field confirmations are performed throughout the flow monitoring period.

The meter housing was secured to the steps or with an eyebolt on the frame of the manhole. A stainless steel, expandable band secured the depth/velocity probe to the channel. The probes were positioned in the flow of the incoming pipe to minimize the effects of flow turbulence and debris buildup that may exist in the manhole.

Telemetry Equipment:

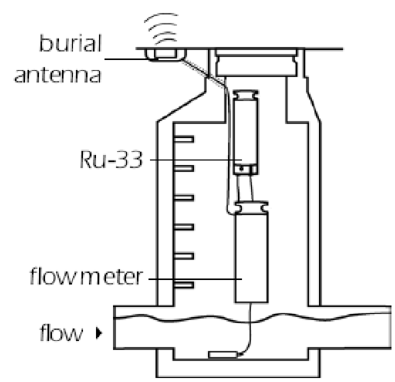


Figure 2.5
Typical Meter Installation

Each Triton Plus flow meter called into RJN's central processing location during the project to perform a data transfer for data collection and review. This process was done through a Remote Transmission Unit (RTU) that collects data and sends the collected data by wireless telemetry to the central processing location. The data was then available for review on the website hosted by RJN.

Each RTU initiated a call into the central location once every 24 hours. The RTU's transmit all of the 5-minute data points recorded that day.

The antennas were typically located just outside the manhole because a cast-iron manhole cover would block the wireless signals otherwise. The antennas were buried just below the existing surface and covered with traffic approved material. A typical installation is shown in Figure 2.5.

Meter Maintenance:

During site visits, data stored in the meter was manually retrieved as a backup to the telemetry download and the meters were inspected to ensure proper operation. The meter depth and velocity readings were taken before and after maintenance/cleaning of the probes. Manual depth and velocity measurements were taken, silt deposit depths were recorded, and the batteries were changed, if necessary. Typically, one internal power supply will provide an average of ten to twelve (10-12) weeks at a 5-minute sample rate for data collection. Offset adjustments were recorded in the site specific meter maintenance log. Six (6) field confirmations were performed on All ten (10) meters.

Table 2.2
FLOW MONITORING LOCATIONS

Meter Site	Manhole Number	Site Address	Pipe Height (in)	Pipe Width (in)
SS-01	1-002	800 Anderson Ave	11.50	11.50
SS-02	2-168	800 Anderson Ave	42.00	46.00
SS-03	3-138	1860 Sycamore Heights Rd	11.50	11.42
SS-04	4-010	313 S Dogwood St	9.75	9.56
SS-05	5-181	401 W University St	17.12	17.12
SS-06	6-001	200 N Maple St	29.75	29.25
SS-07	7-039	302 E Jefferson St	11.75	11.62
SS-08	8-002	905 E University St	11.75	11.75
SS-09	3-179	106 Greenwood	31.00	31.00
SS-10	9-012	1231 N Lincoln St	23.50	23.50

Rainfall Monitoring

Five (5) temporary rain gauges maintained by RJN Group, Inc. were utilized to monitor rainfall during the flow monitoring period. Exhibit 1 shows the location of the temporary rain gauges. Appendix B shows the rain gauge site reports. Rainfall was recorded with a continuously recording rain gauge with an accuracy of 0.01 inches. The rain gauge was equipped with a separate RTU and called in once per day to relay data to RJN's central database.

Rainfall Results:

Frequent rain events were observed during the flow monitoring period. All five (5) rain gauges recorded at least one rain event exceeding a 1-inch rainfall over a 24-hour period. Rain gauge recorded totals and intensities are listed in Table 2.3 on page 2-7. Tabular electronic data is provided with this report.

Table 2.3
RAINFALL SUMMARY

	Rain Gauge No. 1 Pollution Control Plant 975 Anderson Ave		Rain Gauge No. 2 Phillips 66 1295 N Mt Olive St		Rain Gauge No. 3 City Yard 1030 E Ashley St		Rain Gauge No. 4 Waffle House 2723 US-412		Rain Gauge No. 5 Water Treatment Plant 2600 S Carl St	
Date	**Total Daily Rainfall (in.)**	**Peak 60-Minute Rainfall Intensity (in/hr)**	**Total Daily Rainfall (in.)**	**Peak 60-Minute Rainfall Intensity (in/hr)**	**Total Daily Rainfall (in.)**	**Peak 60-Minute Rainfall Intensity (in/hr)**	**Total Daily Rainfall (in.)**	**Peak 60-Minute Rainfall Intensity (in/hr)**	**Total Daily Rainfall (in.)**	**Peak 60-Minute Rainfall Intensity (in/hr)**
2018										
6-Apr	0.05	0.05	*1/*	*1/*	0.07	0.06	0.05	0.04	0.05	0.03
7-Apr	0.01	0.01	*1/*	*1/*	*1/*	*1/*	*1/*	*1/*	*1/*	*1/*
13-Apr	0.59	0.57	*1/*	*1/*	0.93	0.82	1.21	0.99	0.99	0.90
21-Apr	0.70	0.17	*1/*	*1/*	0.73	0.18	0.81	0.20	0.82	0.21
22-Apr	0.60	0.26	*1/*	*1/*	0.61	0.25	0.51	0.17	0.89	0.44
25-Apr	0.45	0.12	*1/*	*1/*	0.47	0.16	0.48	0.18	0.74	0.33
2-May	0.43	0.41	*1/*	*1/*	0.74	0.67	1.02	0.93	0.99	0.92
3-May	2.48	0.72	*1/*	*1/*	2.79	1.01	2.50	0.85	3.50	1.41
4-May	0.07	0.04	*1/*	*1/*	0.07	0.07	0.05	0.05	0.12	0.12
14-May	*1/*	*1/*	1.02	0.97	*1/*	*1/*	0.10	0.10	*1/*	*1/*
15-May	0.10	0.06	0.40	0.40	0.12	0.08	0.13	0.07	0.14	0.09
16-May	0.29	0.12	*1/*	*1/*	0.48	0.22	0.41	0.24	0.29	0.15
20-May	0.19	0.17	*1/*	*1/*	0.06	0.03	0.05	0.03	0.21	0.17
23-May	0.56	0.29	0.44	0.43	1.10	0.87	1.55	1.27	0.69	0.28
24-May	0.62	0.62	0.47	0.47	0.35	0.35	0.37	0.36	1.02	1.02
30-May	0.22	0.19	0.23	0.19	0.23	0.20	0.21	0.18	0.30	0.25
31-May	0.05	0.05	0.04	0.03	0.02	0.02	0.03	0.03	0.03	0.02
1-Jun	*1/*	*1/*	*1/*	*1/*	*1/*	*1/*	0.02	0.02	*1/*	*1/*
2-Jun	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.02	0.02	0.02
12-Jun	0.63	0.36	0.74	0.44	0.71	0.45	0.66	0.41	0.84	0.49
20-Jun	0.02	0.01	0.05	0.05	0.06	0.03	0.03	0.01	0.08	0.05
23-Jun	0.01	0.01	0.01	0.01	*0.01*	*0.01*	0.01	0.01	0.03	0.02
24-Jun	0.24	0.11	0.22	0.10	*0.22*	*0.09*	0.21	0.09	0.27	0.13
26-Jun	0.39	0.38	0.53	0.52	*0.46*	*0.44*	0.51	0.49	0.57	0.57
1-Jul	0.25	0.21	0.26	0.20	0.35	0.31	0.26	0.21	0.37	0.32
Total	**8.96**		**4.42**		**10.59**		**11.20**		**12.96**	

1/ No rainfall recorded

CHAPTER 3 – LIFT STATION EVALUATION

As part of Phase I services, RJN performed city-wide lift station evaluations during winter of 2017. Lift station evaluations through drawdown testing are performed to compare actual pumping rates to original pump curves at the lift station. The evaluations will improve accuracy when constructing the hydraulic model and recommendations to upgrade, expand, or eliminate lift stations within the collection system. An overall area map including the locations and names of the lift stations is shown in Figure 3.1 on page 3-2.

A total of 18 lift stations exist in the City of Siloam Springs wastewater collection system. A total 14 lift stations were visited on November 3, November 9, and November 30, 2017. Full drawdown testing was performed on 9 lift stations.

Lift Station Configurations

RJN was accompanied by a City of Siloam Springs during lift station evaluations to allow access to the lift station, operate pumps during drawdown testing, and troubleshoot pumps during testing. Evaluations of the lift stations identified a wide range lift station configuration. All lift stations had two pumps operating parallel. A summary of lift station configurations can be found in Table 3.1.

Drawdown Testing and Pumping Capacity Analysis

Drawdown testing is a method of calculating pumping capacity for a pump by means of measuring the rate at which the pumps move flow out of the wet well. The following procedure was followed during testing:

1. Determine the cross-sectional area of the wet well.
2. Draw down the wet well until inverts of all pipes are exposed. This is to ensure that fill rates can be properly accounted for.
3. Insert pressure transducer into wet well to determine water depth.
4. Begin drawdown by powering on one pump. Simultaneously, start a stopwatch. Pumping can be recorded either on a time basis (e.g. 1 minute pump run time and record depth difference) or depth basis (e.g. draw down 1 foot and record time). This is the first draw down test.
5. Turn the pump off while simultaneously “lapping” the stopwatch. Record the depth at the lapped time. This begins the fill process of the wet well. Allow wet well to fill and record on time or depth bases as discussed above.
6. Repeat the draw down and fill process for verification.

