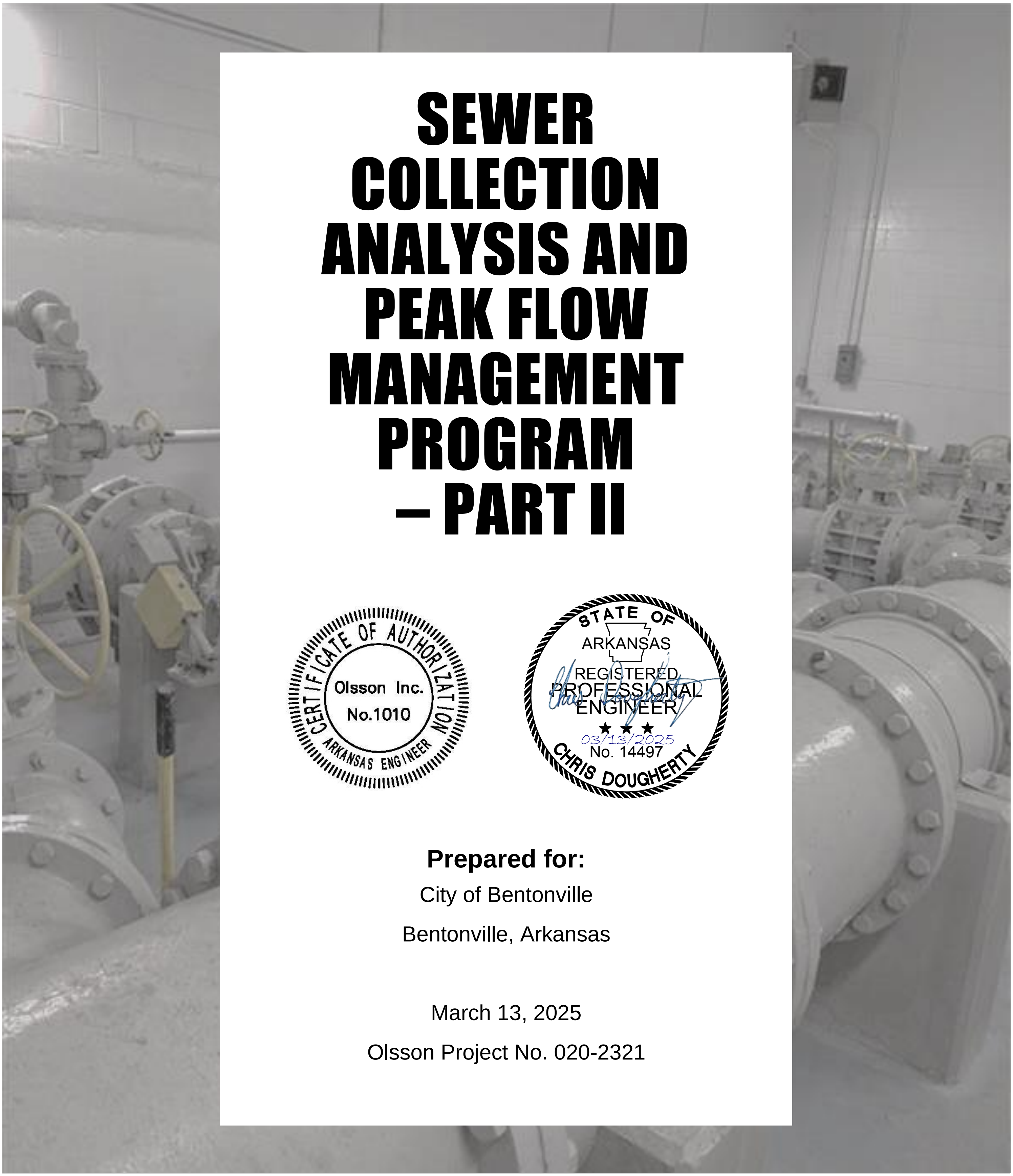

# SEWER COLLECTION ANALYSIS AND PEAK FLOW MANAGEMENT PROGRAM – PART II

CERTIFICATE OF AUTHORIZATION
Olsson Inc.
No.1010
ARKANSAS ENGINEER

STATE OF ARKANSAS
REGISTERED PROFESSIONAL ENGINEER
03/13/2025
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**Prepared for:**

City of Bentonville

Bentonville, Arkansas

March 13, 2025

Olsson Project No. 020-2321

olsson®

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# ACRONYMS AND ABBREVIATIONS

| | |
|---|---|
| AF | Average Annual Flow |
| ARI | Average Recurrence Interval |
| BWU | Bentonville Water Utilities |
| CCTV | Closed-Circuit Television |
| CIP | Capital Improvements Plan |
| DIP | Ductile Iron Pipe |
| DWF | Dry-Weather Flow |
| EPA | Environmental Protection Agency |
| EPS | Extended Period Simulation |
| FLUM | Future Land Use Map |
| FM | Flow Meter |
| Ft | Feet |
| ft/s | Feet per Second |
| GIS | Geographic Information System |
| GPM | Gallons per Minute |
| GPS | Global Positioning System |
| IDM | Inch-Diameter-Mile |
| I/I | Inflow and Infiltration |
| In | Inch |
| LF | Linear Feet |
| MG | Million Gallons |
| MGD | Million Gallons per Day |
| MH | Manhole |
| MK | McKisik |
| NACA | Northwest Arkansas Conservation Authority |
| NDA | Non-Disclosure Agreement |
| NOAA | National Oceanic and Atmospheric Administration |
| PVC | Polyvinyl Chloride Pipe |
| RDII | Rainfall Derived Inflow and Infiltration |
| RTK | Real Time Kinematic |
| SF | Square Feet |
| SL-RAT | Sewer Line Rapid Assessment Tool |
| SLS | South Lift Station |

| | |
|---|---|
| SM | Shewmaker |
| SSES | Sanitary Sewer Evaluation Study |
| SSO | Sanitary Sewer Overflows |
| SSOAP | Sanitary Sewer Overflow Analysis and Planning |
| SUH | Synthetic Unit Hydrograph |
| TB | Town Branch |
| VCP | Vitrified Clay Pipe |
| WWF | Wet-Weather Flow |
| WRRF | Water Resource Recovery Facility |
| WWTF | Wastewater Treatment Facility |

# EXECUTIVE SUMMARY

## Introduction

Bentonville Water Utilities (BWU) management and staff continue to show their commitment to the Utility's Vision Statement that assures customers, citizens, and businesses, that the Utility will continue to be proactive in developing water and wastewater infrastructure and provide a high-quality level of service. This commitment is reflected in the Utility's embarkment on developing the peak flow management program, which is intended to identify constraints in the wastewater collection that that will impact the levels of service offered to current and future customers, and to accommodate the growth projected in the region.

Olsson was retained by the Utility to develop a comprehensive plan that will result in developing alternatives for managing peak flows and recommendations of improvements necessary to continue serving the growth within the City. In order to support the analysis and decision making, the program included data collection that was comprised of a Sanitary Sewer Evaluation Survey (SSES) with Flow Monitoring, which was provided by a Subconsultant, TREKK Design Group, LLC.

Part II of the program continues to build on Part I, which established the baseline capacity for the current sewer system and identified areas that had excessive inflow and infiltration (I/I) for further inspection.

This report presents the outcomes of Part II of the program, which aims to achieve the following:

1) Establishment of Design Parameters.
2) Evaluation and Analysis of Peak Flow Management Alternatives.
3) Development of sewer system improvement recommendations.
4) Completing SSES Study for the areas identified in Part I.
5) Developing sewer rehabilitation recommendations.
6) Develop public and private I/I reduction programs

## Flow and Population Projections

The population of the City was estimated at 59,470 in 2023, according to the US Census Bureau, from which approximately 38,600 people lived within the study area. The City recently updated its Future Land Use Map (FLUM) which projected residential population and commercial population at ultimate buildout conditions, assumed to occur by 2050. The plan projected that the study area will have a total residential population of 116,510 and a commercial population of 149,650. Detailed discussion of the flow projections and model loading is presented in Section 5.2.

Intermediate design horizons were developed to provide a timeline for the recommended improvements. These design horizons are 2025, 2030, 2035, and 2045.

## Hydraulic Model Update

The hydraulic model, developed using InfoWorks ICM, includes facilities in the McKisic, Shewmaker, Town Branch, and South Lift Station basins. The model has a skeletal framework that only includes 25 miles of gravity sewers, 7.4 miles of force mains, and three primary lift stations McKisic (Dogwood and Turner), North, and South, in addition to three smaller lift stations (#39 Allencroft, #12 Blueberry, and #9 Rice Road) that were added later in the process. The extent of the hydraulic model is shown on Figure ES-1.

Dry weather flows and wet weather flow characteristics were updated in the hydraulic model, which was then calibrated against the most recent flow and rainfall data that was collected during this phase. The calibrated hydraulic model will be utilized to evaluate the available capacity within the existing system.

## Existing System Evaluation

Olsson recommended using a design storm with a 5-year return interval and 24-hour duration to evaluate the capacity of the system. This design approach balances preparation for significant events while avoiding over-design for rare extreme events. Design rainfall depth was derived from the National Oceanic and Atmospheric Administration (NOAA) Atlas, while its distribution was modeled after the Natural Resources Conservation Service (NRCS) synthetic storm Type II hyetograph.

The evaluation was intended to identify pipes that surcharge under the design flows, evaluate the performance of pump stations, and estimate design flows at the WRRF for each design horizon.

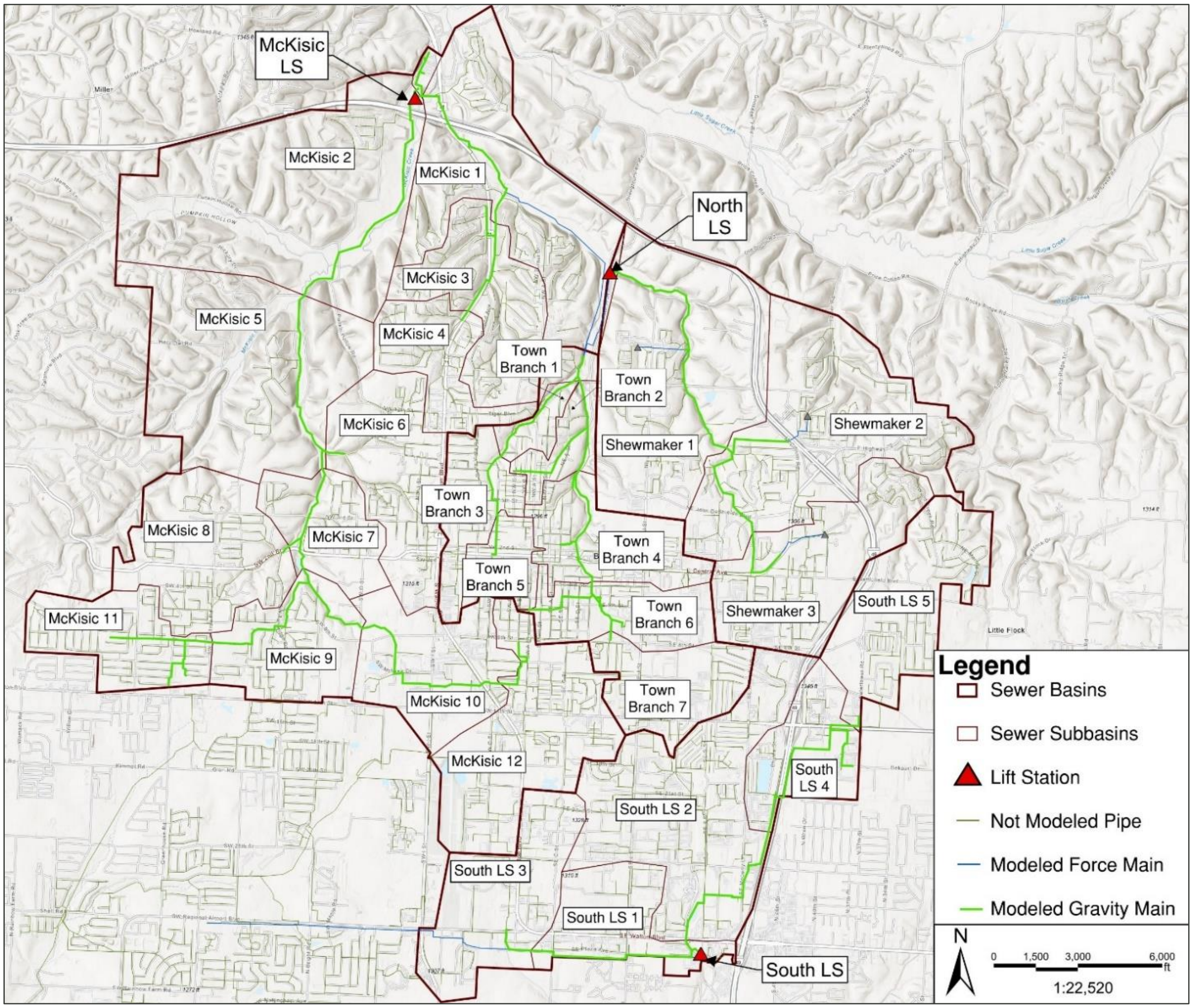



**Figure ES-1. Hydraulic Model Extent.**

## Recommended Pipe and Manhole Rehabilitation

In Part I of this study, Olsson recommended that the City conduct a Sanitary Sewer Evaluation Study (SSES) to identify potential sources of infiltration and inflow (I/I) within the public areas of the collection system. The focus was on seven (7) priority basins with the highest rates of I/I, as shown in Figure 11.

In collaboration with TREKK, the Part II (referred to as Phase II in the TREKK Technical Memo) of the SSES was conducted, which included manhole and pipe inspections, smoke testing, acoustic tests, and CCTV inspections. The objective of Part II was to locate, quantify, and evaluate defective infrastructure and the I/I that is entering the City’s collection system.

By rehabilitating infrastructure where these I/I sources are identified the infrastructure can be renewed to extend service life. This also reduces I/I that enters the system, which has the benefit of reduced transport (pumping cost) and treatment cost. Additionally, it has benefit of freeing capacity for development that is limited in the collection system and treatment system.

TREKK conducted manhole and pipe inspections, smoke testing, acoustic tests, and CCTV inspections that aimed to locate, quantify, and evaluate defective infrastructure and I/I entering the City's collection system. Approximately 209,000 linear feet of sanitary sewers in the Town Branch and McKisic sub-basins were inspected, in addition to a total of 793 manholes and 56 cleanouts that were inspected and GPS surveyed.

TREKK evaluated the inspection results and developed recommendations for sewer repairs to reduce I/I and enhance performance. More details about these improvements can be found on TREKK's online dashboard, as shown in Figure ES-2.