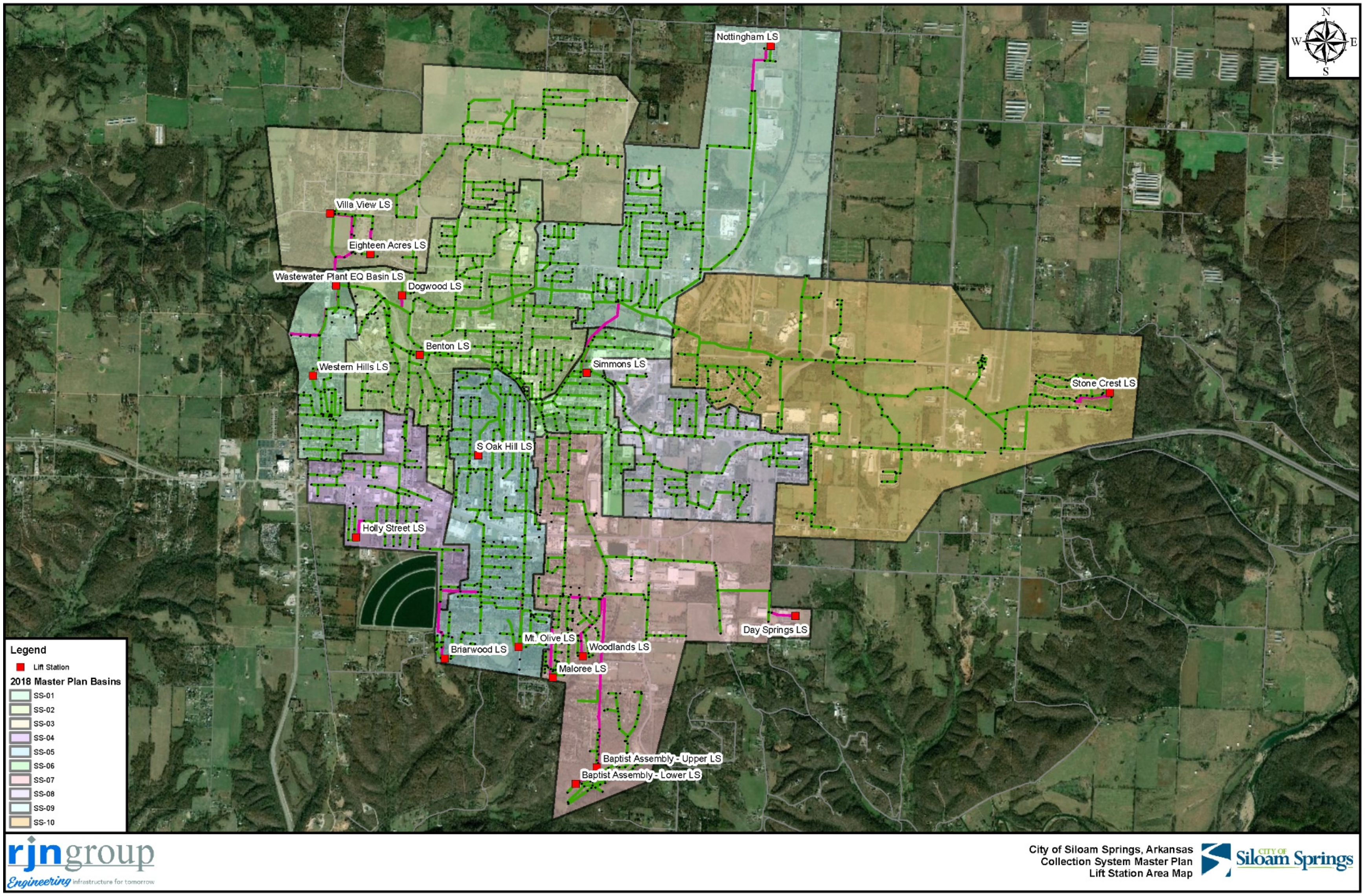


Figure 3.1
Siloam Springs Sewer System and Lift Stations

Table 3.1
Lift Station Configuration Summary

Lift Station Name	Pump Manufacturer	Pump Model	Install Year	HP	Force Main Diameter (in.)	Wet Well Dimensions	
						Diameter (ft)	Depth (ft)
Benton Lift Station	Hydromatic	2" Duplex Non-clog	1992	2	1/	4.0	9.0
Simmons Lift Station	Hydromatic	8" Solids Handling	1999	25	12.0	14.0	21.1
Day Spring Lift Station	Myers	4" Duplex Non-clog	1998	3	1/	6.0	12.7
Baptist Assembly - Upper Lift Station	Hydromatic	4" Duplex Chopper/Solids Handling	2018	15	10.0	6.0	10.0
Baptist Assembly - Lower Lift Station	Hydromatic	4" Duplex Chopper/Solids Handling	2018	15	10.0	6.0	16.3
Maloree Lift Station	1/	1/	1/	1/	6.0	8.0	12.2
S Mt. Olive Lift Station	Myers	4" Duplex Non-clog	1998	5	6.0	8.0	13.5
Briarwood Lift Station	Hydromatic	4" Duplex Non-clog	1993	15	6.0	6 x 6	16.8
Woodlands Lift Station	Flygt	MF 3102 LT	2005	11	6.0	10.0	14.0
Western Hills Lift Station	Hydromatic	4" Duplex Non-clog	2005	8	6.0	8.0	22.5
Dogwood Lift Station	Hydromatic	2" Duplex Grinder	2005	2	2.0	6.0	11.3
Eighteen Acres Lift Station	Hydromatic	2" Duplex Grinder	2005	2	2.0	5.0	11.5
Villa View Lift Station	Hydromatic	4" Duplex Non-clog	2005	10	6.0	8.0	24.0
Wastewater Plant EQ Basin Lift Station	1/	1/	1/	1/	1/	2/	2/
Nottingham Lift Station	Flygt	MF 3102 LT	2006	6	6.0	8.0	N/A
S Holly Street Lift Station	Hydromatic	1/	2006	2	4.0	6.0	16.2
Stone Crest Lift Station	1/	1/	1/	5	6.0	5.0	13.9
S Oak Hill Lift Station	1/	1/	1/	1/	1/	2/	2/

1/ Lift station information not provided
2/ Lift station not visited

Table 3.2
Lift Station Pumping Capacity

Lift Station Name	Force Main Diameter	Total Dynamic Head	Pump 1 Flow Rate (GPM)	Pump 1 Rated Capacity (GPM)	% Operating Flow Pump 1	Pump 2 Flow Rate (GPM)	Pump 2 Rated Capacity (GPM)	% Operating Flow Pump 2
Maloree Lift Station	6	*1/*	96.79	*1/*	*3/*	*4/*	*1/*	*3/*
S Mt. Olive Lift Station	6	30	61.04	140	44%	61.42	140	44%
Briarwood Lift Station	6	71	*4/*	210	*3/*	251.65	210	120%
Western Hills Lift Station	6	41	203.29	410	50%	158.35	410	39%
Dogwood Lift Station	2	39	46.67	42	111%	*4/*	42	*3/*
Eighteen Acres Lift Station	2	80	41.84	42	100%	39.99	42	95%
Villa View Lift Station	6	56	237.08	400	59%	144.42	400	36%
S Holly Street Lift Station	6	*1/*	71.06	*1/*	*3/*	69.26	*1/*	*3/*
Stone Crest Lift Station	6	*1/*	52.92	*1/*	*3/*	*4/*	*1/*	*3/*

1/ Lift station information not provided
2/ Lift station not visited
3/ Unable to calculate
4/ Pump inoperable

Lift Stations Not Visited

A total of 4 out of 18 lift stations were not visited during lift station evaluations. The Benton lift station connects 2 residences and was determined to be too small for evaluation. The Day Springs lift station servicing Day Spring Cards, Inc. is small station and has a relatively low effect on the main interceptor for hydraulic modeling flow and was not evaluated. The wastewater plant EQ basin lift station is continually monitored at the treatment plant and was not evaluated. S Oak Hill is a small, two manhole pump configuration designed to connect a few residences to the gravity main and was not evaluated due to the small size.

Lift Stations Unable to Perform Drawdown Testing

Simmons Lift Station

Simmons lift station was visited on November 3, 2017 and services Simmons Foods. Significant debris was observed in the wet well of the lift station, the pumps were not visible through the blanket of debris.

Simmons Lift Station Area

Simmons Lift Station Wet Well

Camp Siloam/Baptist Assembly Lift Station

There are two lift stations located at Camp Siloam denoted as Baptist Assembly – Upper and Baptist Assembly – Lower. Both lift stations were visited and are scheduled for upgrades, with construction starting in 2018. Due to future upgrades, evaluation was not performed at either the upper or lower lift station.

For upgrades, it is recommended that the lower assembly contain two grinder pumps with the purpose of breaking down debris before transferring to upper lift station. It is recommended the upper contain two non-clog or equivalent pumps to transfer flow out of the lift station. After construction, it is recommended to verify pumps are operating at designed pumping capacity.

Baptist Assembly - Lower

Baptist Assembly - Upper

Woodlands Lift Station

Woodlands lift station was installed in 2005 and has two pumps rated for approximately 115 GPM. It was noted that the wet well diameter is 10 feet and services one small neighborhood. The small pump size and large wet well diameter indicates the wet well was oversized, which can affect efficient operation of the pump.

Woodlands Area

Woodlands Wet Well

Nottingham Lift Station

Nottingham lift station has an 8 foot diameter wet well installed in 2006. It services a small neighborhood comprised of duplexes on the northeast side of Siloam Springs. Due to the small size and low effect on the hydraulic model, draw down testing was not performed.

Nottingham Area

Nottingham Wet Well

Lift Stations Drawdown Testing

Maloree Lift Station

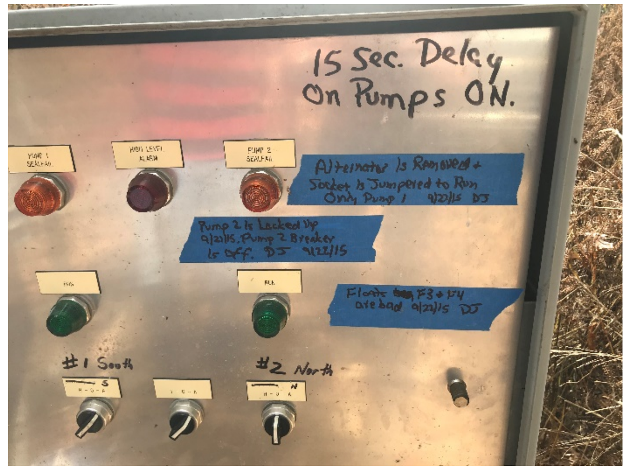
Maloree Area

Maloree Wet Well

The Maloree lift station is 8 feet in diameter with a 6-inch force main. Access to the lift station is difficult due to a steep, rocky drive off Courtney Circle. The lift station does not have a security fence around it. Significant debris was observed in the wet well, including large branches and a dog toy. The lift station has an outdated 3-phase pump coming from a 1-phase power with a converter.

The lift station was noted as surcharged during evaluation. Pump No. 2 was not operating during time of visit in November 2017. Pump 1 was powered on to expose the incoming pipe from the northeast. A metal plate was installed in the wet well directly in front of the incoming northeast pipe. Drawdown testing for pump 1 revealed an average measured flow rate of 97 GPM.

Maloree Plate in Wet Well



Maloree Control Panel

S Mt. Olive Lift Station

Mt. Olive lift station was installed in 1998 and is an 8 foot diameter wet well approximately 12 feet 8 inches in depth. It contains two Myers Duplex Non-clog pumps each rated at 140 GPM pumping capacity. During drawdown testing, Pump 1 exhibited swirling of the localized water around the pump indicating an improper seat of the pump seal. Additionally, the lift station exhibited surcharging and both pumps were powered on to expose the incoming pipes for drawdown testing.

Drawdown testing was performed on both pumps at the Mt. Olive lift station. Pump 1 has a calculated flow rate of 61 GPM. Pump 2 has a calculated flow rate of 61 GPM. The average pumping capacity of both pumps is 44% of the rated pumping capacity of the lift station.

The lift station displayed evidence of surcharge in the wet well. The force main discharges into manhole 5-1803 and surcharge debris was observed around the cover of the manhole. RJN opened the cover of 5-1803 and noted additional surcharge evidence in the manhole.

Mt. Olive Area

Mt. Olive Wet Well

Briarwood Lift Station

Briarwood lift station was installed in 1993 and consists of a square wet well 6 foot by 6 foot with a depth of approximately 16 feet 2 inches. The rated pumping capacity of the Hydromatic Duplex Non-Clog pumps is 210 GPM.

Drawings and schematic of the Briarwood lift station were provided. The float level controls were noted not to match the provided drawing elevations. The float levels were above the crown of the incoming pipe. It is recommended to lower the float controls to create free flowing conditions.

Pump 1 was not in operation during RJN evaluation. The measured flow rate from drawdown testing for pump 2 averaged at 252 GPM. This is above the rated pumping capacity for the station.

It is recommended to service pump 1 and verify through drawdown testing it is operating at design flow pumping capacity.

Briarwood Area

Briarwood Wet Well

Western Hills Lift Station

Western Hills lift station was installed in 2005 and is an 8 foot diameter wet well approximately 23.5 feet deep. The lift station is rated at a total pumping capacity of 410 GPM.

RJN performed a total of four (4) drawdown tests for each pump at Western Hills. Two (2) drawdown test on each pump were performed on November 3, 2017. The average flor rate was 226 GPM for pump 1, and 134 GPM for pump 2. The measured flow rates were 55% and 33%, respectively, of the total rated pumping capacity for the lift station.

During drawdown testing, the air release valve was observed to vent erratically. It is recommended to service the air release valve to ensure a tight seal and operating properly.

Verification of the drawdown tests were performed on November 30, 2017 to confirm that both pumps were operating below the rated capacity. The flow rates for the verification tests were 181

GPM for pump 1 and 183 for pump 2. Additionally, RJN measure pumping flow rates for both pumps powered on. The averaged measured flow rate for both pumps on was 439 GPM.

Western Hills Area

Western Hills Wet Well

Dogwood Lift Station

The Dogwood lift station was installed in 2005 and is a 6 foot diameter wet well that is 11.8 feet in depth. The Hydromatic duplex grinders are rated for a total pumping capacity of 42 GPM. The incoming flow to the Dogwood lift station was determined negligible at the time of drawdown testing.

The average flow rate for pump 1 was determined to be 47 GPM. Pump 1 was determined to be within normal operating limits of the lift station. Pump 2 was not in operation during the time of drawdown testing.

Dogwood Area

Dogwood Wet Well

Eighteen Acres Lift Station

Eighteen Acres lift station was installed in 2005 and consists of a 5 foot diameter wet well and 12 feet in depth. Upon visit of site, the valve vault was observed to be flooded indicating a defective or closed valve vault drain valve in the lift station wet well. The total rated pumping capacity of the lift station is 42 GPM.

Drawdown testing the measured flow rates during draw down testing were 42 for pump 1 and 40 for pump 2. Both pumps were observed to be within normal operating limits of the lift station.

Eighteen Acres Area

Eighteen Acres Wet Well

Eighteen Acres Valve Vault

Villa View Lift Station

Villa view lift station was installed in 2005 and consists of an 8 foot diameter wet well approximately 25 feet deep. Villa View lift station is rated for pumping rate of 400 GPM. The first visit to the lift station for evaluation was November 9, 2017. A drawdown of approximately 3.5 feet was performed to expose the invert of incoming pipe before drawdown testing of the pumps could begin.

The drawdown testing results for the initial Villa View testing revealed pump 1 operating at 107 GPM and pump 2 operating at 102 GPM. The measured operating flow rates are approximately 26% of the rated flow rate of the lift station. Due to the low operating flow rates, RJN performed a second visit of drawdown testing to confirm the measured results.

The second visit to Villa View lift station occurred on November 30, 2017. During the visit, City of Siloam Springs crew noted that the lift station and pumps had been serviced since between the dates of RJN drawdown testing. Second round of drawdown testing revealed measured flow rates of 302 GPM for Pump 1 and 165 GPM for pump 2.