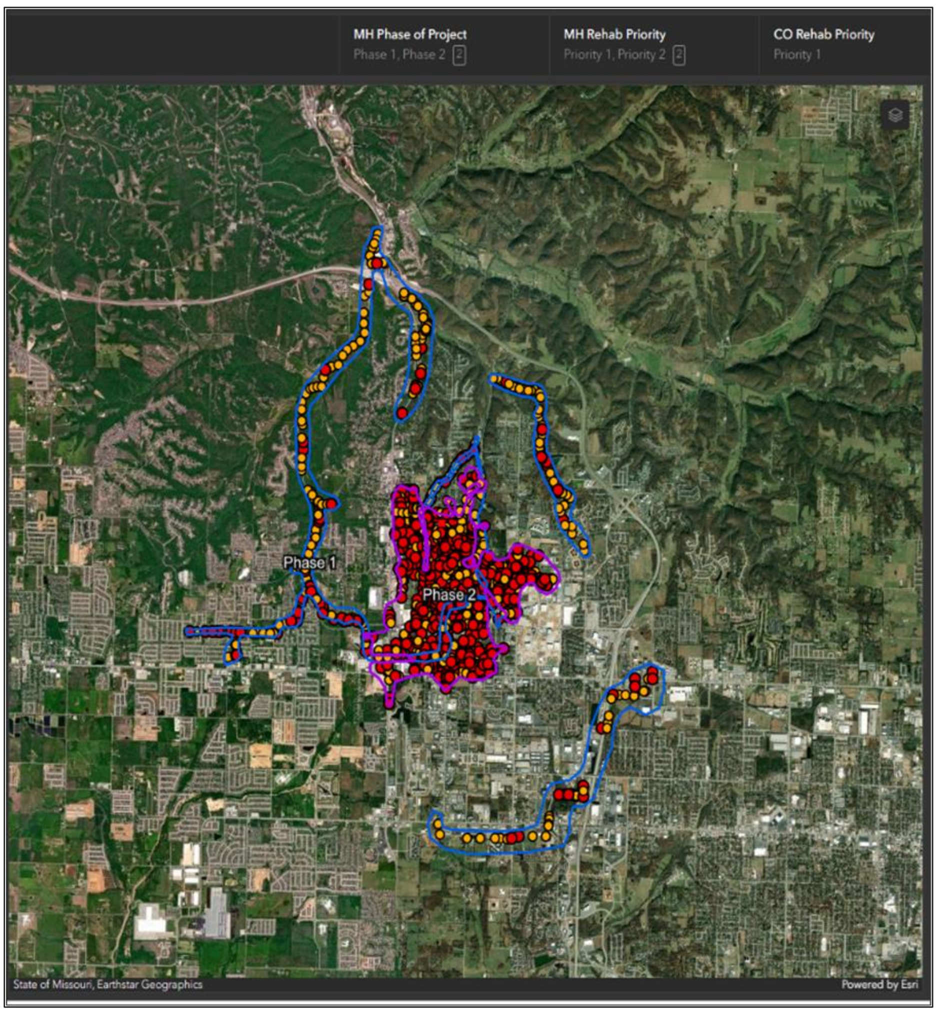



**Figure ES-2. TREKK's SSES Online Dashboard.**

All rehab recommendations were prioritized as 1, 2, or 3. Priority level 1 is defined as "Rehabilitate immediately", priority level 2 is defined as "Needs rehabilitation but no immediate structural concerns -clean, root cut and/or repair and monitor", and priority level 3 is defined as "Needs periodic repair and monitoring. Continue to gauge deterioration." Where recommended capacity improvements to accommodate ultimate buildout overlapped with rehabilitation recommendations, the following exclusions were made from TREKK's recommendations:

- If the recommended capacity improvement was estimated to be needed in 2025, priority 1 and 2 rehab recommendations were excluded.
- If the recommended capacity improvement was estimated to be needed in 2030, only priority 2 rehab recommendations were excluded.
- If the recommended capacity improvement was estimated to be needed after 2030, no rehab recommendations were excluded.

Figure ES-3 shows where TREKK's pipe rehabilitation recommendations overlap with Olsson's capacity improvements and Figure ES-4 shows where TREKK's manhole rehabilitation recommendations overlap with Olsson's capacity improvements. The costs associated with the SSES Part II recommendations, along with the manhole recommendations from SSES Part I (completed in 2019) are summarized in Table ES-1. Costs associated *With Listed Exclusions* have the priority exclusions listed above removed from TREKK's recommendations and the costs associated with *No Capacity Improvement Overlap* exclude any rehabilitation recommendation where there is also a recommended capacity improvement, regardless of estimated horizon.

**Table ES-1.Summary of TREKK's Rehabilitation Recommendations.**

| Study Area Improvements | TREKK's Original Recommendations | With Listed Exclusions | No Capacity Improvement Overlap |
|---|---|---|---|
| Private-Sector I/I Abatement Program[1] | $232,000 | $232,000 | $232,000 |
| Part 1 Manhole Rehabilitation Program (Priority 1 and 2)[2] | $1,670,000 | $1,208,000 | $1,044,000 |
| Part 2 Manhole Rehabilitation Program (Priority 1 and 2)[2] | $674,000 | $674,000 | $674,000 |
| Pipeline Rehabilitation Program (Priority 1 and 2)[1] | $1,544,000 | $1,365,000 | $1,105,000 |
| **Total Cost** | **$4,120,000** | **$3,479,000** | **$3,055,000** |

[1]Costs include 20% contingency.
[2]Costs include 30% contingency.

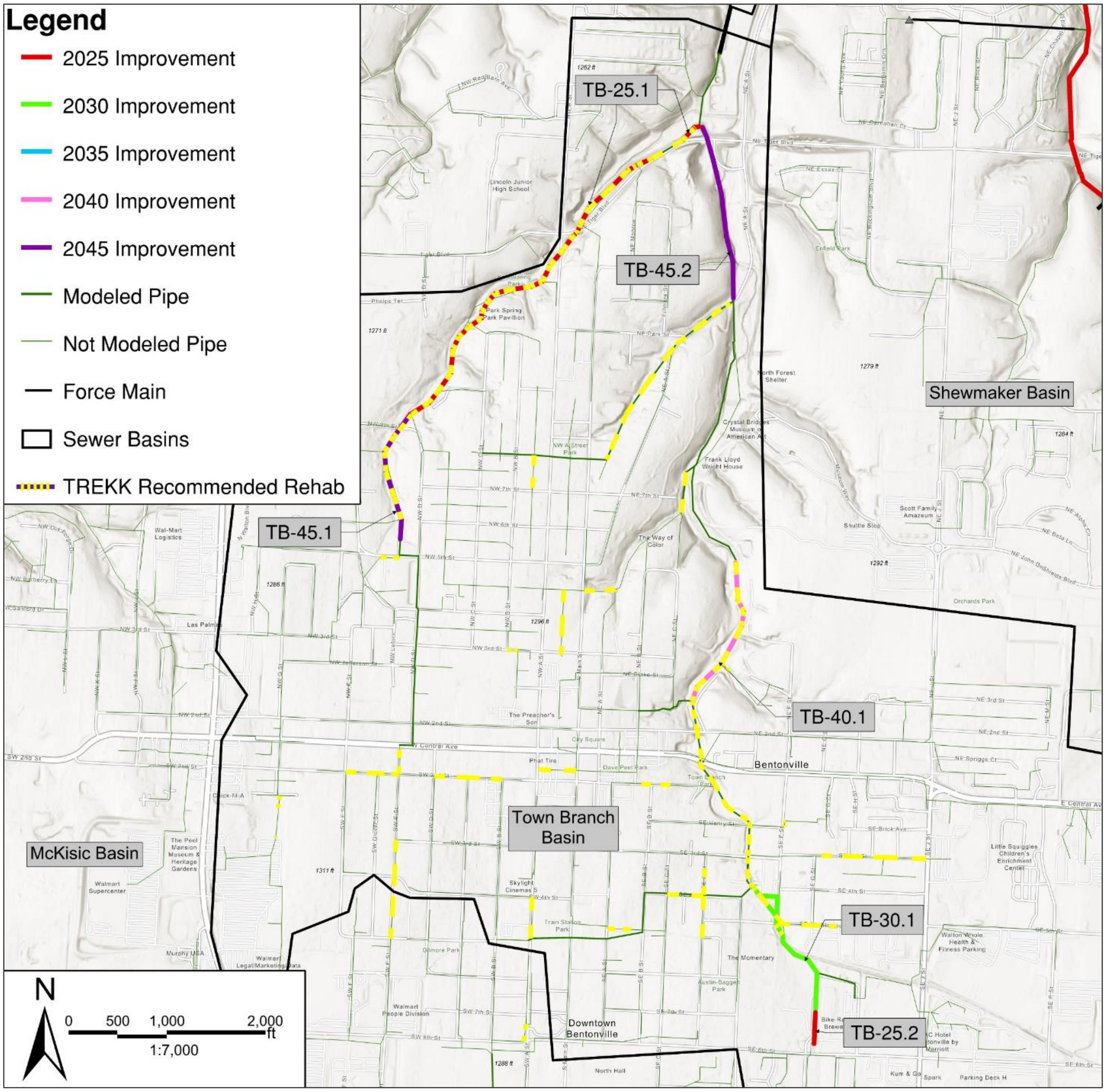



Figure ES-3. TREKK Recommended Pipe Rehab Overlay.

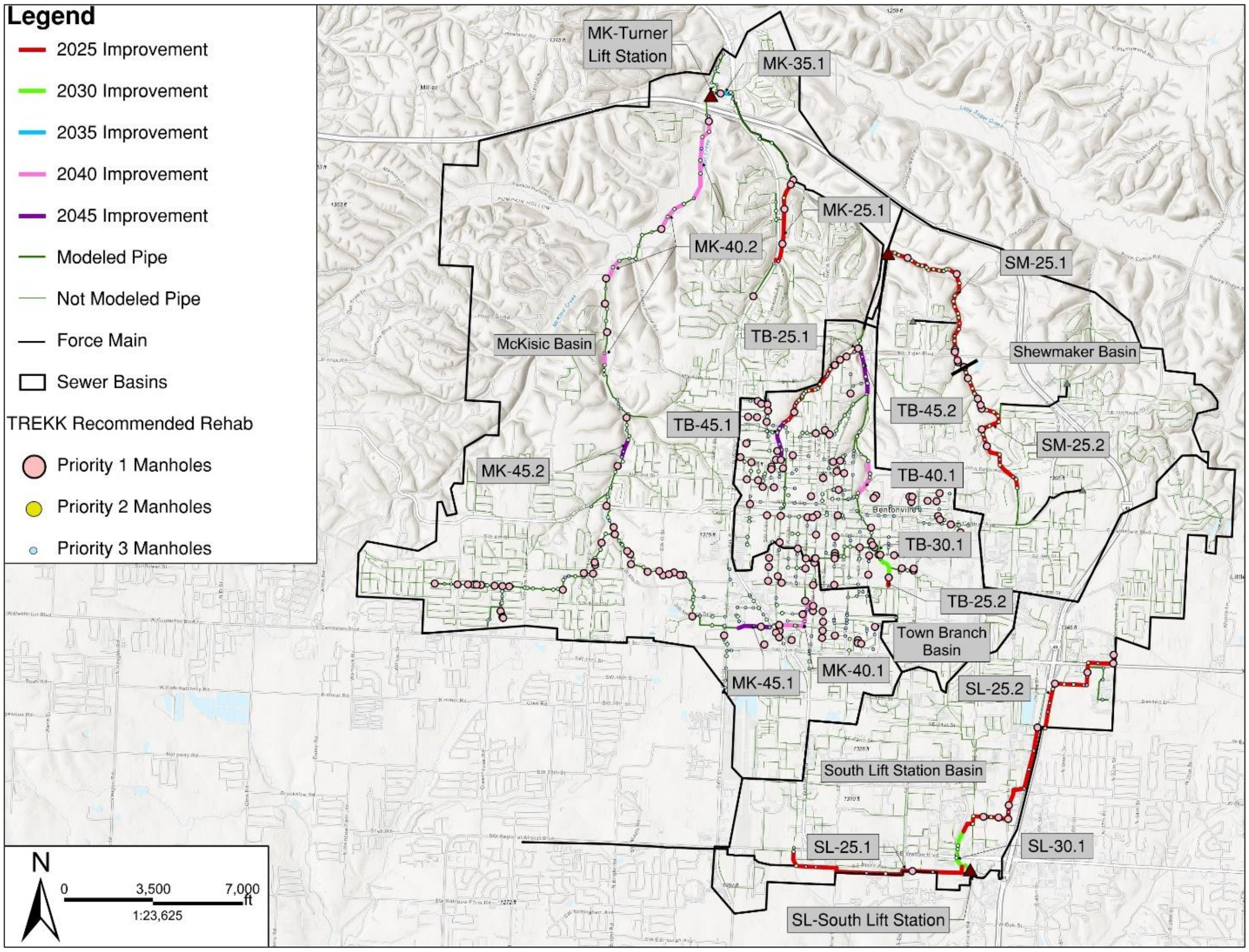


**Figure ES-4. TREKK Recommended Manhole Rehab Overlay.**

## Recommended Improvements

Projects identified for each design horizon include approximately 11.3 miles of conveyance improvements at a total escalated cost of approximately $185.9M. This includes improvements at the McKisic Lift Station with an escalated cost of approximately $547k and improvements at the South Lift Station with an estimated escalated cost ranging from to $44.0M-$55.9M.

A summary of the improvements recommended for each design horizon, grouped by basin, are shown in the following tables and Figure ES-5. *Total Project Cost* is calculated using today's dollar and the *Escalated Total Project Cost* is escalated at 5% a year to the project's estimated design horizon. Projects estimated for the 2025 horizon are escalated at 5% a year to the year 2028.

**Table ES-2. 2025 Recommended Projects.**

| Basin | Total Length (Feet) | Total Project Cost | Escalated Total Project Cost |
|---|---|---|---|
| **McKisic Basin**[1] | 3,167 | $3,799,000 | $4,413,000 |
| **South Lift Station**[2] | 19,549 | $61,608,000 | $71,320,000 |
| **Town Branch Basin** | 4,336 | $6,101,000 | $7,064,000 |
| **Shewmaker Basin** | 13,364 | $17,493,000 | $20,251,000 |
| **2025 Total** | **40,416** | **$89,001,000** | **$103,048,000** |

[1]Includes the MK-Turner Lift Station project costs.
[2]Includes the SL-South Lift Station Option 2, Part 1 project costs.

**Table ES-3. 2030 Recommended Projects.**

| Basin | Total Length (Feet) | Total Project Cost | Escalated Total Project Cost |
|---|---|---|---|
| **South Lift Station** | 1,872 | $3,528,000 | $4,503,000 |
| **Town Branch Basin** | 2,043 | $3,429,000 | $4,377,000 |
| **2030 Total** | **3,915** | **$6,957,000** | **$8,880,000** |

**Table ES-4. 2035 Recommended Projects.**

| Basin | Total Length (Feet) | Total Project Cost | Escalated Total Project Cost |
|---|---|---|---|
| **McKisic Basin** | 462 | $1,150,000 | $1,874,000 |
| **South Lift Station**[1] | - | $5,638,000 | $9,184,000 |
| **2035 Total** | **462** | **$6,788,000** | **$11,058,000** |

[1]Includes the SL-South Lift Station Option 2, Part 2 project costs.

**Table ES-5. 2040 Recommended Projects.**

| Basin | Total Length (Feet) | Total Project Cost | Escalated Total Project Cost |
|---|---|---|---|
| **McKisic Basin** | 8,393 | $12,046,000 | $25,044,000 |
| **Town Branch Basin** | 1,595 | $1,990,000 | $4,138,000 |
| **2040 Total** | **9,988** | **$14,036,000** | **$29,182,000** |

**Table ES-6. 2045 Recommended Projects.**

| Basin | Total Length (Feet) | Total Project Cost | Escalated Total Project Cost |
|---|---|---|---|
| **McKisic Basin** | 1,954 | $7,713,000 | $20,466,000 |
| **Town Branch Basin** | 3,158 | $4,987,000 | $13,233,000 |
| **2045 Total** | **5,112** | **$12,700,000** | **$33,699,000** |

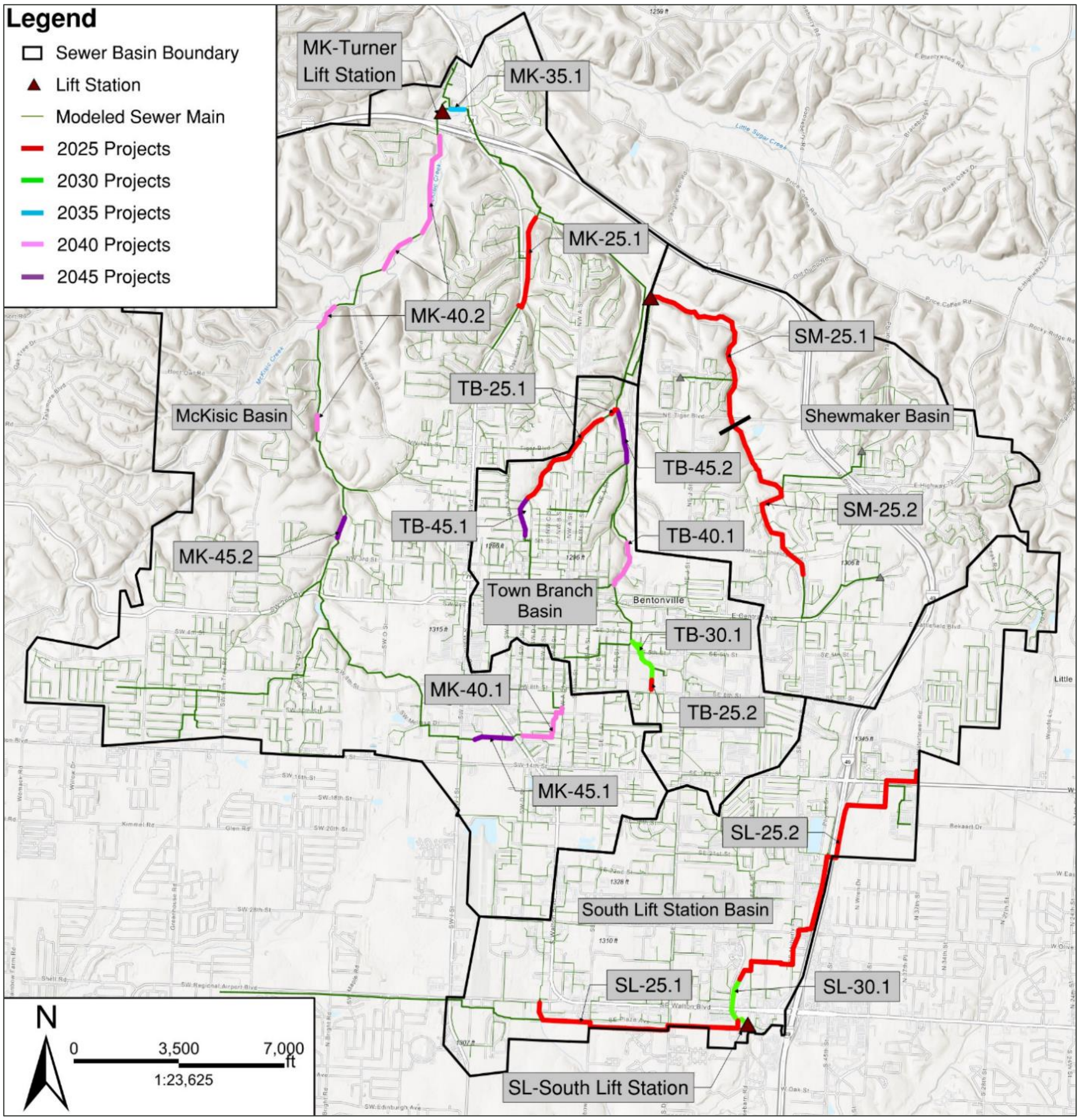



**Figure ES-5. Recommended Capacity Improvements.**

# 1. INTRODUCTION

Bentonville Water Utilities (BWU) is dedicated to delivering high-quality sewer service to meet both current and future demands. To support this commitment, the City of Bentonville, Arkansas, engaged Olsson to evaluate its sanitary sewer collection system and develop a peak flow management program. The program's objective is to safeguard and support BWU's effective management of its wastewater collection system over the next 30 years.

## 1.1 Background

This document represents Part II of BWU's Sewer System Analysis and Peak Flow Management Program. Part I, completed between 2020 and 2022, is documented in the *Bentonville Baseline Sanitary Sewer Capacity Study – Part I* (revised March 2025). Also, Part I included a flow and rainfall monitoring program that captured flow and rainfall data over 284 days across the study area, shown in Figure 1.

The study area, shown in Figure 1, includes the McKisic, Shewmaker, and Town Branch basins that drain to the Water Resource Recovery Facility (WRRF), and the South Lift Station basin that drains to the Northwest Arkansas Conversation Authority's (NACA) Wastewater Treatment Plant (WWTP). Olsson analyzed the collected data, developed a hydraulic model, and assessed the baseline capacity of the sewer collection system. Part II builds on these findings.

## 1.2 Purpose

The purpose of Part II is to formulate a comprehensive peak flow management program for the study area that aligns with the rapid growth trajectory of Bentonville. The program evaluates three strategies to accommodate this growth:

1. Reduction of Rainfall-Derived Inflow and Infiltration (RDII) within the high priority basins identified in Part I,
2. Increase in System Capacity, and
3. Enhancement of System Storage.

Multiple scenarios for intermediate and ultimate build-out horizons were analyzed to evaluate the available capacity. Recommended improvements are presented as capital improvement plan (CIP) projects, including schematic maps, cost estimates, and flow rate triggers that indicate when the improvements should be implemented.

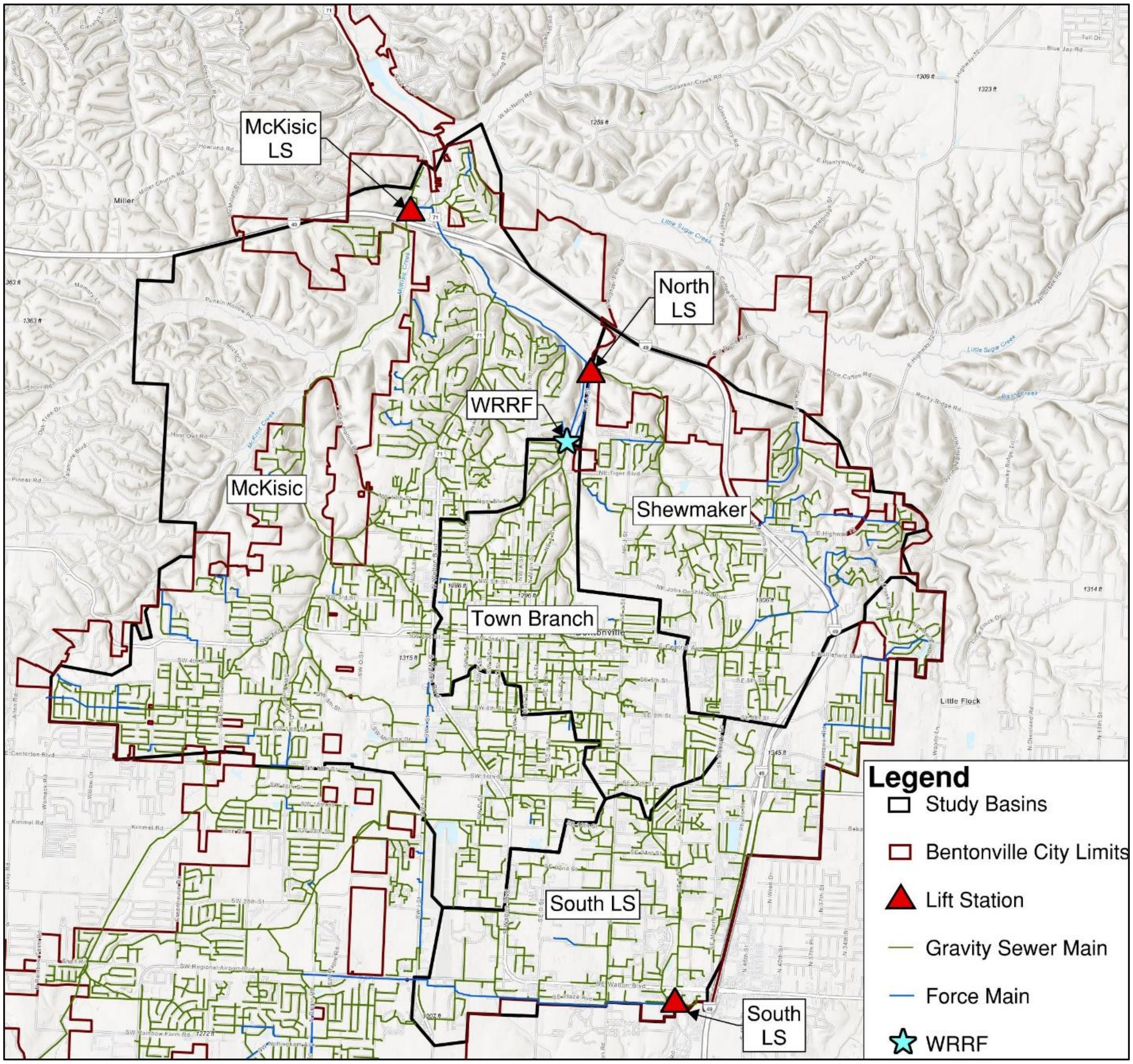



**Figure 1. Collection System Study Area.**

## 1.3 Scope and Need

In collaboration with subconsultant TREKK, who conducted flow and rainfall monitoring over the study area, Olsson prepared this report based on the following evaluation considerations:

### 1.3.1 Establishment of Planning Horizons and Objectives

Olsson reviewed relevant planning documents to project population totals and wastewater generation rates for the ultimate buildout horizon based on the FLUM, assumed to occur by 2050. Ultimate system sizing was determined by preventing any modeled pipes from surcharging using the hydraulic model with the 5-yr Design Storm and the 2050 wastewater generation rates. Sizing was validated using a desktop capacity analysis based on 10-States Standards. Existing flow conditions were established from the flow and rainfall monitoring program and existing population was determined using data from the US Census Bureau. Growth rates were determined at a parcel level between the current and ultimate buildout populations.

Five (5) year intermediate horizons were developed using population values determined by the growth rates and applied to the existing system to determine estimated flows for each horizon. These horizons were used to identify when Wet-Weather Flow (WWF) during the 5-yr design storm exceeded 80% of full pipe capacity. Average Dry-Weather Flow (DWF) values were determined for each recommended improvement to allow for detailed design and construction of the recommended improvement before the pipe is predicted to surcharge.

### 1.3.2 Peak Flow Management Alternatives Evaluation

For each sewer basin, Olsson assessed the impacts of inflow and infiltration (I/I) reduction, equalization storage, and capacity improvements by reviewing the anticipated reduction in surcharge conditions in pipes and sanitary sewer overflows (SSOs). In coordination with TREKK, flow rate and rainfall data were collected from eight key locations to verify existing conditions. This analysis characterized rainfall events, dry weather flow patterns, and quantified RDII for capacity assessment. Recommendations were developed for targeting I/I reduction in both private and public facilities, along with solutions for deficient gravity sewer components and lift stations. Additionally, a Sanitary Service Evaluation Survey (SSES) was conducted in high-priority basins identified in Part I, which is further discussed in Section 7.

# 2. EXISTING COLLECTION SYSTEM

## 2.1 Study Area

The City of Bentonville (City), located in Northwest Arkansas within Benton County, had a population of 59,470 in 2023. The City's wastewater collection system has 68 lift stations, 27 miles of force main, 300 miles of gravity sewer mains, and 7,265 manholes. The system is divided into northern and southern areas, as illustrated in Figure 2. The northern area is served by the City's WRRF, while the southern area is served by the NACA WWTF.

This study, which consists of Parts I and II, focuses primarily on the northern collection system, specifically examining the McKisic, Town Branch, and Shewmaker basins. Additionally, the study includes the South Lift Station basin that flows to NACA WWTF. The current population within this study area is approximately 38,600. The study area includes 50 lift stations, 19.6 miles of force main, 208 miles of gravity sewer mains, and 5,230 manholes. Part I included an evaluation of the three (3) major lift stations - McKisic, North, and South. Part II later incorporated three (3) smaller lift stations: #39 Allencroft, #12 Blueberry, and #9 Rice Road.

## 2.2 Hydraulic Model Extent

The level of detail in a hydraulic model usually depends on the availability and quality of data, including pipe inverts, sizes, and rim elevations. Where information is incomplete, the model extent may be limited to the main trunk lines, key pump stations, and force mains.

For this study, the hydraulic model includes 25 miles of gravity sewer, primarily pipes 12 inches in diameter or larger, the McKisic, North, South, Allencroft, Blueberry, and Rice Road lift stations, and 7.4 miles of force mains, as shown on Figure 3. Additionally, the model incorporates a more detailed representation of the McKisic equalization tanks and lift station to improve accuracy.