Villa View Area

Villa View Wet Well

S Holly Street Lift Station

Holly St was installed in 2006 with a wet well 6 feet in diameter and 16 feet in depth. No client information was provided for the S Holly lift station. Drawdown testing was performed on November 3, 2017. Measured flow rates for pump 1 averaged 71 GPM and pump 2 averaged 69 GPM.

Holly Street Area

Holly Street Wet Well

Stone Crest Lift Station

Stone crest lift station consists of a 5 foot diameter wet well approximately 15 feet deep. No client information was provided on the Stone Crest lift station. The security fencing of the lift station was noted to be in disrepair. Pump 2 was not operating during the time of visit.

Drawdown testing resulted in an averaged measured flow rate of 53 GPM for pump 1.

Stone Crest Area

Stone Crest Wet Well

Lift Stations Recommendations

Table 3.3 below gives a summary of general recommendations from results of lift station evaluations. These recommendations do not include capacity improvement recommendations that are discussed in Chapter 6 – Hydraulic Capacity Analysis.

Table 3.3
General Lift Station Recommendations

Lift Station	Recommendations
Benton Lift Station	Recommend verifying pumping capacity and general maintenance and cleaning.
Simmons Lift Station	General maintenance and cleaning.
Day Spring Lift Station	Install security fencing.
Camp Siloam/Baptist Assembly – Upper	Complete upgrades and verify design pumping capacity.
Camp Siloam/Baptist Assembly – Lower	Complete upgrades and verify design pumping capacity.
Maloree Lift Station	Install security fencing. Convert power phase. Service Pump No. 2. Verify pumping capacity.
S Mt. Olive Lift Station	Replace both pumps/service pumps for designed flow. Reduce surcharge in discharge manhole.
Briarwood Lift Station	Service Pump No. 1 and verify pumping capacity. Recommend adjust floats to ensure incoming pipe is under free flow conditions.
Woodlands Lift Station	Recommend odor control and general maintenance. Lift station pumps are potentially undersized.
Western Hills Lift Station	Replace both pumps. Service air release valve.
Dogwood Lift Station	Service Pump No. 2 and verify pumping capacity.
Eighteen Acres Lift Station	Service drain valve in air release valve vault.
Villa View Lift Station	Replace both pumps.
Wastewater Plant EQ Basin Lift Station	No recommendations.
Nottingham Lift Station	General maintenance and cleaning.
S Holly Street Lift Station	No recommendations
Stone Crest Lift Station	Repair security fencing. Service Pump No. 2
S Oak Hill Lift Station	No recommendations.

CHAPTER 4 – MODEL DEVELOPMENT

Model Description

As part of the wastewater collection system master plan, a sanitary sewer hydraulic model was developed to help assess the current state of the collection system and forecast how it will work with future flows.

The main objectives of the hydraulic model build were to:

- Create a complete sanitary sewer hydraulic model, including pressure systems
- Survey all 10-inch diameter and larger sanitary sewer structures and create a digital surface
- Calibrate the model to reflect recorded flows and surcharge depth data during metering
- Create future growth model reflecting ultimate development projections
- Simulate a design storm on the existing and ultimate development models to identify areas with insufficient capacity and potential overflows
- Develop a staged system implementation strategy to eliminate projected wet-weather sanitary sewer overflows (SSO's) with a minimum design criterion of 3-feet below top of sanitary sewer structures
- Evaluate alternatives to optimize system performance

Model Build

The City of Siloam Springs provided GIS data of their current collection system. Additionally, many record drawings and historical data was provided. The hydraulic model was built incorporating provided record drawings and provided GIS data. Any updates to the connectivity or flow network found during field investigations were incorporated into the model.

It was observed that residential and industrial development had spread beyond the original 1994 sewer basin boundaries, and it was determined that an update to the City's basin boundaries was necessary for 2018 Collection System Master Plan. The sewer flow network was analyzed, and new basins were developed prior to Flow Monitoring beginning. A comparison of previous and current basins can be found in Figure 4.1 on page 4-2.

Survey

RJN identified 10 inch and larger sewer segments and collaborated with the City to determine the critical sewer interceptors for hydraulic capacity analysis. Rim elevation GPS points for each manhole located on the interceptor were provided by the City.

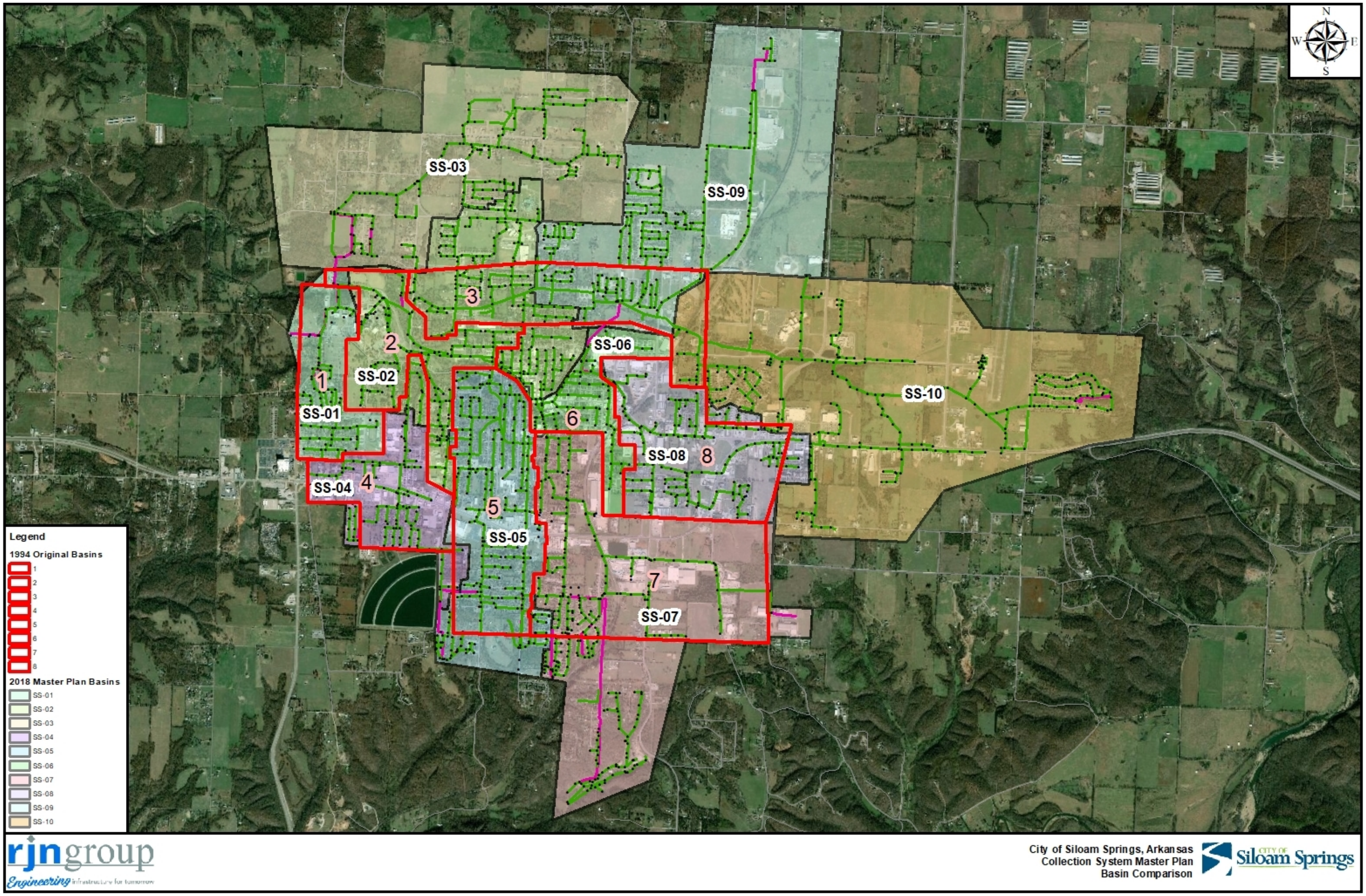


Figure 4.1
Collection System Basin Comparison

Measure downs were performed by RJN on each manhole located on the critical interceptor to gather invert elevations for sewer lines for input into the hydraulic model. Additionally, routing and connectivity between sewer lines was verified during measure down inspections. A total of two hundred ninety-seven (297) manholes were surveyed and contain measure down data. The remaining manhole rim elevations were modeled based on record drawings and contour data.

Invert elevations were then calculated to maintain continuity and slopes based on engineering judgement. The model was then checked for proper slopes and connectivity using validation tools built into the model. Sewer lines found with negative or questionable slopes were manually adjusted based on field verifications or engineer's judgement. Where possible, more accurate information such as record drawings or survey data was used in preference to model interpolated data.

The hydraulic model contains approximately 526,764 linear feet of gravity main, and 29,835 linear feet of force mains. A summary of the collection system incorporated into the model can be found in Table 4.1 below.

Table 4.1
COLLECTION SYSTEM SUMMARY

Gravity Mains		Force Mains	
Pipe Diameter (in)	**Length (ft)**	**Pipe Diameter (in)**	**Length (ft)**
6	127,231	2	1,284
8	298,552	4	3,663
10	24,904	6	10,905
12	31,835	8	3,368
14	2,417	10	7,739
15	6,586	12	21
18	6,121	14	2,855
22	2,611		
24	9,192		
26	616		
27	4,831		
30	9,998		
31	224		
48	1,763		

Lift Stations

Incorporation of the lift stations and lift station evaluation data into the hydraulic model is critical to determine flow regime for wet weather calibration. A total of 18 lift stations exist in the City's collection system. Lift station evaluations are discussed in detail in Chapter 3. Pumping capacity data from lift station evaluations were incorporated into the hydraulic model.

Hydraulic Model Termination

The hydraulic model terminates at the inlet of the influent lift station at the wastewater treatment plant. The outfall conditions at the inlet of the wastewater treatment plant has been represented as a free outfall, or unlimited capacity.

Sub-Catchments

Sub-catchments are geometric sub-areas within each drainage basin. These modeled areas contain all the parameters for loading flow into the sewer network including population, non-residential flow and rainfall derived inflow/infiltration (RDII) runoff parameters. Sub-catchments were imported into the model from existing parcel data from City GIS. By loading flow from individual parcels, more detailed analysis can be performed. This also eliminates the need for time of concentration assumptions.

The sub-catchments encompasses all areas that contribute flow into the sewer system. Areas such as parks, golf courses, cemeteries, etc. that are not connected to the sewer system were not imported from the parcel data as they do not contribute any wastewater flow or RDII.

Existing Population Processing

Households were located within the sanitary sewer system using the residential customer billing data (points) from the City. Populations were assigned to individual households (customer billing points) within each parcel. The populations from the customer billing points were then totaled within each modeled sub-catchment. The total current population in the City of Siloam Springs during the flow monitoring period is estimated at 16,991 and the total population for ultimate developed area is projected to be 26,846.

Commercial/Industrial Loading

Commercial, institutional, and industrial flows were loaded into the model using the customer water billing data provided by the City. The commercial, institutional, and industrial flows were calculated applying a 90% return to sewer factor to the water usage data.

Model Testing

A series of validation tests were undertaken on the model to confirm logical network connectivity as well as consistent vertical alignment. A standard residential hydrograph was applied to each sub-catchment in the model and validation simulation undertaken. After a few minor alterations the model was able to run a 24-hour dry-weather simulation without any issues and was considered to be a stable platform for calibration to proceed.

Model Calibration

Model calibration is necessary for the model to accurately represent the behavior of the sanitary sewer collection system. Model calibration is a process through which model variables and coefficients are adjusted through multiple iterations until flow, depth, and velocity matches within a reasonable accuracy to the actual flow meter data recorded during events. The model is calibrated to recreate sewer performance in both dry-weather and wet-weather conditions.

The model does not use the flow survey data as an input to generate flow in the hydraulic model. Flow survey data was utilized to calibrate points across the system to check modeled flows against the recorded flows. The calibration is considered successful when the total volume at all meter sites matches recorded data within 10% percent.

Dry-Weather Period

Dry-weather calibration ideally requires at least a 7-day period, including one weekend, unaffected by rainfall induced flows. The recorded flow data was assessed in conjunction with the rainfall data and the period from June 4, 2018 through June 11, 2018 was selected as a representative dry-weather period.

Using flow data collected from each meter, weekdays and weekend dimensionless diurnal profiles were developed through a process of groundwater subtraction and normalization. From this process, three unique residential flow profile were created. The profiles were input into the model and used to modulate dry-weather flows in the model. Figure 4.2 on page 4-6 graphically depicts the three residential flow profile used.

Commercial and Industrial areas produce flow pattern that are unique from residential areas. A set of standard profile were used for commercial and industrial areas. The profiles were assigned based on the predominant type of business or industry in each sub-catchment as determined during model construction. Figure 4.3 on page 4-6 graphically depicts both commercial and industrial flow profiles used for this project.