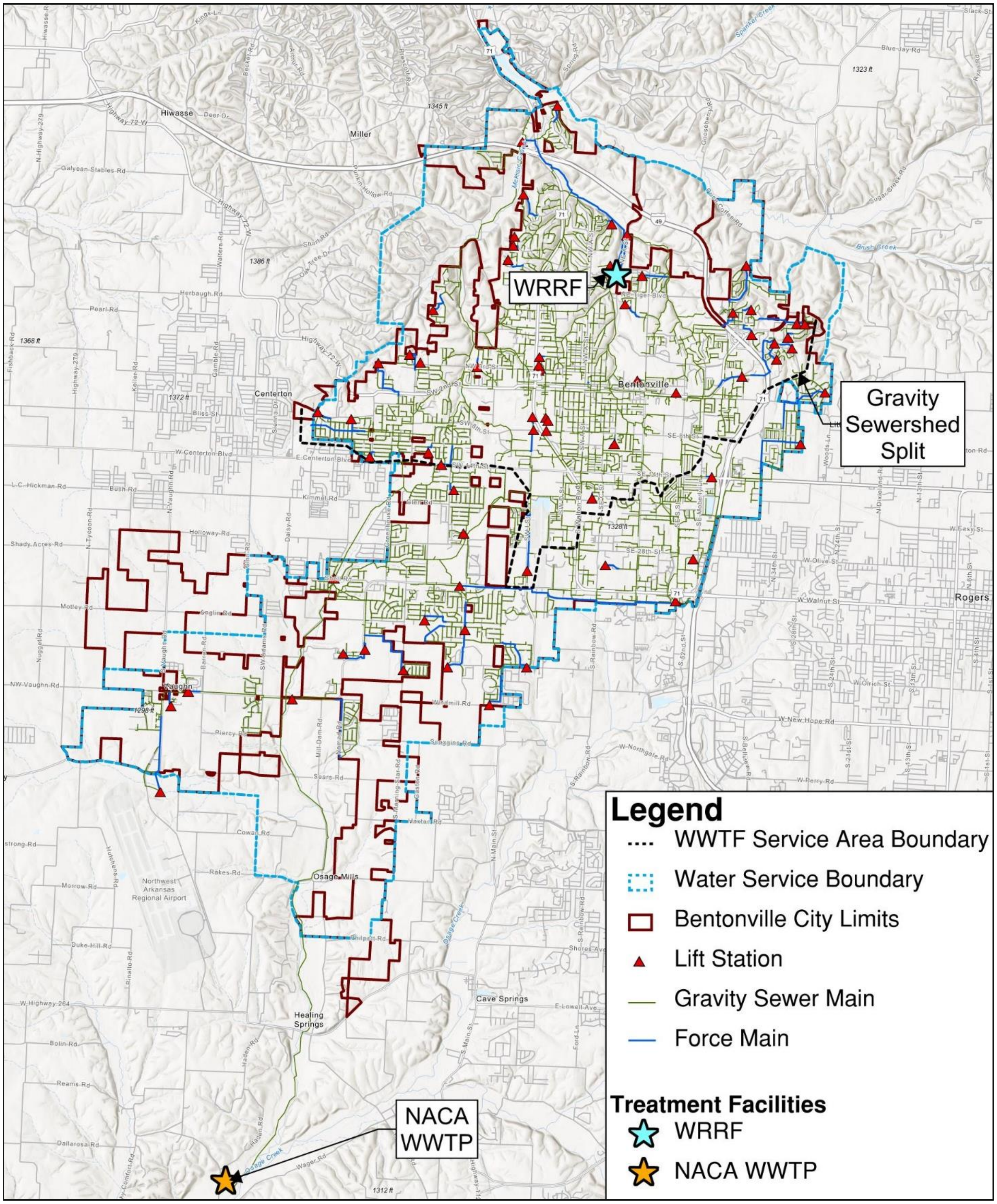



**Figure 2. City of Bentonville, Arkansas Collection System.**

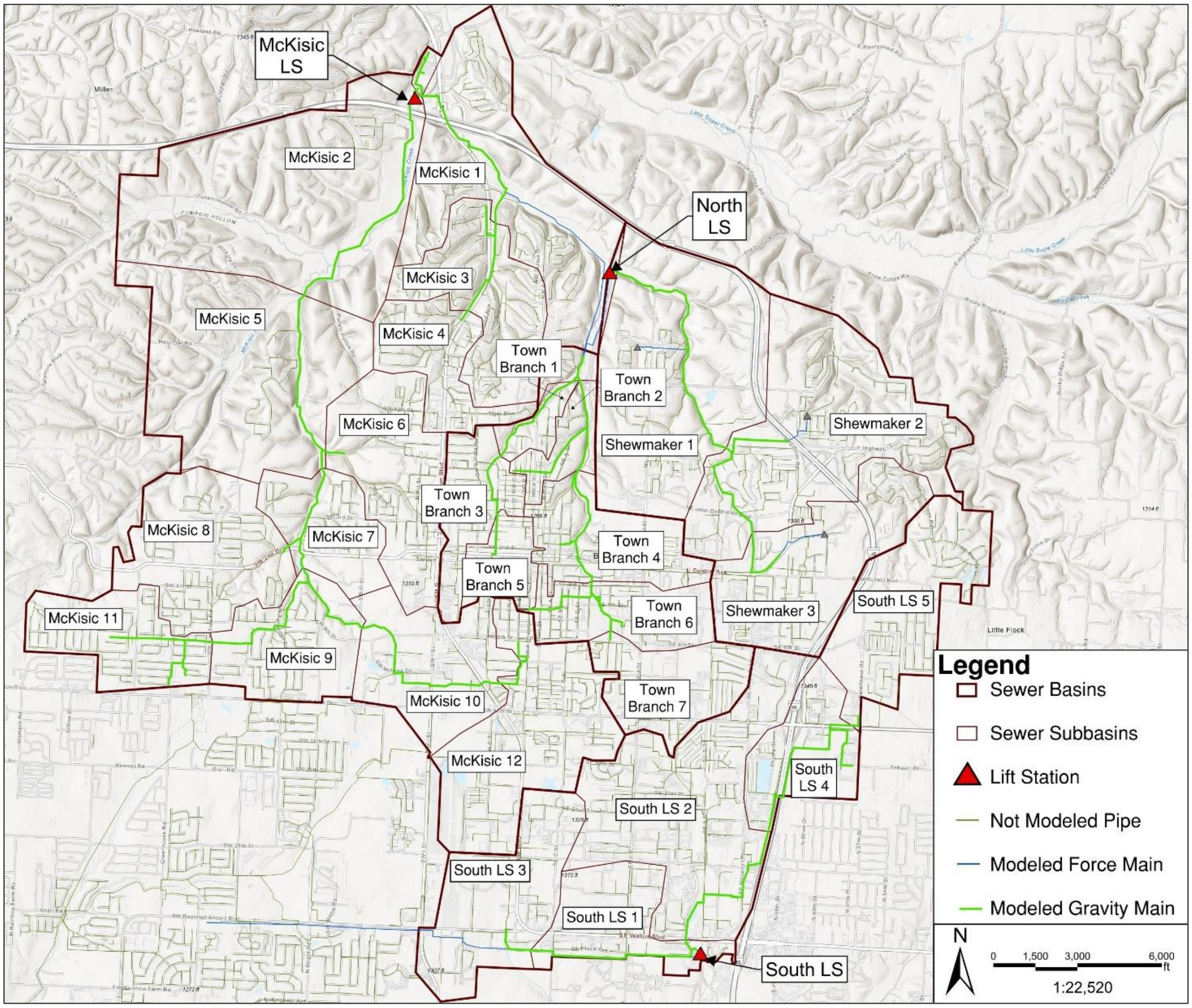



**Figure 3. Hydraulic Model Components.**

# 3. LONG TERM FLOW AND PRECIPITATION MONITORING

Flow and precipitation monitoring are essential for evaluating the performance of sanitary sewer systems. This data provides insights into dry- and wet-weather conditions, seasonal variations, industrial impacts, and I/I trends. Long-term monitoring enhances the accuracy of hydraulic models, supports system capacity planning, and considers regulatory compliance.

## 3.1 Adjusted Flow and Rainfall Monitoring Plan

In Part I, a comprehensive flow monitoring plan deployed twenty-seven (27) temporary flow meters to subdivide the four (4) sewer basins into smaller study areas. This segmentation helped prioritize I/I reduction efforts, particularly in the upper McKissic and South Lift Station basins.

For Part II, BWU transitioned to long-term monitoring and model calibration. Eight (8) flow monitoring units and six (6) rain monitoring units, procured from Blue Siren, Inc., were installed and maintained by TREKK over 16 months. These meters (each capable of recording flow data from two pipes entering the same manhole concurrently) monitored eleven (11) areas, most of which were composed of multiple subbasins. Table 1 and Figure 4 provide the location and subbasins captured by each flow meter. This long-term monitoring approach supports data-driven decision-making for system improvements and regulatory compliance.

**Table 1. Subbasins and Manholes Associated with Part II Flow Meters.**

| Meter ID | Manhole ID | Subbasin IDs |
|---|---|---|
| **FM1** | 236-3842_E | MK1, MK3, MK4 |
| **FM2** | 276-6659_E | MK2, MK5, MK6, MK7, MK8, MK9, MK11 |
| **FM3** *(Relocated April 2024.)* | 402-2478 | MK10, MK12 |
| **FM4** | 278-4437 | SM1, SM2, SM3 |
| **FM5** | 486-5074_S | SLS1, SLS3 |
| **FM6** | 486-5074_NW | SLS2, SLS4, SLS5 |
| **FM7** | 319-3414 | TB1, TB3, TB5 |
| **FM8** | 320-3374 | TB2, TB4 |
| **FM9** | 404-6952 | TB6, TB7 |
| **FM10** *(Not used due to low flows.)* | 236-3842_N | MK1a |
| **FM11** *(Not used due to low flows.)* | 279-6659_W | MK2a |
| **FM12** *(Installed June 2024.)* | 361-506 | TB3, TB5 |

Manholes 236-3842 (FM10), 276-6659 (FM11), and 486-5074 (FM5 and FM6) each monitored two incoming pipes from the same receiving manhole. Manhole 236-3842 contained flow meters FM1 and FM10. FM1 captured flow from subbasins MK1, MK3, and MK4, while FM10 captured flow from a small residential neighborhood. Manhole 276-6659 contained flow meters FM 2 and FM 11. FM2 captured flow from MK2, MK5, MK6, MK7, MK8, MK9, and MK11, while FM11 captured flow from a small residential neighborhood. Neither of the small residential neighborhoods (FM10 and FM11) were able to generate adequate flow depth sufficient enough to be recorded by the flow meter. Due to the relatively small size of these subbasins and the lack of reliable data, they were not analyzed separately.

Initial flow data reviews from the Town Branch (TB) basin indicated suboptimal data from FM7 due to a potential defect near the crossing of Black-Apple Creek. Based on this, it was recommended to relocate meter FM1 to a location upstream of the creek. Site investigations upstream of the creek crossing identified MH361-490 as a suitable alternative due to its accessibility and stable flow conditions. On April 24, 2024, Flow Meter 3 (FM3) from the McKisic basin was relocated from MH402-2478 to MH361-490. FM3 was selected due to the small size of the MK10 and MK12 subbasins, which showed no significant RDII. The new site experienced surcharging shortly after installation, damaging the meter box and creating issues with data collection. This meter was replaced by TREKK and a new site, MH 361-506, was selected and the replacement meter was installed on June 4$^{th}$, 2024. The new meter location, referred to as FM12, resulted in improved data accuracy and reliability, and the data collected at this location was used to recalibrate the area between FM7 location and FM12.

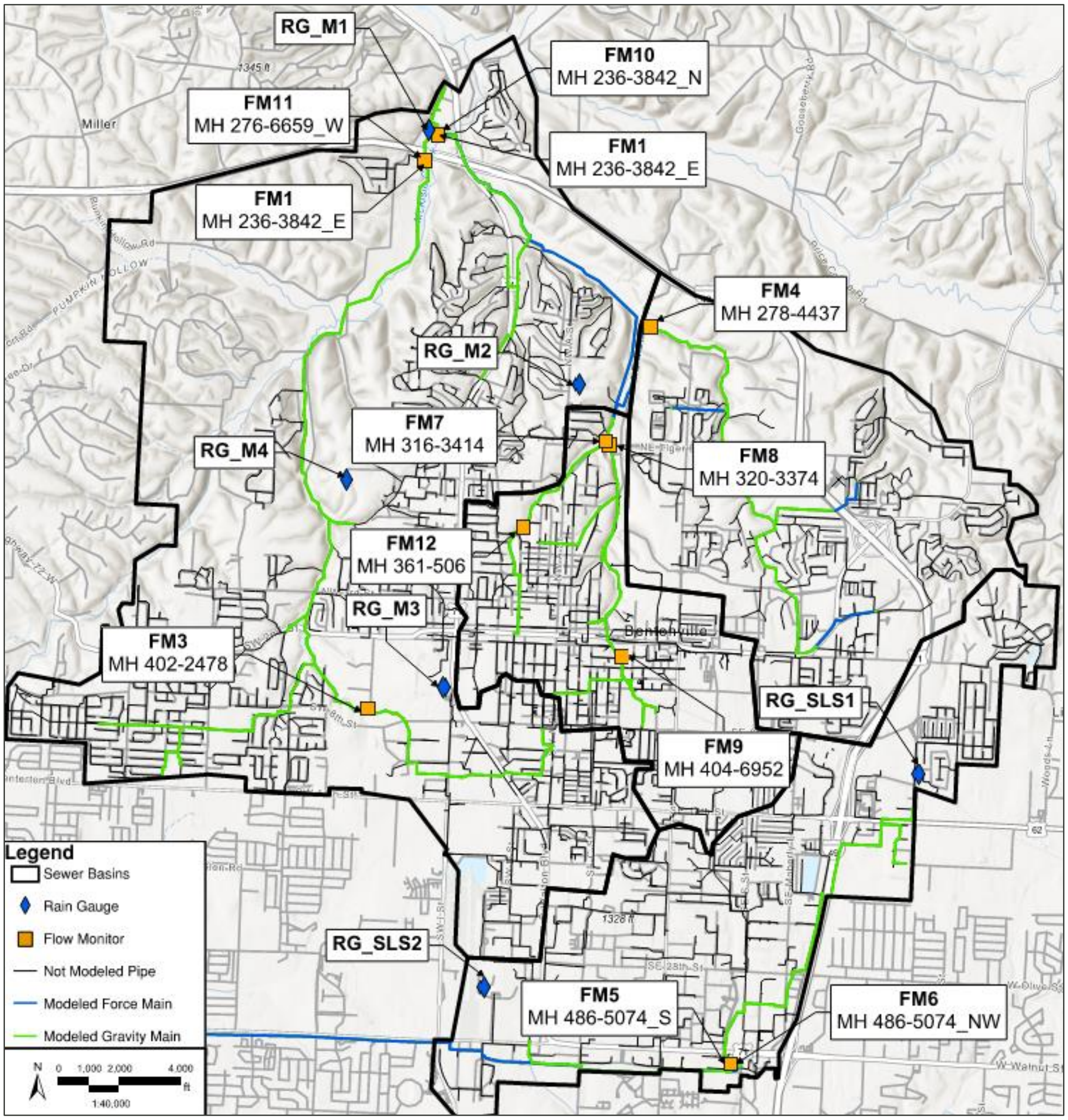



**Figure 4. Flow and Rain Gauge Locations.**

Rain gauges were strategically placed throughout the collection system to capture spatial variations in rainfall within the sewer service area. All rain gauges were tipping-bucket style, capable of recording rainfall at 0.01-inch increments at 1.0-minute intervals. Long-term flow monitoring began on September 6, 2023, and remains ongoing. Data is accessible via TREKK's online platform, WATERSPOUT, which retains records for up to 365 days post-installation, with an additional two years of data access provided for this project. As part of field services, TREKK performs regular maintenance, including battery checks and monthly calibrations, to validate accurate velocity and depth measurements. Table 2 provides additional details on the location of flow monitors and rain gauges used during the data collection period. The sewer model was updated early in the project, incorporating data analysis from September 6, 2023, to November 7, 2023.