Calibrating the model for dry-weather flow was achieved by modifying:

- Permanent groundwater infiltration rates
- Per capita flow rates
- Commercial/Industrial flow rates

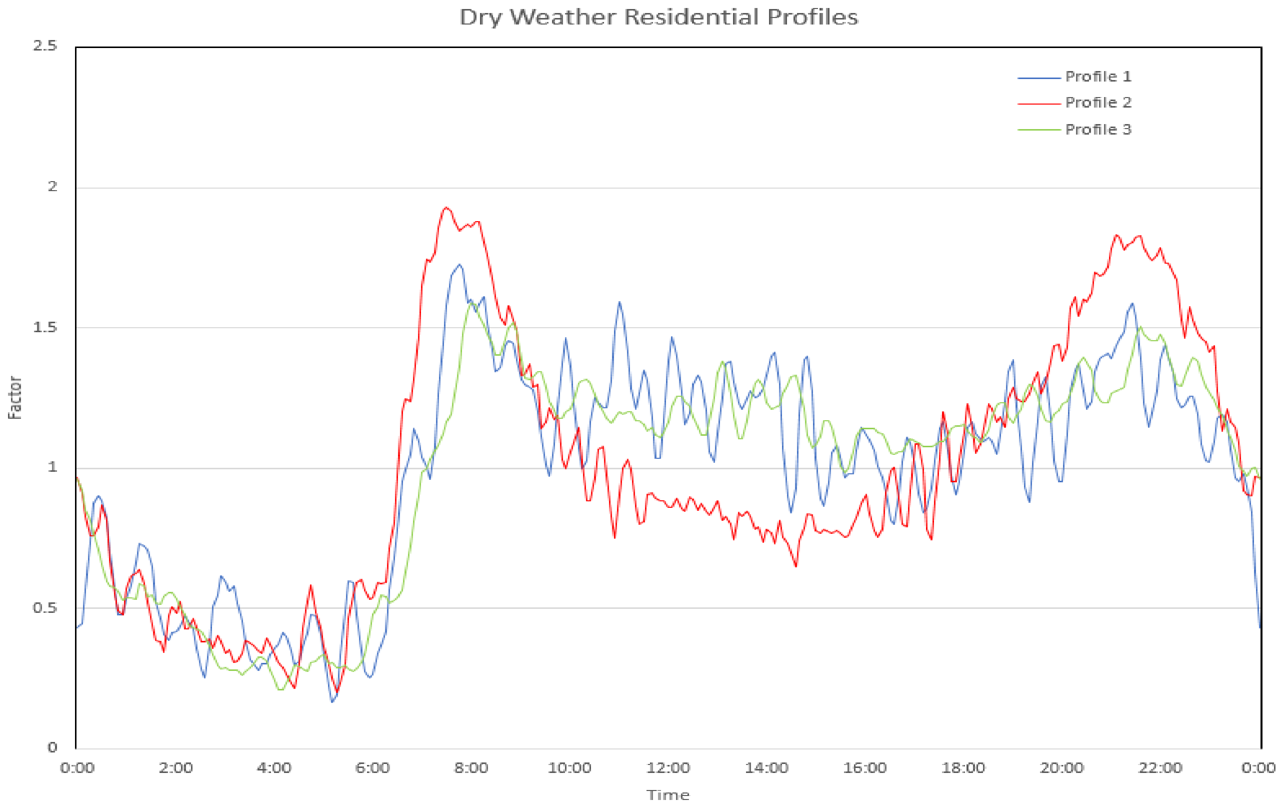


Figure 4.2: Residential diurnal flow profiles (water usage records & estimated flow)

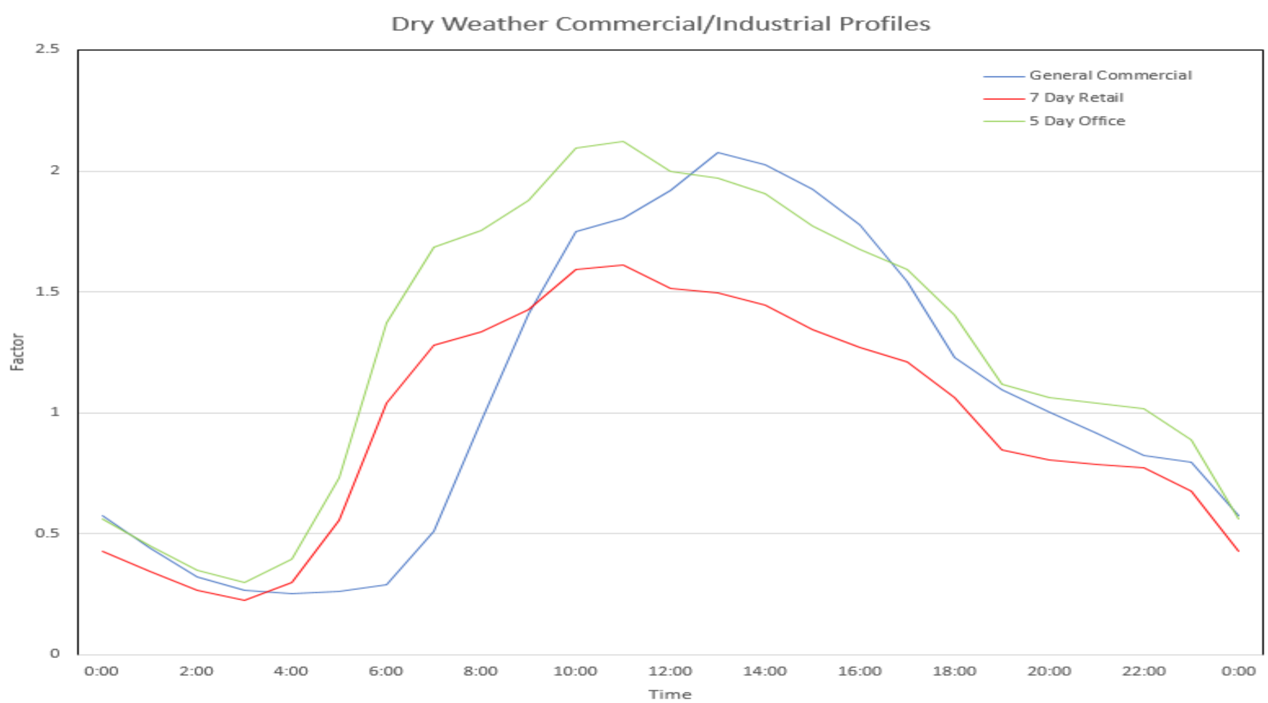


Figure 4.3: Non-residential diurnal flow profiles

Wet-Weather Period

Wet-weather flows were generated in the model using both fixed response surfaces as well as infiltration flows:

- Up to three "fixed" response surface areas were calibrated for each subcatchment. These surface types are fundamentally independent of the catchment condition prior to the rainfall event and represent fast responses from areas such as illegally connected roof drainage and storm water cross connections.

- Infiltration was modeled using hydrology in the Ground Infiltration module within Infoworks. This hydrological module has soil and groundwater storage zones and the inflow into the model is dependent upon the wetness of the catchment prior to the rainfall event. These flows represent the delayed ingress of storm water from the ground into the sewer system through cracks and leaks in sewers and private drains.

Review of the wet-weather response to rainfall indicated that the level of inflow (fast response) was in line with what would be expected in a sanitary sewer system. The wet weather calibration focused primarily on matching the response during storm events. Several iterations were run to get the model to match recorded data. Figure 4.4 shown below is a representative sample of calibration data.

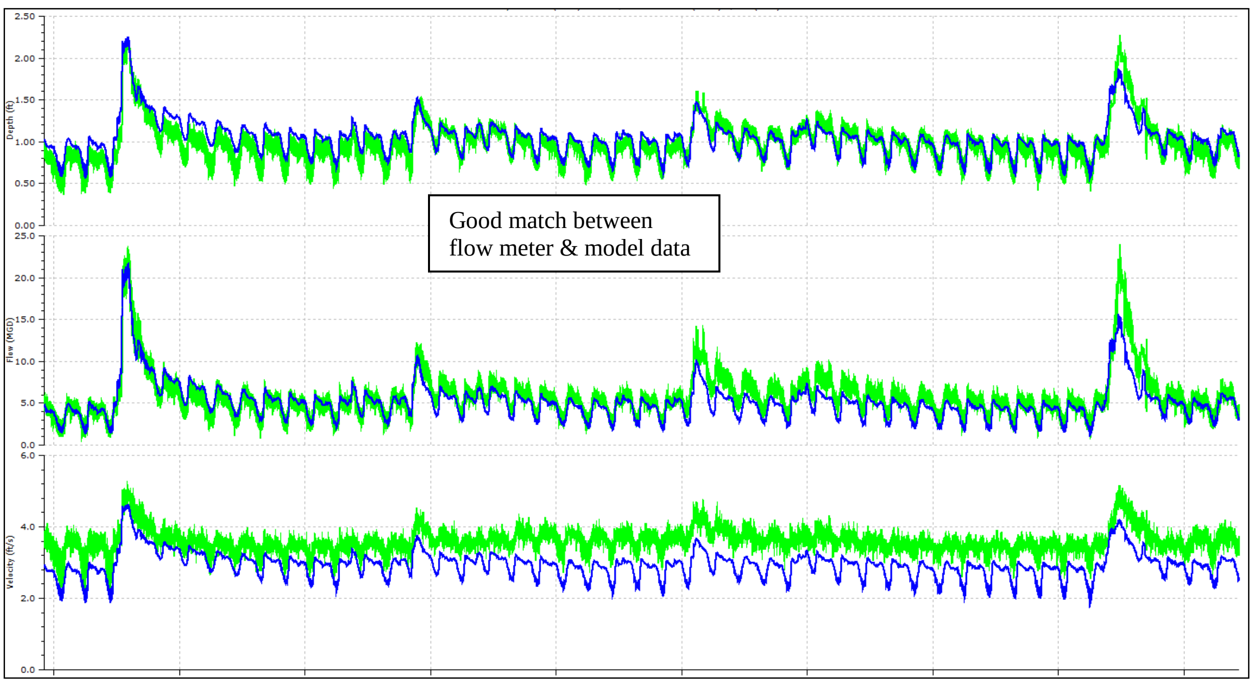


Figure 4.4: Flow Meter Response v Model Response

CHAPTER 5 – DEVELOPMENT OF FUTURE FLOWS

The City of Siloam Springs is projected to have significant growth within the current city limits. Maps were provided by Siloam Springs showing proposed zoning. These maps were analyzed to determine future growth projections.

Existing Dry-Weather Flow

In order to facilitate model flow loading and calibration, existing population and flows were determined and analyzed.

Existing Residential Population

Initial population data was obtained from the U.S. Census Bureau for the City of Siloam Springs. Using this data, population was assigned to residential households using the residential customer billing points provided by the City. The total estimated population in the City of Siloam Springs model is 16,991. A summary of the population by meter-basin is shown in Table 5.1. The estimates in the table below are based on Spring 2017 sewer customers.

Table 5.1

EXISTING RESIDENTIAL POPULATION BY METER BASIN

Meter Basin	Total Population	Average Population Per Household/Parcel
SS-01	900	3.15
SS-02	3,103	3.20
SS-03	666	3.10
SS-04	1,033	3.84
SS-05	2,533	2.72
SS-06	970	3.45
SS-07	1,552	2.81
SS-08	1,554	4.16
SS-09	2,886	3.68
SS-10	1,794	4.23
Total	**16,991**	

Existing Residential Flow

Using the flow data collected from each meter during a dry period, average weekday and weekend hydrographs were calculated and graphed for each flow meter with a primarily residential catchment. Weekday and weekend diurnal profiles of usage were developed by removing permanent groundwater infiltration. Profiles were then grouped based on profile shape and averaged across basins to determine three representative units-less residential patterns. These profiles were input into the model and used to modulate dry-weather flows in the hydraulic model. The weekday profiles used in the model are shown in Chapter 4: Model Development, Figure 4.2.

Commercial and Industrial Flow

Commercial, institutional, and industrial flows were modeled using the customer water billing data provided by the City. The commercial and industrial sewage flows were calculated based on factored water usage for each customer. The City provided trade waste details for the institutional and industrial customers who discharge a significant volume to the sewer. These users were located, and their discharge explicitly modeled.

A summary of existing estimated commercial/institutional/industrial flows is shown in Table 5.2.

Table 5.2
COMMERCIAL AND INDUSTRIAL WASTEWATER FLOW

Meter Basins	Flow (MGD)
SS-01	0.0073
SS-02	0.1030
SS-03	0.0001
SS-04	0.0424
SS-05	0.0496
SS-06	0.3232
SS-07	0.0465
SS-08	0.1441
SS-09	0.1512
SS-10	0.0617
Total	**0.9291**

Summary of Existing Flows

A summary of the existing modeled wastewater collection system flows is given below in Table 5.3.