**Table 2. Installed Flow Monitors and Rain Gauges.**

| ID | Basin | Location | | | Monitored Subbasins |
|---|---|---|---|---|---|
| | | Manhole ID | Model Pipe ID | Pipe Size (in) | |
| **FM1** | McKisic | 236-3842_E | 236-6811.1 | 18 | MK1, MK1a, MK3, MK4 |
| **FM2** | McKisic | 276-6659_E | 279-6660.1 | 30 | MK2, MK2a, MK5, MK6, MK7, MK8, MK9, MK11 |
| **FM3** | McKisic | 402-2478 | 402-2471.1 | 24 | MK10, MK12 |
| **FM4** | Shewmaker | 278-4437 | 320-3402.1 | 12 | SM1, SM2, SM3 |
| **FM5** | South Lift Station | 486-5074_S | 486-5075.1 | 12 | SLS1, SLS3 |
| **FM6** | South Lift Station | 486-5074_NW | 486-999.1 | 18 | SLS2, SLS4, SLS5 |
| **FM7** | Town Branch | 319-3414 | 319-3436.1 | 12 | TB1, TB3, TB5 |
| **FM8** | Town Branch | 320-3374 | 320-5492.1 | 24 | TB2, TB4 |
| **FM9** | Town Branch | 404-6952 | 404-234.1 | 18 | TB6, TB7 |
| **FM10** | McKisic | 236-3842_N | N/A | N/A | Not Used |
| **FM11** | McKisic | 279-6659_W | N/A | N/A | Not Used |
| **FM12** | Town Branch | 361-506 | 361-503.1 | 12 | TB3, TB5 |
| **M1** | McKisic | 3690 Peach Orchard Rd. | | | MK1, MK1a, MK2, MK2a, MK3 |
| **M2** | McKisic | 2000 NW A St. | | | MK1, MK3, MK4, TB1, TB2, TB4, SM1, SM2 |
| **M3** | McKisic | 700 SW I St. | | | MK6, MK7, MK8, MK9, MK10, MK11, MK12, TB3, TB4, TB5, TB6, TB7 |
| **M4** | McKisic | 9375-9599 NW 12$^{th}$ Dr. | | | MK2, MK3, MK4, MK5, MK6, MK7, MK8, MK11 |
| **SLS1** | South Lift Station | 1007 Water Tower Rd. | | | SLS2, SLS4, SLS5, SM2, SM3, TB6, TB7 |
| **SLS2** | South Lift Station | 501 Airport Rd. | | | SLS1, SLS2, SLS3, MK10, MK12, TB7 |

## 3.2 Rainfall Monitoring Data Collection

One year of rainfall data, September 2023 through August 2024, is shown below in Figure 5. Total rainfall observed for Bentonville during the year was 44.79-inches, with the minimum of 1.26-inches occurring in February 2024 and maximum of 8.19-inches occurring in May 2024. For comparison, the state averages 47.09-inches annually, with a minimum of 2.61-inches in February and a maximum of 5.72-inches in May. This indicates that Bentonville experienced a slightly drier-than-average year, with significant monthly variability. Notably, February's rainfall was less than half it's the state average, while May's total was 40% higher than the typical statewide amount, as shown in Figure 5.

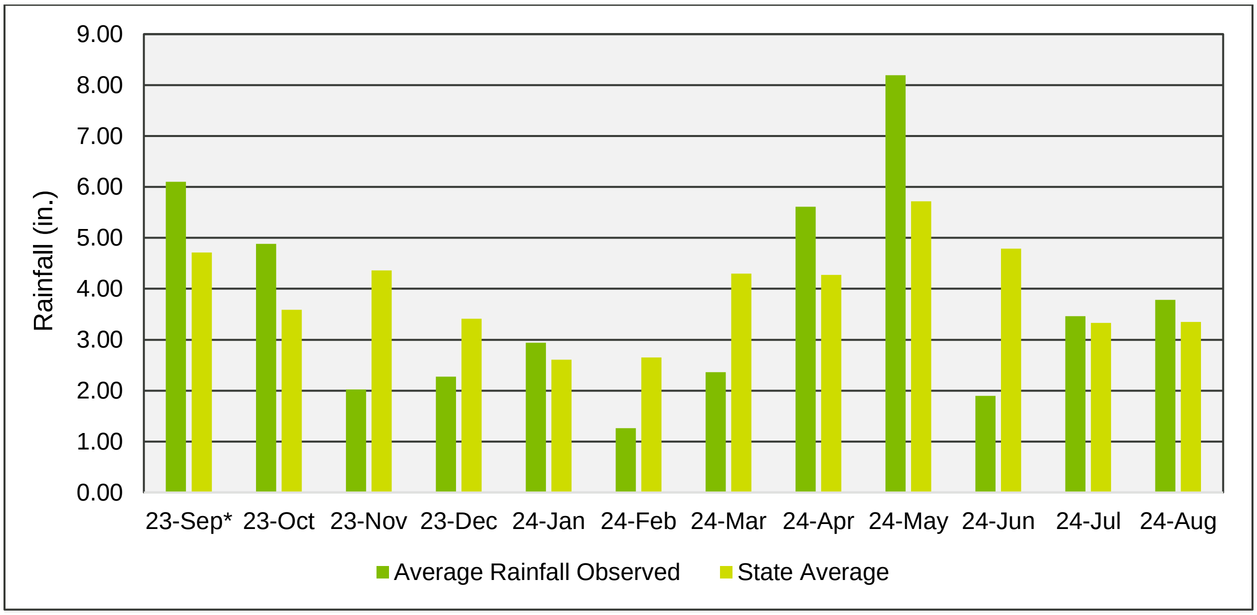



**Figure 5. Monthly Rainfall Summary.**

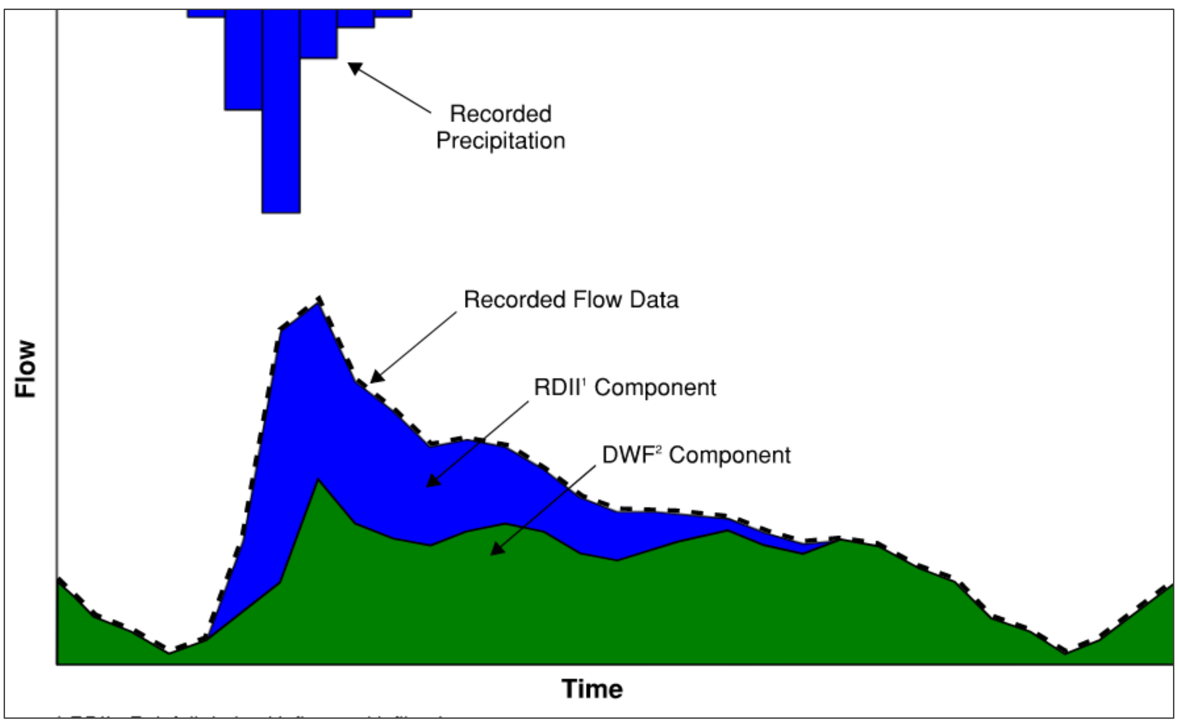
# 4. RAINFALL-DERIVED INFLOW AND INFILTRATION ANALYSIS

Following the completion of the flow and rainfall monitoring period, Olsson conducted a comprehensive analysis to quantify RDII entering the collection system during wet weather events and to update the existing hydraulic model.

## 4.1 Analysis Approach

Flow and rainfall data were collected from September 6, 2023, to November 7, 2023. Extended monitoring periods (60 days or more) are essential for capturing a representative sample of typical dry weather flows and the RDII response in the collection system. To analyze this data, Olsson employed the United States Environmental Protection Agency (EPA) Sanitary Sewer Overflow Analysis and Planning (SSOAP) Toolbox software.

The SSOAP Toolbox was utilized to develop typical DWF for the system and to assess how the system responds to wet weather events by separating recorded flow data into its DWF and RDII components. Figure 6 illustrates how recorded flow data is dissected into these components.




[1] *RDII – Rainfall derived inflow and infiltration;* [2] *DWF – Dry weather flow*

**Figure 6. Components of Wastewater Flow During Wet Weather.**

# 4.2 SSOAP Toolbox Overview

The SSOAP Toolbox, shown in Figure 7, aids users in establishing typical dry weather flow patterns and utilizing the Real Time Kinematic (RTK) method for RDII prediction to generate RDII hydrographs for each area of interest. A screenshot of the primary SSOAP Toolbox interface is shown below for context. The RTK method, detailed in the EPA publication *Review of Sewer Design Criteria and RDII Prediction Methods (EPA/600/R-08/010)*, produces values that define flow responses to various wet weather events. Olsson matches these values with the system's actual responses to wet-weather conditions, aligning them with the hydraulic model's responses to similar events of similar duration and intensity.

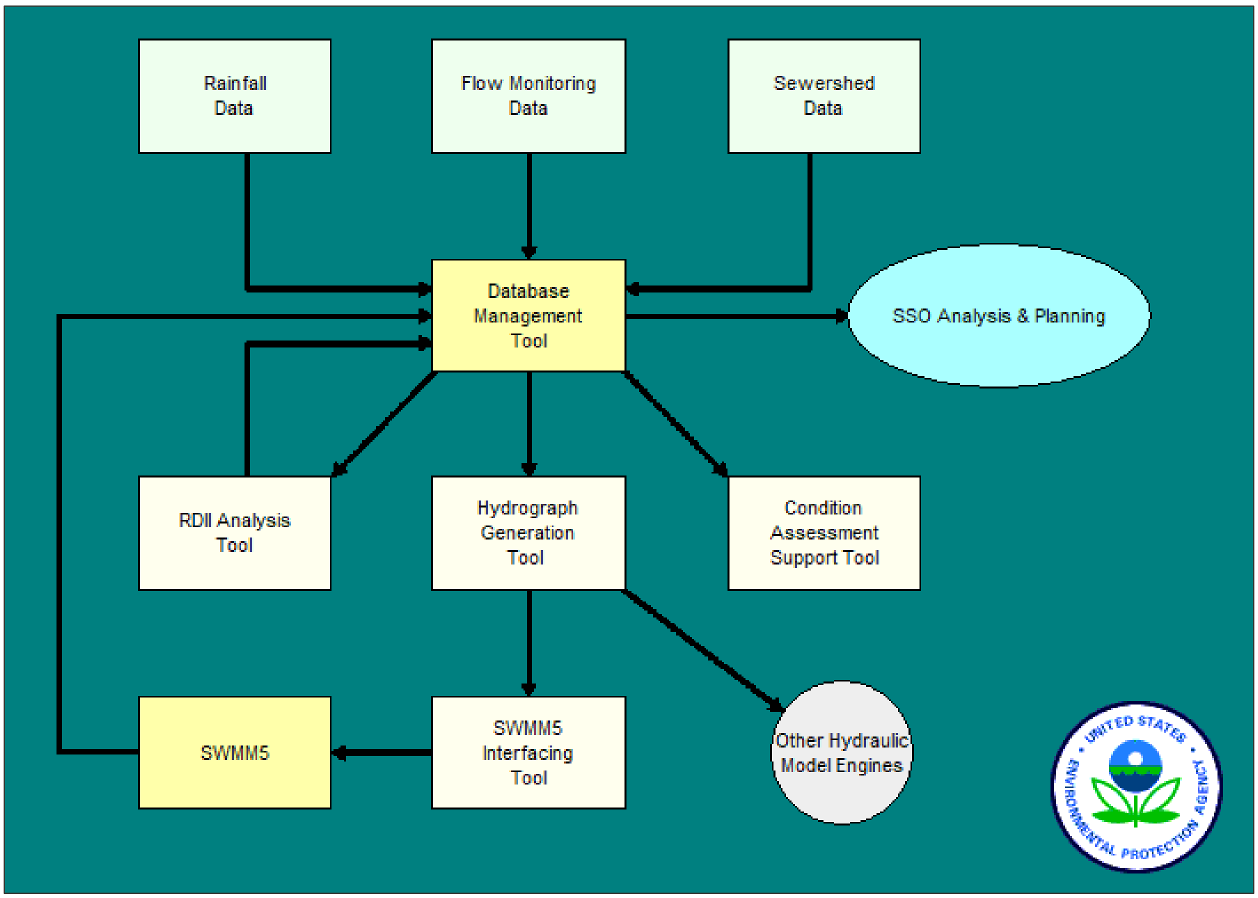



**Figure 7. SSOAP Toolbox User Interface.**

The following steps were performed for each of the nine (9) subbasins analyzed in this project. A detailed description of the SSOAP Toolbox is provided in the EPA report *Computer Tools for Sanitary Sewer System Capacity Analysis and Planning (EPA/600/R-07/11).*

1. Isolate flow data by subtracting the upstream data from the downstream meter data.
2. Enter rainfall, observed flow, and sewer service area data into the SSOAP Toolbox.
3. Evaluate the data with the RDII Analysis Tool, and perform the following tasks:
    a. Determine number of DWF days.
    b. Review DWF hydrographs for weekdays and weekends.
    c. Identify WWF events for RDII analysis.
    d. Develop a simulated RDII hydrograph through decomposition of the observed RDII hydrograph for each wet-weather event, resulting in 'R,' 'T,' and 'K' values representing the subbasin's RDII response.
4. Export the DWF patterns and RTK pattern values for incorporation into the hydraulic model.

## 4.3 Dry Weather Flow Determination

The SSOAP's DWF Analysis Tool features an Automatic DWF Determination Function that identifies dry weather days based on user-defined parameters. For this project, the parameters included:

- No missing data
- Cumulative rainfall limits:
    - 0.25" for the current day
    - 0.5" for the past two (2) days
    - 1.0" for the past three (3) days
    - 2.0" for the past four (4) days
    - 3.0" for the past five (5) days
    - 4.0" for the past six (6) days
    - 5.0" for the past seven (7) days
- Minimum, maximum, and average flow must remain within one standard deviation of the overall values.

SSOAP differentiates between dry-weather weekdays and weekends, allowing for manual review and adjustments. This distinction is critical, as flow rates can vary significantly based on day type. Each meter's dry-weather weekday and weekend flows are averaged to create a 24-hour DWF diurnal hydrographs for both day types. In commercial areas, flow rates may decrease during the weekends when businesses are closed. In residential areas, flow rates may increase during the weekends when residents who work away from home during the week are home. An example diurnal curve from the SSOAP Toolbox is shown on Figure 8, and the average daily DWF flow observed during the monitoring period is shown in Table 3.

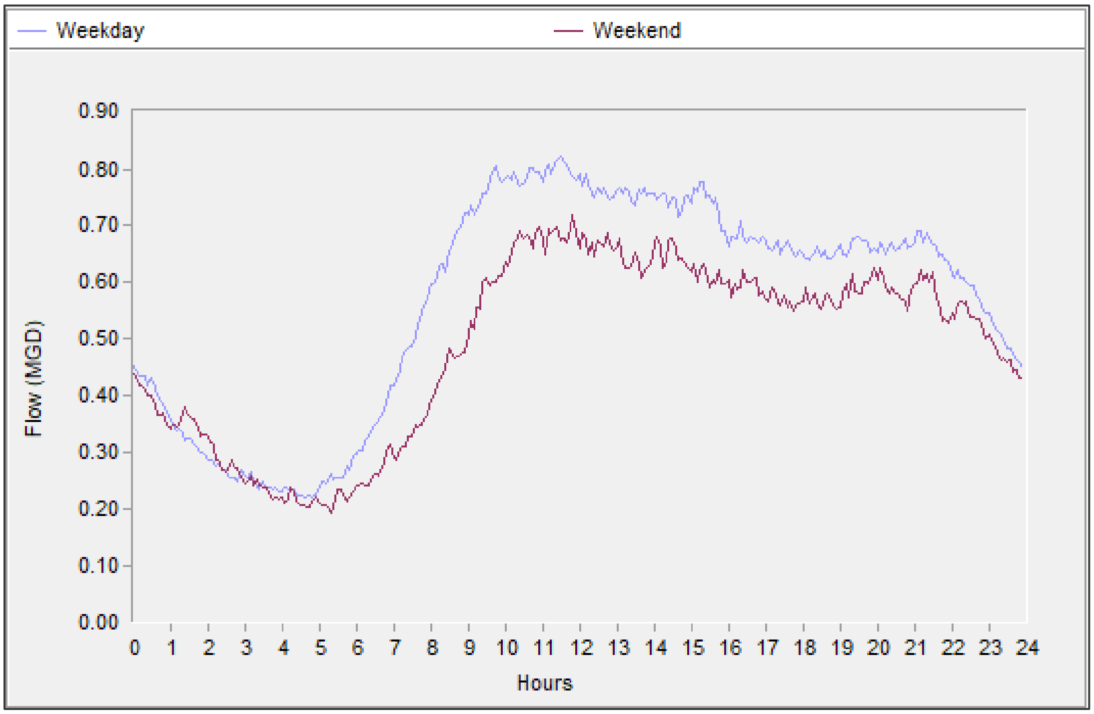



**Figure 8. Sample Dry Weather Flow Output from the SSOAP Toolbox.**

**Table 3. Summary of Dry-Weather Flows.**

| Meter Basin | FM1 | FM2 | FM3 | FM5 | FM5 | FM6 | FM7 | FM8 | FM9 |
|---|---|---|---|---|---|---|---|---|---|
| **Average Flow (MGD)** | 0.20 | 0.48 | 0.35 | 0.45 | 0.27 | 0.57 | 0.45 | 0.55 | 0.42 |
| **Peak Hour Flow (MGD)** | 0.29 | 0.65 | 0.46 | 0.61 | 0.40 | 0.80 | 0.61 | 0.64 | 0.48 |

## 4.4 Wet Weather Flow Determination

The SSOAP's WWF Analysis Tool provides an Automatic RDII Event Identification function that selects RDII events based on user-defined parameters. For this study, events were identified based on a minimum duration of six (6) hours and rainfall volume of at least 0.5-inches. The user had the option to manually add or remove events. An example wet weather event identified in the SSOAP Toolbox is shown in Figure 9.

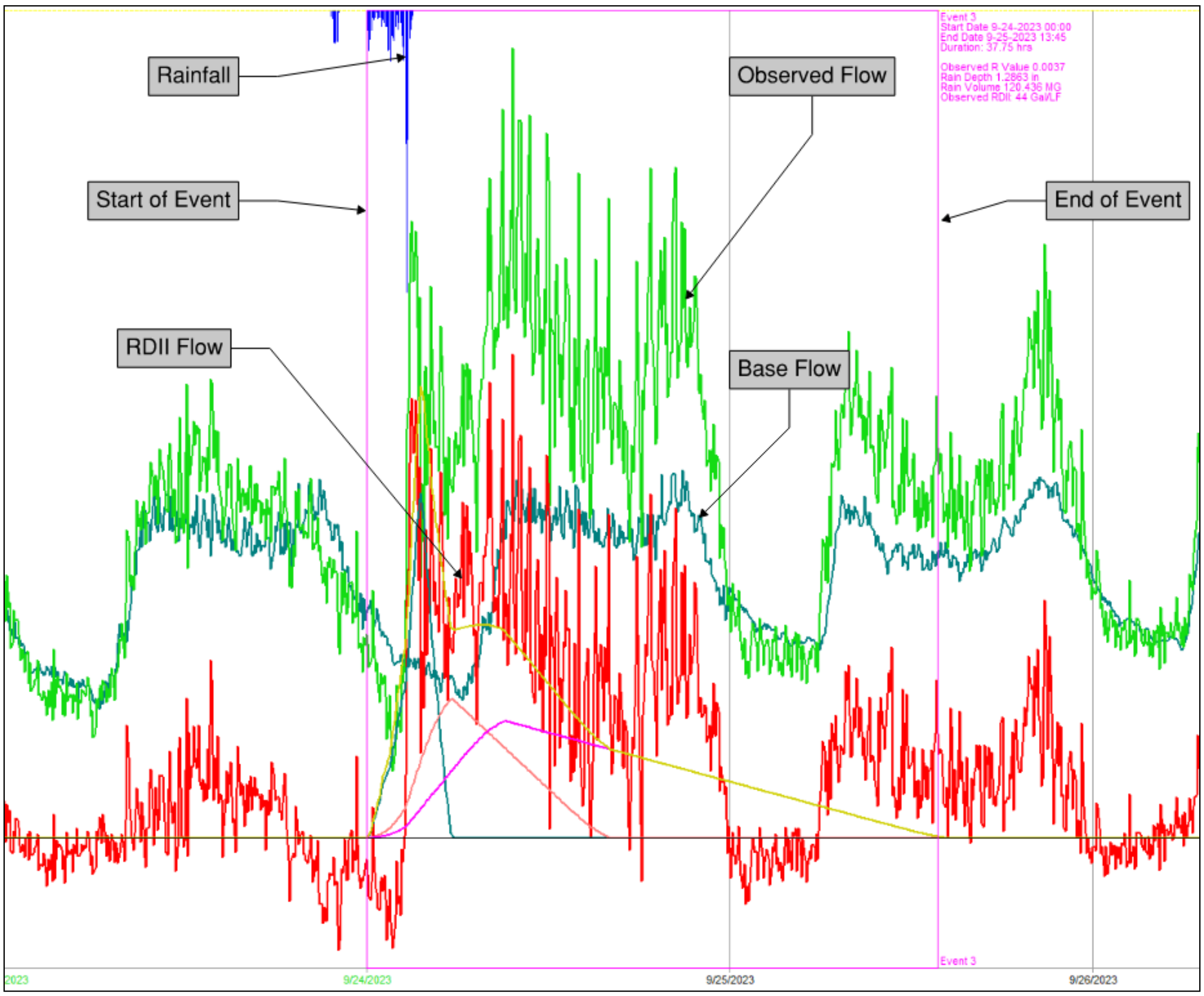



**Figure 9. Sample Wet-Weather Response Analysis from the SSOAP Toolbox.**

## 4.5 RDII Event Hydrograph Decomposition and Unit Hydrograph Curve Fitting

The SSOAP toolbox automatically performs hydrograph decomposition by separating the observed flows into their DWF and RDII components. This information is accessible through the RDII Graph feature. The SSOAP Toolbox was used to further decompose the RDII flows by applying the RTK curve fitting method, developing Synthetic Unit Hydrograph (SUH) parameters from the observed RDII hydrographs. This method fits three (3) triangular unit hydrographs to each rain event's observed RDII hydrograph:

- The first triangle captures rapidly occurring inflow.
- The second triangle encompasses both inflow and infiltration.
- The third triangle addresses infiltration occurring after the rain event ends.

Each hydrograph is represented by three (3) parameters:

- 'R' is the fraction of rainfall entering as RDII during and immediately after a rainfall event
- 'T' is the time for RDII to peak, and
- 'K' is the ratio of time of recession to 'T.'

This trial-and-error fitting process culminates in a simulated RDII hydrograph for each rain event, as illustrated below in Figure 10. The average RTK values derived from these events are then input into the hydraulic model.

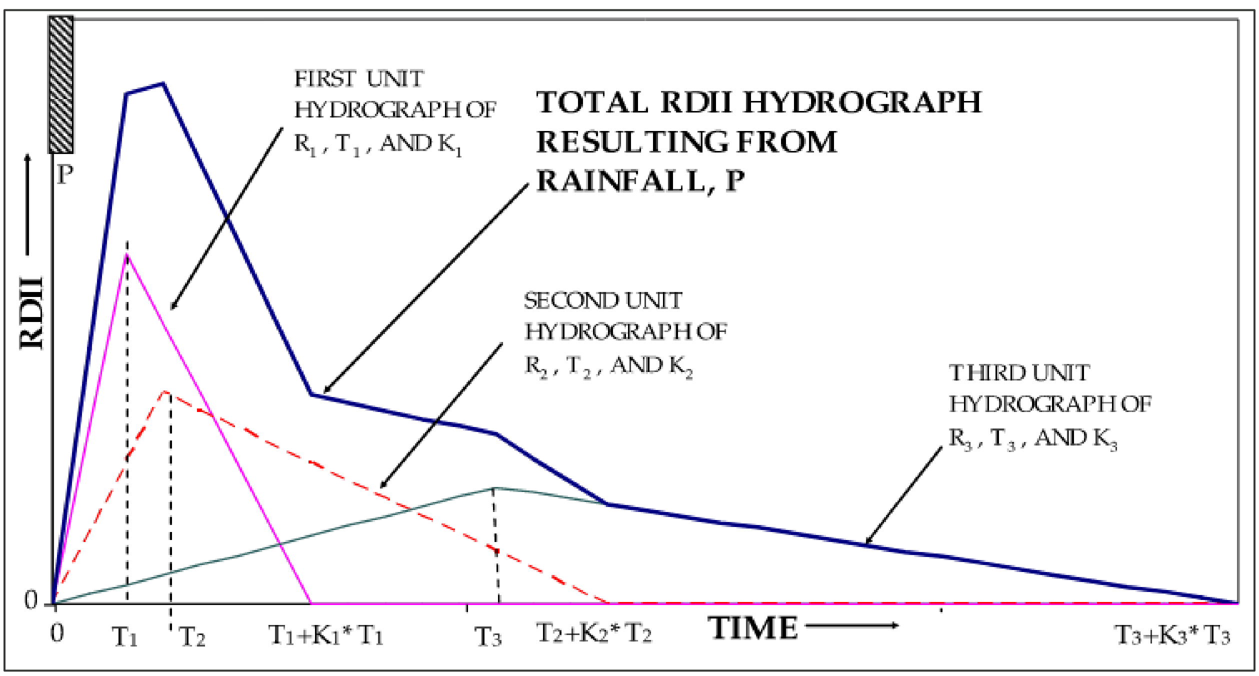



**Figure 10. Components of RTK Hydrograph.**

# 5. LANDUSE, FLOW, AND POPULATION PROJECTIONS

The City of Bentonville recently collaborated with another consulting firm to develop their Future Land Use Map (FLUM). The FLUM was the primary source utilized for estimating future sanitary sewer flows for a fully developed (ultimate buildout) condition in the year 2050.

## 5.1 2050 Future Land Use Plan Update

The updated FLUM provides the foundation for predicting future sanitary sewer flows. It offers a comprehensive framework for land use and development, including zoning regulations, land use categories, and development guidelines, all of which are critical for estimating future flows.

The version used in this report, dated December 18, 2024, includes residential population and commercial population projections for each parcel under the Ultimate Buildout Conditions in 2050. The population value represents the number of residents on a parcel, while the employment value reflects the number of workers. Table 4 below shows the distribution of the residential and commercial populations by subbasin, as projected by the FLUM in 2050.

**Table 4. Subbasin Totals by Users.**

| User | McKisic (MK) | Shewmaker (SM) | South Lift Station (SLS) | Town Branch (TB) | Total |
|---|---|---|---|---|---|
| **Residential Population** | 35,399 | 25,493 | 35,815 | 19,801 | **116,510** |
| **Commercial Population** | 23,742 | 31,265 | 56,156 | 38,490 | **149,654** |

## 5.2 Flow Projections and Model Loading

Various methods are used across the country to project sanitary sewer flows. In Arkansas, the Arkansas Department of Health (ADH) requires designs to conform to the *Recommended Standards for Wastewater Facilities* by the Great Lakes-Upper Mississippi River Board, commonly known as the 10 States Standards. This document simplifies future wastewater loading by using a standard rate of 100 gallons per capita per day (gpcd) for population, in addition to industrial contributions. However, since Bentonville is not only growing in population but also emerging as a tourist destination and business hub, a broader, more tailored approach was preferred.

The updated FLUM provided the total count of residents and employees for each parcel within the study area. In discussions with BWU staff regarding future design approval from ADH, there were concerns about using a loading methodology that suggested a value less than 100 gpcd. Total values for population and employment were each used to determine the necessary improvements needed to the existing system. According to the U.S. Census Bureau, the estimated total population of the study area as of July 2023 was 38,600.

Several approaches were considered for projecting the 2050 flows, based on population, employment, and land use types presented in the FLUM. Applying the 10 States Standards to the existing system would predict a flow of 3.86 million gallons per day (MGD). However, the average flow during the observation period was only 3.28 MGD, or 18% lower than the predicted flow. Olsson also analyzed historical water meter data provided by BWU, which indicated that commercial water use in the study area was approximately half of residential demand. Using this relationship, along with the goal of achieving a total projected design flow that meets or exceeds 10 States Standards value for total system flow, the following flow generation rates were applied: 65 gpcd for the residential population count and 30 gpcd for the commercial population count.

Table 5 provides a comparison of the presented model loading approach and the 10 States Standards, using the 2050 projections. This approach not only exceeds the 10 States Standards but also more accurately reflects the composition of the BWU collection system.

**Table 5. Comparison of the 10 States Standards and Adopted Model Loading Approach.**

| | | Presented Approach | | 10 States Standards | |
|---|---|---|---|---|---|
| | | GPCD | Flow (MGD) | GPCD | Flow (MGD) |
| **Residential Population** | 116,510 | 65 | 7.6 | 100 | 11.7 |
| **Commercial Population** | 149,654 | 30 | 4.5 | - | - |
| | | **Total Flow (MGD)** | **12.1** | **Total Flow (MGD)** | **11.7** |

To verify that the chosen method would satisfy regulatory requirements, Olsson performed a spreadsheet capacity analysis using 10 States Standards approach. This analysis is discussed in more detail in Section 8.5.

Flow triggers were developed to determine when system improvements are needed in order to prevent surcharging and SSOs as population and flows increase. To develop these flow triggers, a growth rate was calculated for each parcel, which was then used to calculate projected population and estimate flows. An exponential growth rate was calculated at the parcel level using the 2023 and ultimate buildout population. For parcels that had no existing population but are projected for development in the FLUM, a linear growth was rate calculated.