Table 5.3
EXISTING DRY-WEATHER FLOW CONDITIONS

Meter Basin	Pop.	Average Per Capita Flow (GPM)	Residential Flow (MGD)	Commercial/ Industrial Flow (MGD)	Permanent Infiltration (MGD)	Meter Basin ADWF (MGD)	Cumulative ADWF (MGD)
SS-01	900	60	0.046	0.007	0.000	0.054	0.054
SS-02	3,103	50	0.133	0.103	0.000	0.236	1.427
SS-03	666	50	0.029	0.000	0.000	0.029	0.029
SS-04	1,033	50	0.044	0.042	0.000	0.087	0.087
SS-05	2,533	50	0.109	0.050	0.045	0.204	0.204
SS-06	970	60	0.050	0.323	0.000	0.373	0.724
SS-07	1,552	50	0.067	0.047	0.000	0.113	0.113
SS-08	1,554	50	0.067	0.144	0.026	0.237	0.237
SS-09	2,886	50	0.124	0.151	0.000	0.275	0.414
SS-10	1,794	50	0.077	0.062	0.000	0.139	0.139
Total	**16,991**						

Ultimate Build-Out

The City provided GIS data of their current collection system. Additionally, many record drawings and historical data was provided. The hydraulic model was built incorporating provided record drawings and provided GIS data. Any updates to the connectivity or flow network found during field investigations were incorporated into the model.

It was observed that residential and industrial development had spread beyond the original 1994 sewer basin boundaries, and it was determined that an update to the City's basin boundaries was necessary for Collection System Master Plan. The sewer flow network was analyzed, and new basins were developed prior to Flow Monitoring beginning. A comparison of previous and current basins can be found in Figure 4.1 on page 4-2.

Proposed Development and Future Zoning

To meet the objectives of the Master Plan Update, the hydraulic model was simulated to include future ultimate growth projections.

Growth was applied utilizing the zoning boundaries within the existing city limits and sewer basins. Generalized development categories for residential, commercial, and industrial users are as follows:

1. Low Intensity
2. Medium Intensity
3. High Intensity
4. Medium Commercial
5. Light Commercial
6. Industrial

It was assumed that no significant sewage producing development will occur on land outside of City-incorporated limits.

CHAPTER 6 – HYDRAULIC CAPACITY ANALYSIS

This Chapter provides a summary of the hydraulic model capacity analysis for the existing system and ultimate build-out scenarios.

Existing System Analysis

The model was used to evaluate the existing system for dry-weather and a wet-weather storm of 1-year/1-hour interval. This allows a system evaluation that stresses the collection system but does not surcharge the sewer lines.

Determination of Average Daily Dry-Weather Flow

Flow data collected during dry weather/low groundwater periods was analyzed to determine the average daily dry weather flow for each basin in the Siloam Springs sanitary sewer system. During analysis, care was taken to select dry weather/low groundwater periods that were representative for each flow meter site.

Residential flows were initially estimated with an assumed daily flow of 80 GPD with the flows modulated over the day using a dimensionless diurnal curve generated from the recorded flow data as illustrated in Figure 4.2 on page 4-6. Per capita flow rates were adjusted, and additional curves were generated as necessary to achieve calibration with the majority of locations assigned per capita flow rates between 40 and 60 GPD.

Average Daily Dry-Weather Flow Peaking Factor

Wastewater flow during dry-weather periods will vary during the day in response to water consumption. By examining the diurnal curves for each monitored drainage basin, a peaking factor was determined. The peaking factor is the ratio of the peak hourly flow rate and the average daily flow. Peaking factors varied from 1.28 to 2.24 in the Siloam Springs collection system area and are given for each basin in Table 6.1.

Table 6.1 AVERAGE DAILY DRY-WEATHER FLOW				
Meter Basin	**Cumulative Average Daily Dry-Weather Flow (MGD)**	**Basin Average Daily Dry-Weather Flow (MGD)**	**Peak Hourly Flow Rate (MGD)**	**Dry Weather Flow Peaking Factor**
SS-01	0.054	0.054	0.120	2.24
SS-02	1.679	0.300	0.504	1.68
SS-03	0.028	0.028	0.048	1.69
SS-04	0.086	0.086	0.120	1.40
SS-05	0.212	0.212	0.288	1.36
SS-06	0.609	0.309	0.480	1.55
SS-07	0.116	0.116	0.216	1.86
SS-08	0.244	0.244	0.312	1.28
SS-09	0.412	0.274	0.360	1.32
SS-10	0.137	0.137	0.192	1.40
	Total	**1.760**	**Average**	**1.58**

Determination of Peak Infiltration

Infiltration may enter the system through pipe joints, sewer line defects (including main sewer lines and building sewer lines), and defective manhole walls, benches, and pipe seals. There are two types of infiltration that can be determined during a study, permanent infiltration and peak infiltration. Permanent infiltration is defined as extraneous flow that enters the sewer system through the ground during periods of dry-weather and low groundwater. Peak infiltration is defined as the maximum extraneous flow that enters the sanitary sewer system during high groundwater conditions after the inflow effects of a rain event have ended. Peak infiltration was used to evaluate the effects of infiltration on the sewer system.

The determination of peak infiltration requires analysis of the flow data obtained during dry-weather/high-groundwater conditions. Care must be exercised in the analysis to exclude days that are too close to rainfall events. This is necessary to avoid including residual inflow (rainfall induced infiltration) that may lead to an over-estimation of peak infiltration. Generally, periods following significant rainfall, excluding the day immediately following a rain event, are used for determining peak infiltration.

Analyses of the flow data following major rain events indicated infiltration entering the wastewater collection system. Systems with infiltration problems typically demonstrate a pattern of elevated flow levels after the rain has passed indicating slow percolation of groundwater into the sewer system through pipe defects. The peak basin unit infiltration rate expressed in gallons per day per 1,000 linear feet (GPD/1,000 LF) is a method of expressing the magnitude of peak infiltration relative to other basins. In general, a unit infiltration rate of 5,000 GPD/1,000 LF is designated as excessive infiltration. Summary infiltration data is shown in Table 6.2 below.

Table 6.2

SUMMARY OF PEAK MONITORED INFILTRATION

Meter Basin	Basin Footage (LF)	Basin Peak Infiltration (MGD)	Basin Peak Unit Infiltration (GPD/1000 LF)	Ranking
SS-01	21,737	0.061	2,787	2
SS-02	96,578	0.156	1,615	4
SS-03[1/]	37,871	0.013	343	9
SS-04	28,444	0.039	1,371	5
SS-05	68,992	0.026	382	8
SS-06	30,181	0.009	299	10
SS-07	62,904	0.025	401	7
SS-08	41,763	0.235	5,636	1
SS-09	67,110	0.031	463	6
SS-10	71,174	0.184	2,579	3
Total	**526,754**	**0.779**		

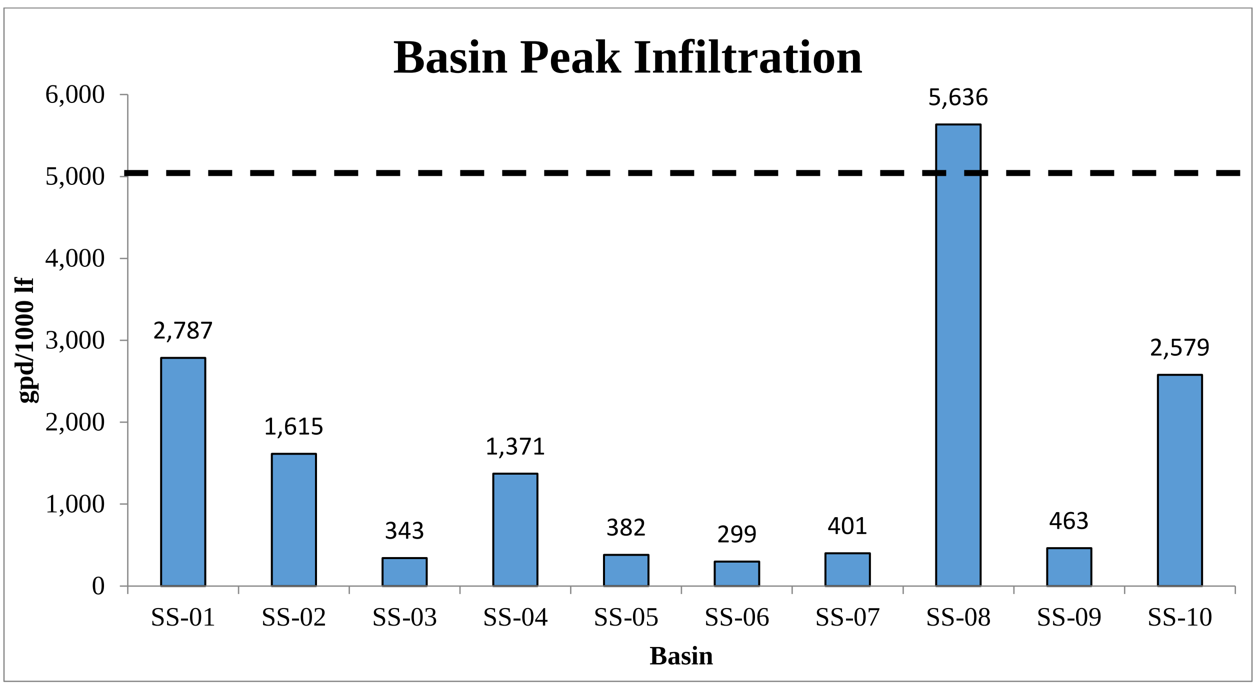


Figure 6.1 – Basin Peak Infiltration

Design Storm

The National Oceanic Atmospheric Administration (NOAA) has developed updated precipitation frequency estimates and temporal distributions as part of the NOAA Atlas 14. Two design storms were used for the Collection System Master Plan. A 1-Year/1-Hour design storm was used for RDII analysis to project wet weather flows. A 2-Year/24-Hour design storm was used for hydraulic capacity analysis. The rainfall intensities for the associated designs storms are shown in Table 6.3 below.

Table 6.3

RAINFALL INTENSITIES FOR VARIOUS STORM RECURRENCE INTERVALS

Storm Intensity (Year-Duration)	Total Rainfall (in)
1-Year/1-Hour	1.45
2-Year/24-Hour	4.10

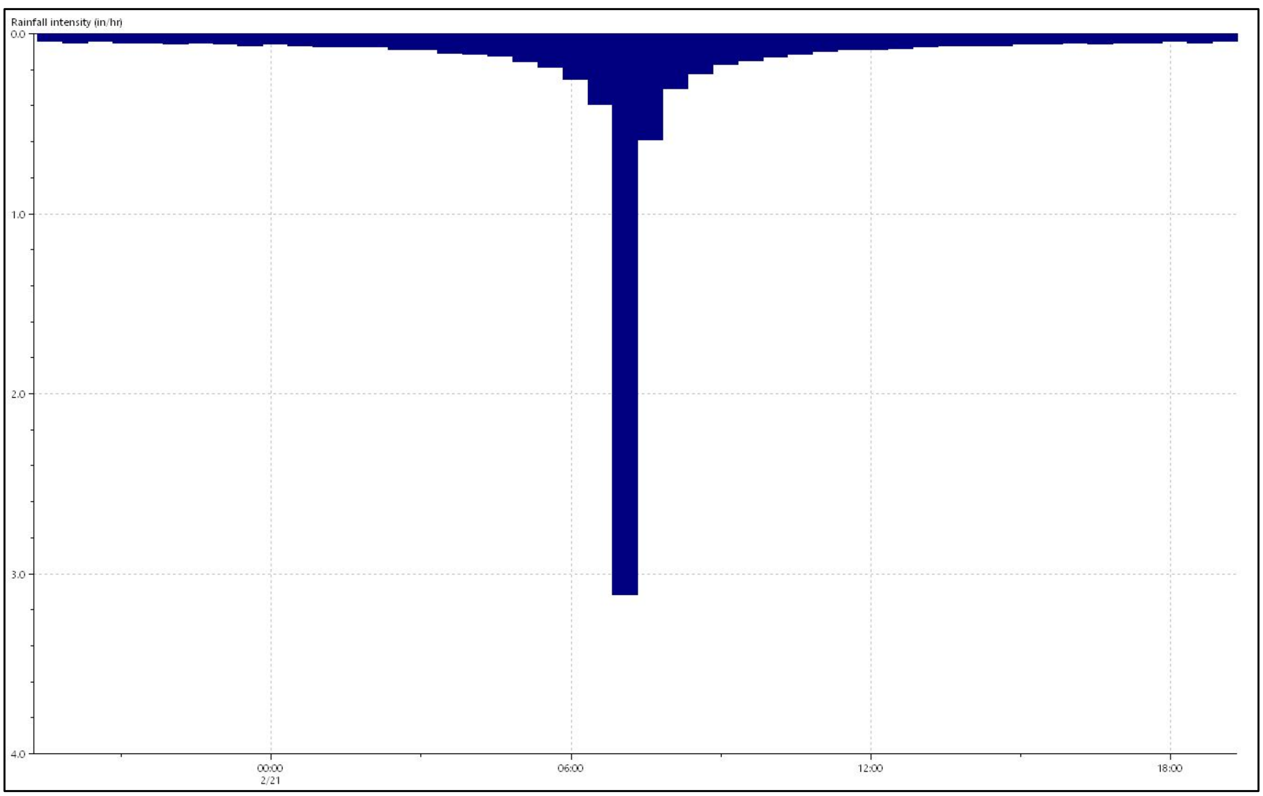


Figure 6.2 – 2-Year/24-Hour Design Storm

Determination of Peak Inflow

Inflow in a sanitary sewer system is defined as extraneous flow that is a direct result of stormwater runoff. Inflow may enter the sanitary sewer system through directly connected downspouts, area drains, cleanouts, and building sewers. Stormwater may also enter the system through direct or indirect connections between the sanitary sewers and storm drains or ditches, sewer line defects, and through defective manhole covers, frame seals, corbels, and manhole walls.