# 6. HYDRAULIC MODEL UPDATE AND CALIBRATION

## 6.1 Background

During Part I, a hydraulic model of the sewer system was developed using InfoSewer, a modeling software provided by Autodesk. However, towards the end of Part I, InfoSewer was discontinued, prompting existing users to transition to InfoWorks ICM, a more advanced sewer modeling software also by Autodesk. BWU engaged Autodesk to convert the model from InfoSewer to InfoWorks ICM format. For Part II, Olsson received the updated hydraulic model in InfoWorks ICM format.

During Part II, Olsson updated and calibrated the model utilizing the most recent flow and rainfall data, as detailed in the subsequent sections. The calibrated model was used to evaluate the existing system's capacity under design conditions, which is discussed further in this report.

## 6.2 Modeled Facilities

The hydraulic model, developed during Part I, serves as a skeletal framework that includes 25 miles of gravity sewers, 7.4 miles of force mains, and three (3) primary lift stations—McKisic (and Turner), North, and South. Additionally, three (3) smaller lift stations (#39 Allencroft, #12 Blueberry, and #9 Rice Road) were incorporated later in the process. A breakdown of modeled vs existing gravity and force main lengths by pipe diameter in the study area are presented in Table 6 and Table 7, and details of the modeled pump stations are shown in Table 8.

**Table 6. Gravity Main Lengths by Size.**

| Diameter | Existing (ft) | Modeled (ft) |
|---|---|---|
| 8 | 695,140 | 9,190 |
| 10 | 41,920 | 10,280 |
| 12 | 70,620 | 52,650 |
| 15 | 4,280 | 2,260 |
| 18 | 34,210 | 22,360 |
| 24 | 31,570 | 30,540 |
| 30 | 1,610 | 1,490 |
| 36 | 1,870 | 2,020 |
| Unknown | 17,120 | - |
| **Total (ft)** | **898,340** | **130,790** |
| **Total (miles)** | **170.1** | **24.8** |

**Table 7. Force Main Lengths by Size.**

| Diameter | Existing (ft) | Modeled (ft) |
|---|---|---|
| 2 | 8,800 | - |
| 3 | 4,910 | - |
| 4 | 24,560 | 1,640 |
| 6 | 15,400 | 3,290 |
| 8 | 670 | - |
| 12 | 3,480 | 3,400 |
| 16 | 18,510 | 18,280 |
| 18 | 76 | 76 |
| 24 | 13,250 | 13,220 |
| Unknown | 6,860 | - |
| **Total (ft)** | **96,520** | **39,910** |
| **Total (miles)** | **18.3** | **7.6** |

**Table 8. Modeled Lift Stations.**

| Pump Station Name | Force Main Size (in) | Force Main Length (ft) |
|---|---|---|
| Turner LS | 18 | 76 |
| McKisic LS | 24 | 13,220 |
| North LS | 12 | 3,400 |
| South LS | 16 | 18,280 |
| #39 Allencroft LS (Benjamin Green) | 4 | 1,640 |
| #12 Blueberry LS | 6 | 1,880 |
| #9 Rice Rd LS (Crescent) | 6 | 1,410 |

## 6.3 Model Update

The model encompasses various physical system elements, including gravity mains, force mains, manholes, storage facilities, lift stations, weirs, and sluice gates. A sub catchments layer—comprised of polygon features—was used to allocate dry weather flows to manholes and generate runoff from rainfall events. The sub catchments were refined to exclude areas that do not contribute to the observed wet weather response in the sanitary system.

All modeled lift stations were cross verified against record drawings to confirm accurate representation of pump numbers, pump curves, force main sizes and alignments, and pump controls. The McKisic equalization tanks were modeled in greater detail to reflect the record drawings accurately. Additionally, data for the modeled gravity mains were validated using manhole survey data collected during Part I. Further manhole surveys were conducted in Part II, and the model was updated with this new data.

## 6.4 Dry Weather Flow Modeling

To model DWF in InfoWorks ICM, both the daily flow volume and diurnal pattern are required. In this project, the approach involved isolating DWFs recorded by a flow meter and distributing those flows to each subcatchment in proportion to its area, as discussed in Section 4.3.

## 6.5 Wet Weather Flow Modeling

In Part I, WWF was modeled using the RTK method, which employs a unit hydrograph approach. This method combines runoff from three-unit hydrographs representing fast, medium, and slow responses to a rainfall event. Part II uses the same method, as detailed in Section 4, but is based on data collected in 2023.

## 6.6 Model Calibration

Calibration adjusts model parameters until results - such as flow rate, volume, and depth – align with historical field data within an acceptable range of error. The goal of calibration is to establish and demonstrate a higher level of confidence in the model.

Calibration comprises dry weather and wet weather calibration. Dry weather calibration validates system connectivity and normal operation of pump stations and ancillary structures. Starting with upstream meters, a 7-day dry period between 10/16/2023 and 10/23/2023 was identified to compare model outputs with historical meter data. For those meters where the data quality during the selected calibration period was not of good quality, they were calibrated to an alternative period that had higher quality data. Similarly, model results for wet weather events were compared with the corresponding meter data. The calibration criteria used to determine whether the level of calibration is acceptable is listed in Table 9.

**Table 9. Model Calibration Criteria.**

| Parameter | Dry Weather Calibration Criteria | Wet Weather Calibration Criteria |
|---|---|---|
| Peak Flow | ±0.1 MGD **or** ±10% | +25% to -15% |
| Flow Volume | ±10% | +20% to -10% |
| Shape | Good Match | Good Match |
| Time of Peaks and Troughs | ±0.5 hour | ±0.5 hour |
| Peak Depth (Not surcharged) | ±10% | Maximum of ±10% or ±4-inches |
| Peak Depth (Surcharged) | N/A | +20-inch to - 4-inch |

Deviations from these criteria are expected during calibration. When deviations occurred, the model was adjusted as necessary to reconcile the discrepancy and determine when further calibration was not feasible. Discrepancies may stem from poor data quality or inaccurate infrastructure representation (for example, a pump that is not operating on its pump curve). The results of the DWF and WWF calibration are summarized in the following sections.

### 6.6.1 Dry Weather Calibration

Olsson completed the dry weather calibration for nine (9) metered subbasins during the monitoring period. The objective is to predict DWFs within +/- 10 percent of observed flow. Model results and flow monitoring data were compared on a total volumetric basis and a peak flow basis as shown in Table 10. Adjustments were made for discrepancies exceeding the threshold.

**Table 10. Dry Weather Flow Calibration Volume Comparison.**

| Meter | Volume (MG) | | Difference | |
|---|---|---|---|---|
| | Observed | Modeled | (MG) | (%) |
| **FM1** | 0.18 | 0.17 | 0.01 | 5.1% |
| **FM2** | 0.84 | 0.84 | 0.00 | 0.4% |
| **FM3** | 0.30 | 0.33 | -0.03 | -8.9% |
| **FM4** | 0.46 | 0.46 | 0.01 | 1.2% |
| **FM5** | 0.27 | 0.26 | 0.01 | 4.7% |
| **FM6** | 0.52 | 0.55 | -0.03 | -5.7% |
| **FM7** | 0.43 | 0.43 | 0.00 | -0.3% |
| **FM8** | 0.67 | 0.65 | 0.02 | 3.1% |
| **FM9** | 0.43 | 0.39 | 0.03 | 8.7% |

### 6.6.2 Wet Weather Calibration

Despite the drier conditions in 2023, wet weather responses were analyzed across three rainfall events between September and October. Each meter volume, peak flow, and depth were compared to the corresponding model results and ranked as either "Meets Criteria", "Over Predicting", or "Under Predicting" based on the calibration criteria described in Table 9. A summary of the calibration ranking for each event is shown in Table 11, and a summary of the calibration ranking for all events and meters is shown on Table 12.

**Table 11. Wet Weather Calibration by Rainfall Events.**

| Rainfall Event | Calibration Ranking | Volume | Peak Flow | Max Depth |
|---|---|---|---|---|
| **9/19/2023** | Meets Criteria | 6 | 8 | 9 |
| | Under Predicting | 0 | 0 | 0 |
| | Over Predicting | 3 | 1 | 0 |
| **9/23/2023** | Meets Criteria | 4 | 4 | 9 |
| | Under Predicting | 5 | 5 | 0 |
| | Over Predicting | 0 | 0 | 0 |
| **10/4/2023** | Meets Criteria | 7 | 8 | 9 |
| | Under Predicting | 0 | 0 | 0 |
| | Over Predicting | 2 | 1 | 0 |

**Table 12. Wet Weather Calibration Summary for all Rainfall Events.**

| Calibration Ranking | Volume | Peak Flow | Max Depth |
|---|---|---|---|
| **Meets Criteria** | 17 | 20 | 27 |
| **Under Predicting** | 5 | 5 | 0 |
| **Over Predicting** | 5 | 2 | 0 |

# 7. SANITARY SYSTEM EVALUATION SURVEY RESULTS

## 7.1 Introduction

In Part I of this study, Olsson recommended that the City conduct a Sanitary Sewer Evaluation Study (SSES) to identify potential sources of infiltration and inflow (I/I) within the private service lines and public areas of the collection system. The focus was on seven (7) priority basins with the highest rates of I/I, as shown in Figure 11. Recommendations from Part I are included Section 10.6, however this section focuses on the results of Part II.

In collaboration with TREKK, the Part II (referred to as Phase II in the TREKK Technical Memo) of the SSES was conducted, which included manhole and pipe inspections, smoke testing, acoustic tests, and CCTV inspections. The objective of Part II was to locate, quantify, and evaluate defective infrastructure and the I/I that is entering the City's collection system.

By rehabilitating infrastructure where these I/I sources are identified the infrastructure can be renewed to extend service life. This also reduces I/I that enters the system, which has the benefit of reduced transport (pumping cost) and treatment cost. Additionally, it has benefit of freeing capacity for development that is limited in the collection system.

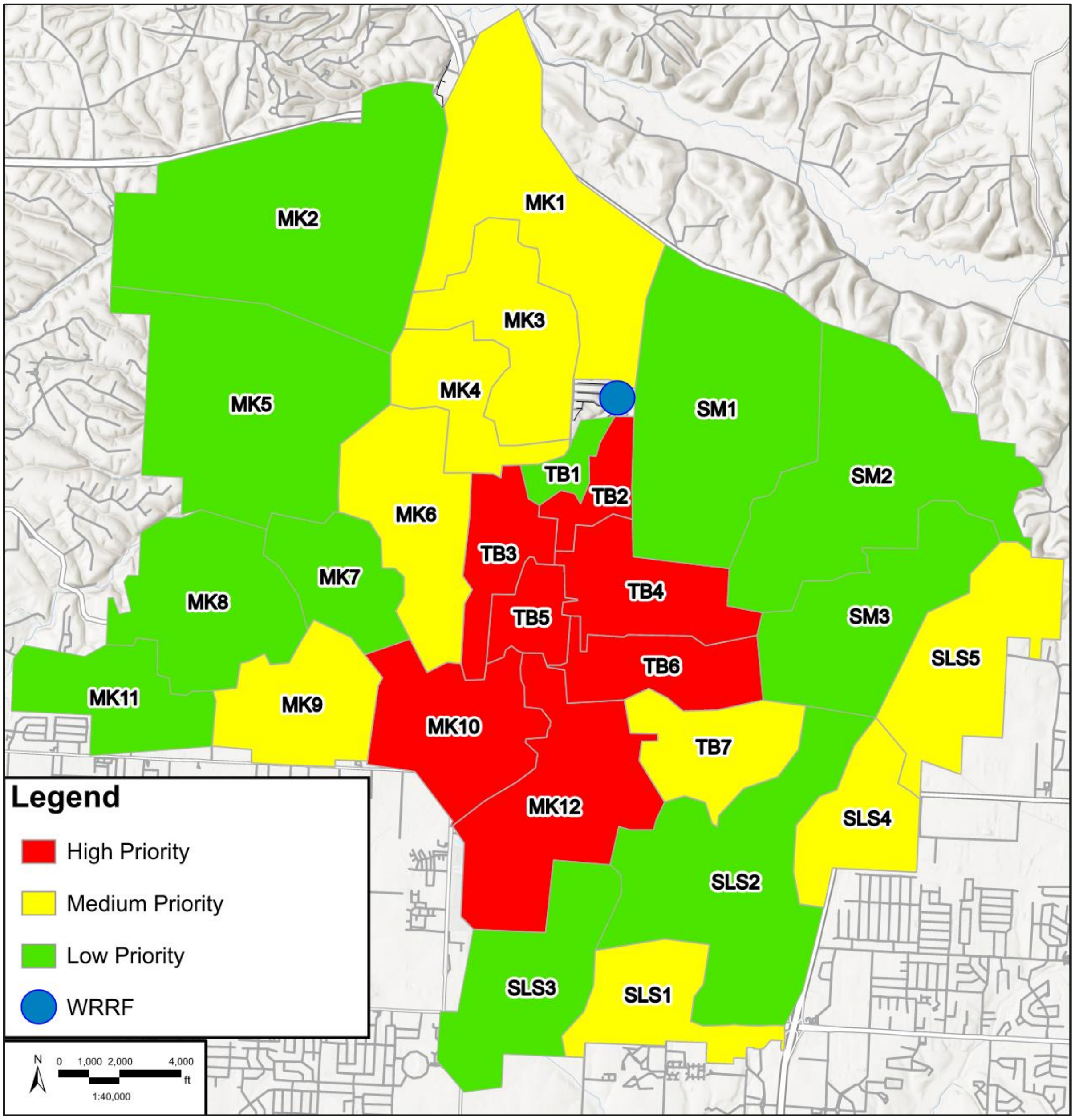



**Figure 11. Part I Prioritized Public I/I Reduction Project Areas.**

# 7.2 Results

A total of 793 manholes and 56 cleanouts were inspected and GPS surveyed and approximately 215,000 linear feet of sanitary sewers in the Town Branch and McKisic sub-basins were inspected. The sources of I/I identified in Part 2 are presented below

The manhole inspections resulted in the identification of a total of 1,033 defects. Additionally, a total of 1,995 visual pipe inspections were performed, revealing 57 potential pipe structural defects and 118 potential seal defects.

Smoke testing of over 209,000 linear feet of pipe uncovered 72 potential public-sector I/I source defects and 168 potential private-sector source defects. The majority of these defects included uncapped or broken cleanouts and defective service laterals. Acoustic sound testing was conducted on a total of 166,100 linear feet of pipe, with 33% of the pipes tested receiving a fair or worse rating. Based on the results of the smoke testing, acoustic sound testing, and visual pipe inspections, 98 pipe segments totaling 21,021 linear feet of sanitary sewer were identified for cleaning and CCTV inspection. TREKK successfully televised 19,886 linear feet of pipe. There were eight (8) pipe segments that could not be fully televised due to roots and blockages. All televised pipe segments, except for two of the eight, were evaluated for potential rehabilitation measures.

The field investigations revealed a total of 722 public-sector defects contributing an estimated 725 GPM of I/I flow, summarized in Table 13 and 168 private-sector I/I defects contributing an estimated 514 GPM of I/I flow, summarized in Table 14. These findings led to recommendations for sewer repairs aimed at reducing I/I and improving system performance. The recommendations were assigned a Priority level of 1, 2, or 3 depending on the severity of the defect. Priority 1 recommendations indicate that rehabilitation is needed immediately. Priority 2 recommendations indicate that rehabilitation is needed but there are no immediate structural concerns. This priority level generally indicated that the defect requires cleaning, root cutting and/or repair and monitoring. Priority 3 recommendations indicate that the pipe or manhole needs periodic repair and monitoring to continue to gauge deterioration.