Flow data collected during wet-weather periods was analyzed to determine peak inflow originating in each basin. To determine the peak inflow rate, the sum of base flow and infiltration was subtracted from the peak instantaneous flow observed immediately following a rain event. Base flow and infiltration was generally determined from flow data 24 hours prior to the time that the maximum flow rate occurred following the rainfall event. The peak inflow rate was then plotted against the 1-hour and the 24-hour rainfall intensity for the corresponding rain event. Historical rainfall intensities data for the selected rainfall events are given in Table 6.3 on page 6-4.

The analysis projected the peak 1-year/1-hour storm (1.45 inches/hour) inflow rate to be 2.948 MGD. A summary of the peak wet-weather flow rates during a 1-year/1-hour storm event is given in Table 6.4 shown below.

Table 6.4
SUMMARY OF INFLOW RATES: 1-YEAR/1-HOUR

Meter Basin	Basin Footage (LF)	Basin Peak 1-Year/1-Hour Wet Weather Flow Rate (MGD)	Basin Unit Inflow Ratio (GPD/1,000 LF)	Ranking
SS-01	21,737	0.217	9,983	2
SS-02	96,578	0.901	9,329	3
SS-03	37,871	0.057	1,505	9
SS-04	28,444	0.129	4,535	6
SS-05	68,992	0.325	4,711	5
SS-06	30,181	0.110	3,645	8
SS-07	62,904	0.392	6,232	4
SS-08	41,763	0.450	10,775	1
SS-09	67,110	0.084	1,252	10
SS-10	71,174	0.283	3,976	7
Total	**526,754**	**2.948**	**5,594**	

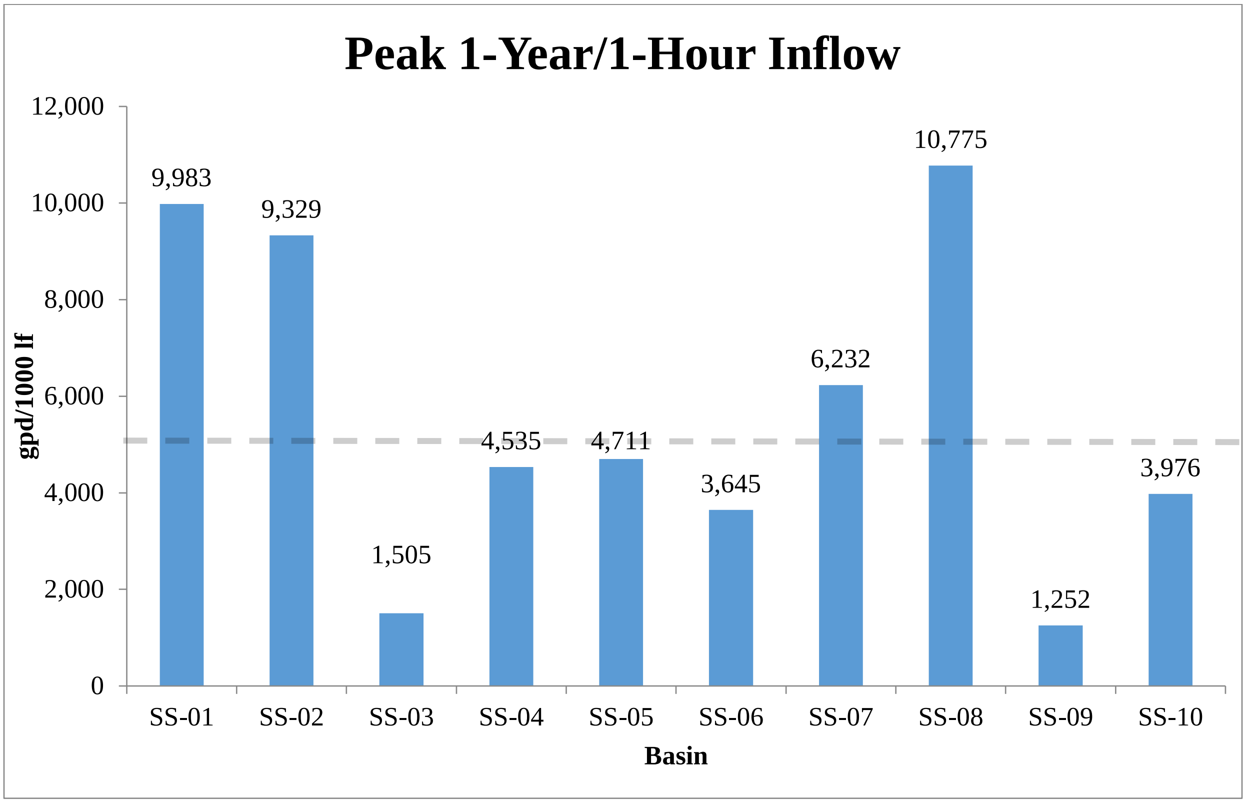


Figure 6.3 – Peak 1-Year/1-Hour Inflow

The discrete basin inflow rate from the analysis can be expressed in terms of a discrete basin unit inflow coefficient of GPD/1,000 LF. The discrete basin unit inflow coefficient expresses the magnitude of peak inflow relative to other basins.

For this report, excessive 1-year/1-hour inflow rate is considered to be an inflow coefficient greater than 5,000 GPD/1,000 LF. Inflow analysis determined basin SS-08 exhibits an excessive 1-year/1-hour inflow rate of 10,775 GPD/1,000 LF.

Summary of Inflow and Infiltration Rates

A summary of inflow and infiltraton rates are show in Table 6.5. Recommendations for future SSES are listed in Table 6.6 with Priory 1 basins having the most impact on reducing I/I. It should be noted that basin SS-10 showed an inflow spike on May 3, 2018. This is indicative to a flash flood event that overtopped a manhole cover for a short time as the flow recovered quickly after the storm passed through. This data was excluded from the I/I analysis.

Table 6.5 SUMMARY OF INFLOW AND INFILTRATION RATES						
Inflow Ranking	**Meter Basin**	**Basin Footage (LF)**	**Basin Peak 1-Year/1-Hour Inflow Rate (MGD)**	**Basin Unit Inflow Ratio (GPD/1,000 LF)**	**Basin Peak Infiltration (MGD)**	**Basin Peak Unit Infiltration (GPD/1000 LF)**
1	SS-08	41,763	0.450	10,775	0.235	5,636
2	SS-01	21,737	0.217	9,983	0.061	2,787
3	SS-02	96,578	0.901	9,329	0.156	1,615
4	SS-07	62,904	0.392	6,232	0.025	401
5	SS-05	68,992	0.325	4,711	0.026	382
6	SS-04	28,444	0.129	4,535	0.039	1,371
7	SS-10	71,174	0.283	3,976	0.184	2,579
8	SS-06	30,181	0.110	3,645	0.009	299
9	SS-03	37,871	0.057	1,505	0.013	343
10	SS-09	67,110	0.084	1,252	0.031	463
	Total	**526,754**	**2.948**		**0.779**	

Table 6.6 RECOMMENDED SSES PRIORITY BY BASIN		
Priority	**Basins**	**Total Length (LF)**
1	SS-08	222,982
	SS-01	
	SS-02	
	SS-07	
2	SS-05	303,772
	SS-04	
	SS-10	
	SS-06	
	SS-03	
	SS-09	
Total		**526,754**

Existing Capacity Analysis

Dry-Weather

The current network has sufficient capacity to transport the dry-weather flows within the calibrated model. No improvements are recommended at this time.

Wet-Weather

Manhole spills are predicted to occur at approximately 32 locations within the existing system scenario. A map indicating the existing model predicted SSOs is shown in Exhibit 2.

Contributing factors are undersized sewer lines, excessive I/I, and undersized or underperforming pump stations. For the purpose of master planning, capital improvements will be recommended for future growth.

Lift Stations

Please refer to Table 3.2 Lift Station Pumping Capacity in Chapter 3 – Lift Station Evaluations for immediate existing needs. Items to address are pumps not operating and/or pump underperformed relative to the original design capacity. A detailed capital improvement plan is presented in Chapter 7 – Capital Improvement Plan for lift stations requiring future capacity upgrades.

Summary of Immediate Capacity Improvements

Model predicted overflows are located with basins SS-01, SS-02, SS-05, SS-07, and SS-08. SSES is recommend in basins SS-01, SS-02, SS-07, and SS-08.

Basin SS-05 is not recommended for SSES. Approximately 4,300 feet of 10-inch and 12-inch gravity sewer upgrades are recommended in basin SS-05 from W. Harvard St. upstream to W. Hannibal St. These improvements will eliminate a hydraulic bottleneck and convey wastewater flows downstream for both current and future flows. The Mt. Olive lift station is currently operating at approximately 61 GPM which is 44% of the design capacity of 140 GPM.

A detailed capital improvement plan is presented in Chapter 7 – Capital Improvement Plan.

Ultimate Build-Out Capacity Analysis

Details of the system performance of the ultimate build-out network are provided below.

Dry-Weather

The ultimate build-out network has sufficient capacity to transport the dry-weather flows predicted for the ultimate scenario. It is anticipated that growth areas within SS-07 and SS-10 will require gravity sewer extensions. The sewer lines should be sized for both dry-weather and wet-weather capacities.

Wet-Weather

Manhole spills are predicted to occur at approximately 78 locations within the ultimate build-out scenario. A map indicating the ultimate model predicted SSOs is shown in Exhibit 3.

Contributing factors are undersized sewer lines, excessive I/I, and undersized or underperforming pump stations.

Lift Stations

Pump stations that are recommended for maintenance or upgrades are listed below:

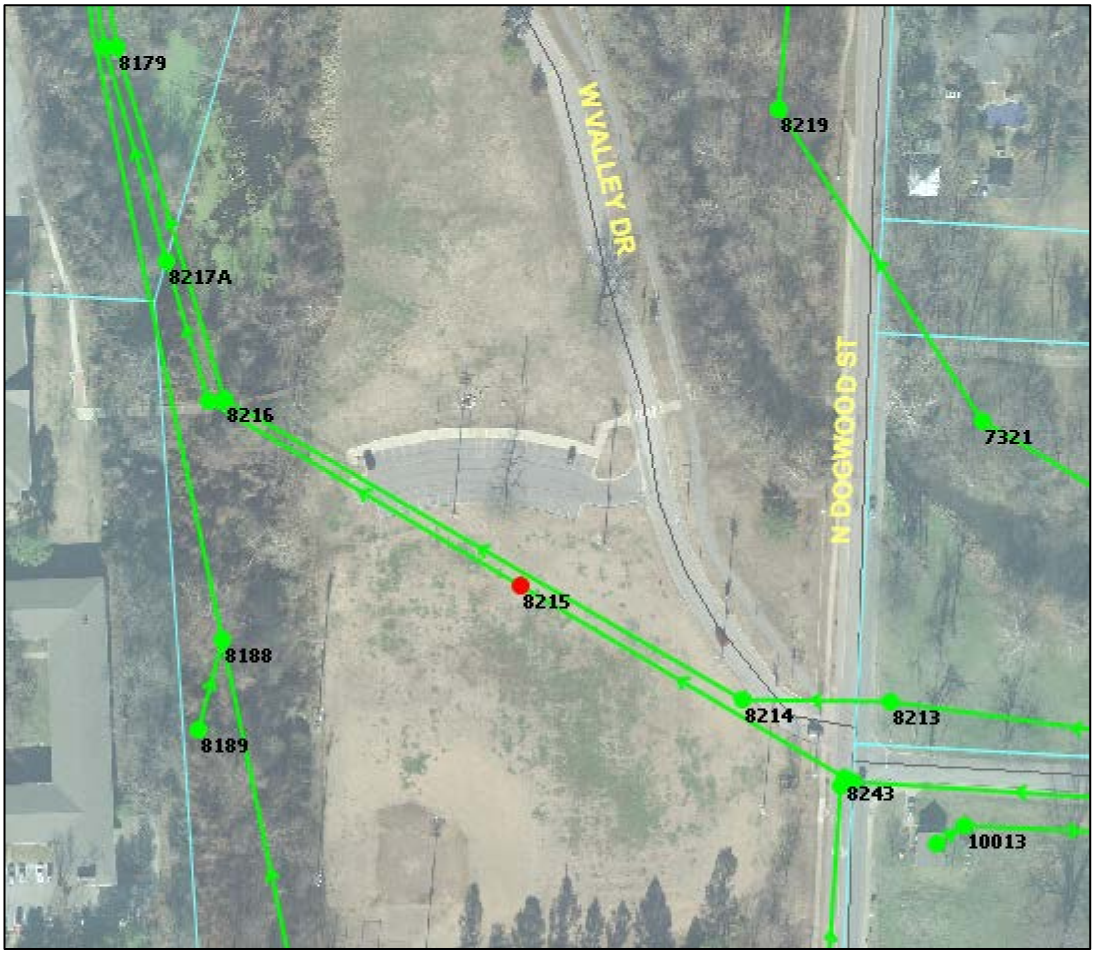
Table 6.7
LIFT STATION EXISTING AND ULTIMATE PUMPING CAPACITY

Lift Station	Existing Pumping Capacity (GPM)	Ultimate Pumping Capacity (GPM)
Dogwood LS Pump 1	41.7	97.2
Dogwood LS Pump 2	41.7	97.2
Mt. Olive LS Pump 1	111.1	152.8
Mt. Olive LS Pump 2	111.1	152.8
Stone Crest LS Pump 1	52.9	125.0
Stone Crest LS Pump 2	52.9	125.0
Villa View LS Pump 1	486.1	729.1
Villa View LS Pump 2	486.1	729.1
Maloree LS Pump 1	97.2	111.1
Maloree LS Pump 2	97.2	111.1

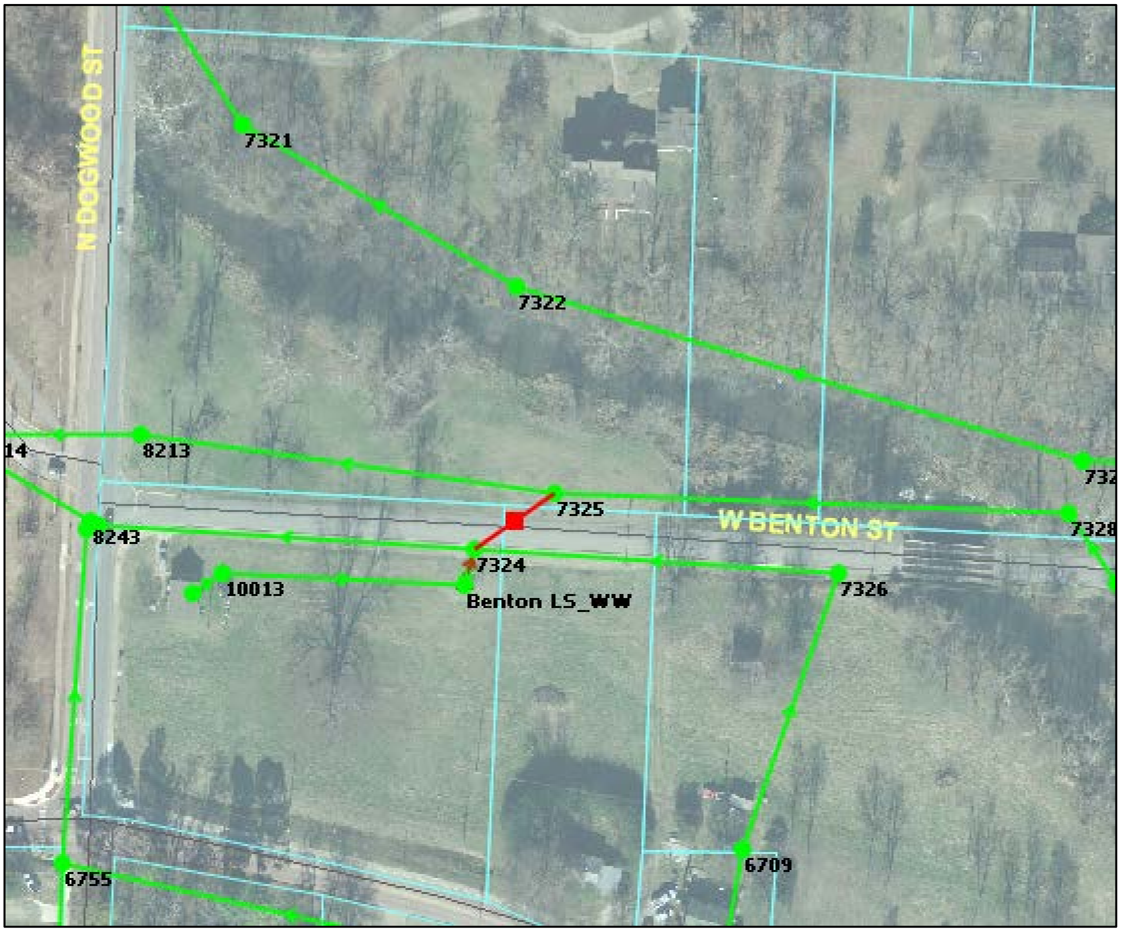
The Simmons LS has been modeled as a wet-weather diversion during the design storm. All incoming flow is pumped into SS-09. It is recommended to adjust the operating levels to Pump 1 ON at 1097 feet, Pump 2 ON at 1098 feet, and all Pumps OFF at 1094 feet. Other alternatives at the Simmons LS site can be considered in the future.

Additional Collection System Recommendations

A few items are recommended within the collection system to mitigate overflows during the ultimate build-out scenario.



Install bolt down frame and cover on MH 8215.



Install high flow connection between MH 7325 and MH 7324 at elevation 1065 feet.

Raise interconnection between MH 7057 and MH 7056 by 2 feet.

Abandon sewer lines 7068:7060 and 10030:7060.

Summary of Ultimate Capacity Improvements

Five (5) sewer lift stations are recommended for capacity improvements beyond their current design capacities. A parallel 8-inch force main is recommended for the Villa View lift station. To mitigate overflows during the ultimate design scenario, a minimum recommendation of gravity sewer lines ranging from 8-inches to 48-inches in diameter for approximately 33,600 feet are required to maintain sewer flows within the collection system. Force main upgrades of approximately 1,050 feet ranging from 4-inches to 8-inches in diameter are required for two lift stations.

An additional approximately 6,600 feet of gravity sewer upgrades ranging from 10-inches to 30-inches in diameter are required to maintain the hydraulic grade line a minimum of three (3) feet below the top of any given manhole within the collection system. Approximately 13,600 feet of gravity sewer ranging in size from 10-inches to 15-inches in diameter may be required to service areas that do not currently have sanitary sewer service.

A detailed capital improvement plan is presented in Chapter 7 – Capital Improvement Plan.

Wastewater Treatment Plant Flows

A summary for WWTP flows for dry-weather and wet-weather design storm for existing and ultimate build-out scenarios are shown in Table 6.8 below.

Table 6.8
WASTEWATER TREATMENT PLANT FLOWS

Descriptions	ADWF (MGD)	Peak Hour ADWF (MGD)	Peak Hour 2-yr/24-hr (MGD)	Peak Hour 2-yr/24-hr (MGD)
Existing System	1.77	2.38	12.13	24.34*
Buildout System	3.72	4.88	15.80	36.93

**Value does not account for 0.57 MG overflow in the system*

CHAPTER 7 – CAPITAL IMPROVEMENT PLAN

Immediate conveyance improvements without storage is not a recommended alternative. This alternative will not allow for growth and development within the City of Siloam Springs, as it requires maintaining the existing population. The recommended improvements are based on full conveyance of model predicted flows without a reduction in inflow or infiltration. Identifying and reducing I/I will directly impact recommended sewer improvements. The sewer hydraulic model can be used to intermittently assess the collection system to determine future planning needs.

Priority 1 recommendations mitigate model predicted SSOs with surcharging, but without overflowing the system. Priority 2 recommendations mitigate model predicted SSOs without overflowing the system while maintaining a minimum 3 feet of freeboard in all sanitary sewer manholes. Costs are based on present value construction and do not include a 15% engineering fee or 20% contingency.

SSES Recommendations

Excessive I/I was identified in four (4) basins. SSES projects may identify sources of I/I and be a cost effective measure to reduce wastewater flows during wet-weather. Reducing I/I will reduce the recommended capacity upgrades. The basins recommended for future SSES are identified in Table 7.1 below.

Table 7.1
RECOMMENDED SSES PRIORITY BY BASIN

Priority	Basins	Total Length (lf)
1	SS-08	41,763
	SS-01	21,737
	SS-02	96,578
	SS-07	62,904
Total		**222,982**

Budgetary pricing for full SSES is approximately $3.50 per linear foot for a total of $780,437.

Capacity Improvement Analysis

Phase 1

Recommendations for Priority 1 improvements are listed in Table 7.2 below.

Table 7.2 PHASE 1 CAPACITY IMPROVEMENTS						
US MH	**DS MH**	**Existing Diameter (in.)**	**Proposed Diameter (in.)**	**Length (LF)**	**Unit Cost ($)**	**Total Cost ($)**
10097	10098	8	10	7.7	$150	$1,155
10098	7170	8	10	51.4	$150	$7,710
10099	7162	8	10	308.3	$150	$46,245
10100	7155	8	10	297.4	$150	$44,610
10101	7154	8	10	230.2	$150	$34,530
10286	7464	8	10	362.2	$150	$54,330
10333	7499	8	12	86.9	$180	$15,642
10339	7497	8	12	96.9	$180	$17,442
10348	7498	8	12	135.5	$180	$24,390
10389	7847	8	10	48.0	$150	$7,200
10392	10389	8	10	398.1	$150	$59,715
6709	7326	8	10	241.4	$150	$36,210
6710	6709	8	10	213.4	$150	$32,010
6711	6710	8	15	337.7	$225	$75,983
6713	6711	8	15	351.3	$225	$79,043
6716	6713	6	8	213.1	$120	$25,572
6722	6716	6	8	264.7	$120	$31,764
6723	6722	6	8	161.0	$120	$19,320
6725	6723	6	8	283.6	$120	$34,032
6726	6725	6	8	148.7	$120	$17,844
6730	6726	6	8	206.2	$120	$24,744
6731	6730	6	10	66.4	$150	$9,960
6732	6731	6	10	97.3	$150	$14,595
6733	6732	6	10	220.8	$150	$33,120
6734	6733	6	10	175.7	$150	$26,355
6736	6734	6	10	56.9	$150	$8,535
6934	6933	10	12	179.5	$180	$32,310
6935	6934	10	12	69.7	$180	$12,546
6937	6940	8	12	276.7	$180	$49,806
6938	6939	8	10	143.1	$150	$21,465
6939	6937	8	12	18.2	$180	$3,276
6940	6940A	8	12	50.6	$180	$9,108
6940A	6941	8	12	76.4	$180	$13,752

Table 7.2
PHASE 1 CAPACITY IMPROVEMENTS (Contd.)

US MH	DS MH	Existing Diameter (in.)	Proposed Diameter (in.)	Length (LF)	Unit Cost ($)	Total Cost ($)
6941	6942	10	12	141.7	$180	$25,506
6942	8141	10	12	89.6	$180	$16,128
6944	6935	10	12	184.0	$180	$33,120
6968	6938	8	10	477.8	$150	$71,670
6969	6968	8	10	324.9	$150	$48,735
6976	6969	8	10	332.2	$150	$49,830
6978	6978A	8	10	92.8	$150	$13,920
6978A	6976	8	10	68.3	$150	$10,245
6979	6978	8	10	253.3	$150	$37,995
6980	6979	8	10	128.9	$150	$19,335
6982	6980	8	10	298.0	$150	$44,700
6985	6982	8	10	426.4	$150	$63,960
6989	6985	8	10	114.5	$150	$17,175
6990	6989	8	10	140.3	$150	$21,045
6999	6990	8	10	324.0	$150	$48,600
7062	10030	15	18	49.2	$270	$13,284
7065	7062	12	18	55.5	$270	$14,985
7067	7065	12	18	164.4	$270	$44,388
7074	7067	10	18	94.9	$270	$25,623
7075	7074	10	18	121.8	$270	$32,886
7076	7075	10	18	102.3	$270	$27,621
7079	7076	10	18	142.5	$270	$38,475
7080	7079	15	18	74.1	$270	$20,007
7081	7080	12	18	142.9	$270	$38,583
7082	7081	15	18	45.1	$270	$12,177
7090	7082	15	18	696.9	$270	$188,163
7091	7090	15	18	142.7	$270	$38,529
7092	7091	15	18	202.5	$270	$54,675
7100	7103	15	18	170.1	$270	$45,927
7103	7092	15	18	201.9	$270	$54,513
7123	7100	15	18	319.1	$270	$86,157
7128	7123	15	18	319.8	$270	$86,346
7149	7130	10	15	270.0	$225	$60,750
7152	7149	8	10	208.6	$150	$31,290
7153	7152	8	10	316.2	$150	$47,430
7154	7153	8	10	219.9	$150	$32,985
7155	10101	8	10	73.3	$150	$10,995
7156	10100	8	10	108.2	$150	$16,230
7157	7156	8	10	152.9	$150	$22,935

Table 7.2 PHASE 1 CAPACITY IMPROVEMENTS (Contd.)						
US MH	**DS MH**	**Existing Diameter (in.)**	**Proposed Diameter (in.)**	**Length (LF)**	**Unit Cost ($)**	**Total Cost ($)**
7162	7157	8	10	314.8	$150	$47,220
7169	10099	8	10	290.9	$150	$43,635
7170	7169	8	10	233.2	$150	$34,980
7372	7371	6	8	160.0	$120	$19,200
7466	10286	8	10	125.0	$150	$18,750
7467	7466	8	10	184.7	$150	$27,705
7468	7467	8	10	347.2	$150	$52,080
7482	7480	30	36	235.9	$540	$127,386
7485	7482	30	36	136.5	$540	$73,710
7486	7485	30	36	138.7	$540	$74,898
7487	7486	30	36	198.4	$540	$107,136
7488	7487	30	36	246.8	$540	$133,272
7492	7488	30	36	208.1	$540	$112,374
7497	7523A	8	12	271.9	$180	$48,942
7498	10339	8	12	272.8	$180	$49,104
7499	10348	8	12	142.3	$180	$25,614
7500	10333	8	10	198.2	$150	$29,730
7549	7548	6	8	324.1	$120	$38,892
7719	7755	8	12	373.6	$180	$67,248
7720	7719	8	12	366.9	$180	$66,042
7721	7720	8	12	301.6	$180	$54,288
7722	7721	8	12	212.2	$180	$38,196
7723	7722	8	12	218.2	$180	$39,276
7724	7723	8	12	332.9	$180	$59,922
7725	7724	8	12	328.6	$180	$59,148
7726	7725	8	15	350.0	$225	$78,750
7727	7726	8	15	290.8	$225	$65,430
7728	7727	8	15	350.0	$225	$78,750
7729	7728	8	15	350.0	$225	$78,750
7730	7729	8	15	371.3	$225	$83,543
7731	7730	8	10	96.4	$150	$14,460
7732	7731	8	10	385.7	$150	$57,855
7755	7756	8	15	101.3	$225	$22,793
7756	7757	8	15	208.8	$225	$46,980
7757	7758	8	15	668.6	$225	$150,435
7758	7759	8	15	483.3	$225	$108,743
7759	7760	8	15	274.8	$225	$61,830
7760	7761	8	15	291.3	$225	$65,543
7761	7762	10	15	215.9	$225	$48,578

Table 7.2
PHASE 1 CAPACITY IMPROVEMENTS (Contd.)

US MH	DS MH	Existing Diameter (in.)	Proposed Diameter (in.)	Length (LF)	Unit Cost ($)	Total Cost ($)
7814	7814A	12	15	372.5	$225	$83,813
7815	7814	12	15	104.0	$225	$23,400
7819	7815	12	15	356.2	$225	$80,145
7822	7819	12	15	296.7	$225	$66,758
7823	7822	12	18	315.7	$270	$85,239
7824	7823	12	18	219.0	$270	$59,130
7825	7824	12	18	197.6	$270	$53,352
7826	7825	12	18	252.4.0	$270	$68,148
7827	7826	12	18	195.0	$270	$52,650
7828	7857	8	10	369.9	$150	$55,485
7847	7848	8	10	273.0	$150	$40,950
7848	7851	8	10	34.7	$150	$5,205
7850	7853	8	10	316.7	$150	$47,505
7851	7850	8	10	351.8	$150	$52,770
7853	7854	8	10	244.7	$150	$36,705
7854	7855	8	10	169.4	$150	$25,410
7855	7828	8	10	341.6	$150	$51,240
7858	7827	12	18	446.5	$270	$120,555
7874	7858	12	15	175.0	$225	$39,375
7875	7874	12	15	399.6	$225	$89,910
7876	7875	12	15	247.3	$225	$55,643
7877	7876	12	15	261.1	$225	$58,748
7878	7877	12	15	77.9	$225	$17,528
7897	7878	12	15	206.7	$225	$46,508
7898	7897	8	15	111.3	$225	$25,043
7899	7898	8	15	224.3	$225	$50,468
7900	7899	8	15	319.7	$225	$71,933
7901	7900	8	15	458.0	$225	$103,050
8141	6944	10	12	53.3	$180	$9,594
8180	8180A	22	30	67.9	$450	$30,555
8180A	7479	22	30	308.3	$450	$138,735
8183	8180	22	30	209.2	$450	$94,140
8210	8211	22	30	122.5	$450	$55,125
8211	8183	22	30	251.5	$450	$113,175
8240	7479	30	48	15.8	$720	$11,376
9918	7468	8	10	344.9	$150	$51,735
9919	9918	8	10	339.0	$150	$50,850
9920	9919	8	10	289.3	$150	$43,395
		Subtotal Gravity		**33,608.20**		**$7,067,763**

Phase 2

Lift station force main upgrades are shown in Table 7.3. Lift station upgrade improvements are shown in Table 7.4.

Table 7.3
PHASE 2 FORCE MAIN IMPROVEMENTS

US MH	Existing Diameter (in.)	Proposed Diameter (in.)	Length (LF)	Unit Cost ($)	Total Cost ($)
Dogwood FM	2	4	452.5	$80	$36,200
Villa View FM	8	8	3,346.10	$120	$401,532
Western Hill FM	6	8	589.7	$120	$70,764
		Total	**4,388.30**		**$508,496**

Table 7.4
LIFT STATION EXISTING AND ULTIMATE PUMPING CAPACITY

Lift Station	Existing Pumping Capacity (gpm)	Ultimate Pumping Capacity (gpm)
Dogwood LS Pump 1	41.7	97.2
Dogwood LS Pump 2	41.7	97.2
Mt. Olive LS Pump 1	111.1	152.8
Mt. Olive LS Pump 2	111.1	152.8
Stone Crest LS Pump 1	52.9	125.0
Stone Crest LS Pump 2	52.9	125.0
Villa View LS Pump 1	486.1	729.1
Villa View LS Pump 2	486.1	729.1
Maloree LS Pump 1	97.2	111.1
Maloree LS Pump 2	97.2	111.1

Recommendations for Priority 2 improvements are listed in Table 7.5 below.

Table 7.5
PHASE 2 CAPACITY IMPROVEMENTS

US MH	DS MH	Existing Diameter (in.)	Proposed Diameter (in.)	Length (LF)	Unit Cost ($)	Total Cost ($)
6995	6994	8	10	309.7	$200	$61,940
7005	6995	8	10	117.6	$150	$17,640
7007	7005	8	10	164.6	$150	$24,690
7008	7007	8	10	295.1	$150	$44,265
7470	9921	8	10	289.7	$150	$43,455
7471	7470	8	10	178.3	$150	$26,745
7558	7557	8	10	385.5	$150	$57,825
7563	7558	8	10	291.3	$150	$43,695
7565	7563	8	10	288.4	$150	$43,260
7913	7901	8	10	137.7	$150	$20,655
7914	7913	8	10	116.9	$150	$17,535
7918	7914	8	10	151.9	$150	$22,785
9921	9920	8	10	302.4	$150	$45,360
9981	8033	8	10	385.8	$150	$57,870
10045	6765	10	12	215.0	$180	$38,700
6762	6761	10	12	287.5	$180	$51,750
6763	6762	10	12	135.0	$180	$24,300
6765	6763	10	12	172.2	$180	$30,996
6779	6773	10	12	128.7	$180	$23,166
6780	6779	10	12	147.4	$180	$26,532
6829	6780	10	12	754.8	$180	$135,864
8190	8188	12	15	397.1	$225	$89,348
8212	8190	12	15	76.5	$225	$17,213
8178	8210	22	30	188.2	$450	$84,690
8215	8217	22	30	324.8	$450	$146,160
8217	8217A	22	30	133.9	$450	$60,255
8217A	8178	22	30	198.7	$450	$89,415
			Total	**6,574.7**		**$1,346,109**

Phase 3

Recommendations for Priority 3 sewer extensions are listed in Table 7.6 below.

Table 7.6 PHASE 3 SEWER EXTENSIONS					
US MH	**DS MH**	**Proposed Diameter (in.)**	**Length (LF)**	**Unit Cost ($)**	**Total Cost ($)**
FG-01	FG-02	10	686.4	200	$137,280
FG-02	FG-03	10	556.0	150	$83,400
FG-03	FG-04	10	407.4	150	$61,110
FG-04	FG-05	10	780.6	150	$117,090
FG-05	FG-06	10	662.9	150	$99,435
FG-06	7245	10	395.1	150	$59,265
FG-07	FG-08	10	745.6	150	$111,840
FG-08	FG-09	10	596.3	150	$89,445
FG-09	FG-10	10	679.1	150	$101,865
FG-10	FG-11	10	482.6	150	$72,390
FG-11	9981	10	221.6	150	$33,240
FG-21	FG-17	10	1,937.9	150	$290,685
FG-22	FG-21	10	1,312.2	150	$196,830
FG-17	FG-18	15	1,313.0	225	$295,425
FG-18	FG-19	15	1,387.9	225	$312,278
FG-19	FG-20	15	990.7	225	$222,908
FG-20	8022A	15	442.0	225	$99,450
		Total	**13,597.30**		**$2,383,935**

Wastewater Treatment Plant Improvements

Wastewater treatment plant improvements will need to be considered when the average daily flows exceed 80% of the current design capacity of 4.4 MGD which is 3.52 MGD.

Capital Improvement Plan Cost Summary

A summary of phased linework improvements is shown in Table 7.7 below.

Table 7.7 CAPITAL IMPROVEMENT PLAN SUMMARY	
Category	**Estimated Cost ($)**
Phase 1 gravity	$7,067,763
Phase 2 force main	$508,496
Phase 2 gravity	$1,346,108
Phase 3 gravity extensions	$2,383,935
Total gravity/force main	**$11,306,302**

Lift station pump upgrades will be site specific. Various factors to consider depending on the pump manufacturer, power requirements, site conditions, etc. Estimated costs for pump station upgrades are listed in Table 7.8 below.

Table 7.8 LIFT STATION IMPROVEMENT SUMMARY	
Category	**Estimated Cost ($)**
Dogwood LS	$30,000
Mt. Olive LS	$60,000
Stone Crest LS	$45,000
Villa View LS	$75,000
Maloree LS	$50,000
Total	**$260,000**

Capital Improvement Plan Schedule

Timing for the capital improvement plan is directly dependent on sewer rates to support the expenditures. It is recommended that the capital costs be utilized in the next sewer rate analysis and a 10-year schedule be developed.

Cleaning and Televising

As part of the future collection system maintenance recommendations, cleaning & televising 20% of the wastewater collection system is recommended annually. Cleaning and/or televising every five years is a common guideline used by the EPA and other agencies. The benefits include dry weather overflow (blockage) prevention and allowing continual observation of the system. The operators will also be able to keep a plan in place in regard to age and condition of the system. Cleaning and televising also helps keep mapping activities up to date with measured distances between manholes.

Hydraulic Model Maintenance and Updating

The City has made an investment to build and calibrate the wastewater collection system model. The hydraulic model will need to be kept up to date periodically by including infrastructure improvements and performance checks with the recommended permanent flow meters. Ideally, the model would be kept up to date and used as a planning tool for analyzing each major development for both existing and ultimate build-out scenarios at the design storm. Also, the model should be updated with all constructed sewer improvements on an annual or bi-annual basis with record drawings. This also serves a dual purpose to keep the wastewater collection system GIS map up to date.

It is recommended the wastewater collection system model be re-calibrated approximately every 5-7 years depending on the results of the annual or bi-annual updates. The re-calibration of the model will include installation of temporary flow meters and rain gauges to supplement the City's permanent flow meter network. Sub-basins and catchments will be reviewed and updated. Population and land use data will also be evaluated and updated to reflect current conditions.

Maintaining a model will involve adding new sewer lines to the model as a result of development, sewer capacity enhancements, adding storage, updating WWTP inlet capacity, and lift station capacity improvements as they are built.