**Table 13. Summary of Public-Sector Inflow/Infiltration Sources.**

| Inflow/Infiltration Source | Number of Sources | Flow Rate (gpm) |
|---|---|---|
| Manhole | 616 | 240.58 |
| Main Sewer | 98 | 278.12 |
| Fountain Drain | 1 | 15.00 |
| Storm Ditch – Mainline | 5 | 120.30 |
| Storm Ditch – Storm Inlet | 2 | 70.65 |
| **Total Public-Sector I/I:** | **722** | **724.65** |

**Table 14. Summary of Private-Sector Inflow/Infiltration Sources.**

| Inflow/Infiltration Source | Number of Sources | Flow Rate (gpm) |
|---|---|---|
| Area Drain | 3 | 6.55 |
| Downspout | 1 | 13.19 |
| Foundation Drain | 4 | 17.69 |
| Grease Trap | 2 | 2.00 |
| Septic | 2 | 15.00 |
| Service Lateral | 39 | 76.85 |
| Storm Ditch – Service Lateral | 1 | 2.29 |
| Uncapped Cleanout | 116 | 381.17 |
| **Total Private-Sector I/I:** | **168** | **513.74** |

Pipe and manhole recommendations, including the manholes inspected in Part 1, were developed by TREKK and are presented in Section 10.6 and are also viewable in TREKK's provided online dashboard, shown in Figure 12. Additionally, a spreadsheet detailing the rehab recommendations was also provided to BWU for their use.

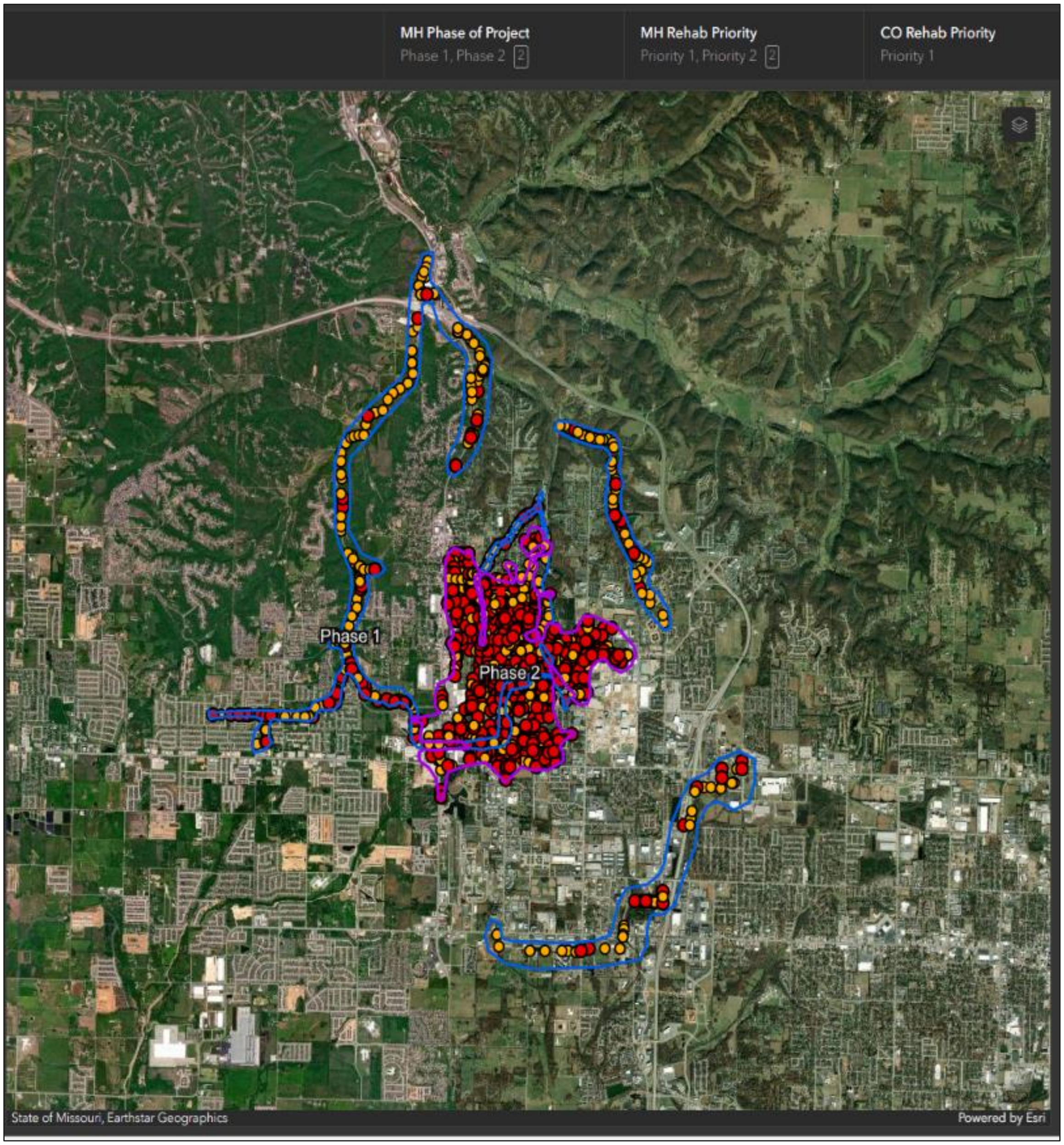



**Figure 12.TREKK's SSES Online Dashboard.**

# 8. PLANNING CRITERIA

This section presents the criteria and methodologies that were used to evaluate the existing collection system, identify facilities that need to be upgraded, and design future improvements.

## 8.1 Introduction

The criteria outlined in this section were developed using industry-accepted standards, including those employed by other utilities and best practices. These criteria are essential for assessing system performance and identifying areas where facilities need to be upgraded.

For each of the evaluation horizons described in Section 1.3.1, improvements were developed using the hydraulic model to alleviate surcharging in pipes throughout the system and prevent predicted SSOs. Where pipe capacity deficiencies were identified, recommended improvements were sized to accommodate projected flows in ultimate buildout.

## 8.2 Flow and Depth Ratios

To identify the risk of surcharge and overflow, pipes are classified based on two key ratios: the ratio of flow depth to pipe size (d/D) and the ratio of peak flow to full pipe capacity (q/Q).

Pipe risk of surcharge or overflow is classified based on the d/D and q/Q ratios as follows:

- < 0.5 is classified as low risk,
- 0.5-0.8 is classified as moderate risk,
- >0.8 is classified as high risk.

## 8.3 Slope and Velocity

During dry weather flow conditions, a minimum velocity of 2 ft/s is recommended to achieve scouring. Velocities less than 2 ft/s may result in buildup at the bottom of the pipes, which could reduce available pipe capacity leading to surcharge and overflow conditions. This would require more frequent maintenance to clear the debris by flushing these pipes.

Pipe velocity is classified as follows:

- < 2 ft/s is classified as low,
- 2-8 ft/s is classified as acceptable,
- > 8 ft/s is classified as high.

This classification helps with identifying potential issues such as sedimentation when the velocity is low or surcharge potential when the velocity, and therefore flow rate, is high. For design purposes, recommended pipe alignment, size, and slope would meet acceptable pipe velocity during peak flows.

## 8.4 Peak Design Flow and Peaking Factors

Dry weather flows have an assigned diurnal pattern that is derived from flow monitoring data. The diurnal pattern has hourly multipliers that represent relative changes in the flow rate over the course of a day and estimate the peak hour flow rate during dry weather conditions.

For design purposes, the wet weather flow calculated by the model during a 5-YR design storm determines the design flow for each pipe. Also, the Ten States Standards were used to calculate a peaking factor based on population served, which was compared with corresponding model results to validate recommended sizing and slopes. Any pipe segments identified as overcapacity in the model or desktop analysis were included in the recommended improvements.

## 8.5 10 States Standards

The Arkansas Department of Health requires all sewer facilities within the state to be designed according to the *Recommended Standards for Wastewater Facilities,* commonly known as the *10 States Standards,* by the Great Lakes-Upper Mississippi River Board of State and Provincial Public Health and Environmental Managers; (2014). These standards have separate requirements for hydraulic capacity of existing collection systems (Section 11.242) and new collection systems (Section 11.243). This section of the report discusses how these standards were applied in this study.

### 8.5.1 Existing Collection System Hydraulic Capacity

Section 11.242 of the 10 States Standards requires existing collection systems hydraulic capacity be analyzed using projections made from actual flow data and that critical data and methodology used shall be included in the report. The standard is quoted below:

*"11.242 Hydraulic Capacity for Wastewater Facilities to Serve Existing Collection Systems*

> *a. Projections shall be made from actual flow data to the extent possible.*
>
> *b. The probable degree of accuracy of data and projections for all critical design flow conditions shall be evaluated. This reliability estimation should include an evaluation of the accuracy of existing data, and an evaluation of the reliability of estimates of flow reduction anticipated due to I/I reduction, or flow increases due to elimination of sewer bypasses and backups or hydraulic restrictions. To achieve a higher degree of accuracy, estimates of I/I reduction shall consider design precipitation events with representative runoff characteristics and groundwater elevations.*
>
> *c. Critical data and methodology used shall be included. Graphical displays of critical peak wet weather flow data [refer to Paragraphs 11.241(b), 11.241(c) and 11.241(d)] should be included for a sustained wet weather flow period of significance to the project."*

To meet these requirements, this study used extensive flow monitoring, flow data analysis and projection methodologies consistent with EPA recommendations, and a computational sewer model has been used to analyze the existing sewer system.

### 8.5.2 New Collection System Hydraulic Capacity

This study includes recommendations to improve the existing sanitary sewer collection system. Based on experience with ADH, improvements to existing collection sewer systems have been required to comply with Section 11.243 concerning the hydraulic capacity of new collection systems. The requirements of this section are quoted below for reference:

*"11.243 Hydraulic Capacity for Wastewater Facilities to Serve New Collection Systems*

> *a. The sizing of wastewater facilities receiving flows from new wastewater collection systems shall be based on an average daily flow of 100 gallons (380 L) per capita plus wastewater flow from industrial plants and major institutional and commercial facilities unless water use data or <u>other justification upon which to better estimate flow is provided.</u>*
>
> *b. The 100 gal/cap/d [380 L/(capita/d)] value shall be used in conjunction with a peaking factor from Figure 1 to cover normal infiltration for systems built with modern construction techniques. Refer to Section 31. However, an additional allowance should be made where conditions are unfavorable.*
>
> *c. If the new collection system is to serve existing development the likelihood of I/I contributions from existing service lines and non-wastewater connections to those service lines shall be evaluated and wastewater facilities designed accordingly."*

Part A of this Standard requires justification to use an alternative flow estimate. Using the 10 States Standards value of 100 gpcd overestimated the existing conditions flow by approximately 18% - 3.86 MGD vs. 3.28 MGD. Overestimating flow can cause pipes to be oversized for the users they serve and can create more maintenance and longevity issues when flows are too low. The overestimation led to the development of a modified 10 States approach presented in Section 9.4.

Wet weather using 10 States is estimated using a peaking factor based on population to give a calculated peak hourly flow rate. The peaking factor covers normal infiltration for systems built with modern construction techniques.

Olsson's 10 States analysis spreadsheet is provided in Appendix D. Each basin is analyzed starting from the furthest upstream manhole to the furthest downstream pipe.

## 8.6 Design Storm Development

In Part I of this study, the existing sewer collection system was analyzed using design storms of various frequency. Design rainfall depths were derived from the National Oceanic and Atmospheric Administration (NOAA) Atlas 14 for 1-, 2-, 5-, 10-, and 25-year return interval rain events for the City of Bentonville. A return interval year is directly related to the frequency a storm is likely to occur. For example, a 10-year storm has a 1 in 10 or 10% probability of happening in any given year. The 24-hour rainfall amounts are as follows:

- 1-year design storm: 3.36-inches
- 2-year design storm: 3.79-inches
- 5-year design storm: 4.53-inches
- 10-year design storm: 5.19-inches
- 25-year design storm: 6.16-inches

For part II, a single design storm was used as the basis for system analysis and recommended improvements. Olsson recommended using a design storm with a 5-year return interval and 24-hour duration for design purposes. This design approach balances preparation for significant events while avoiding over-design for rare extreme events. Olsson distributed the statistical rainfall event over a 24-hour period using the Natural Resources Conservation Service (NRCS) synthetic storm Type II hyetograph. Figure 13 illustrates the 5-year, 24-hour design storm hyetograph used in the model.

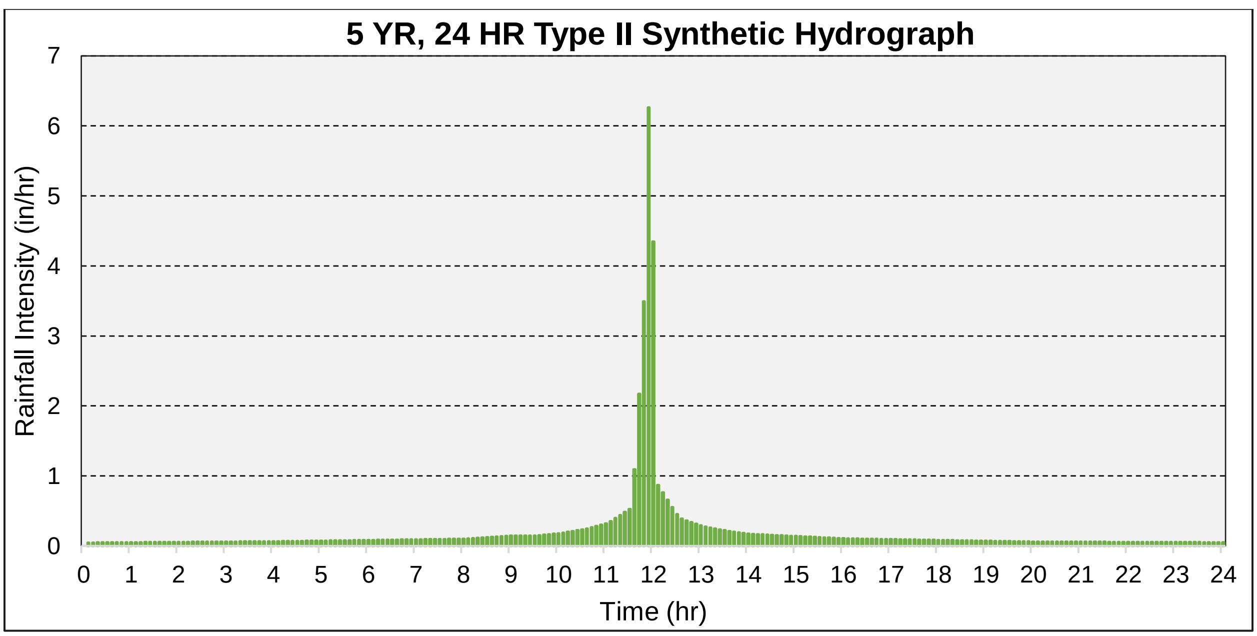



**Figure 13. 5YR-24HR Type II Synthetic Hydrograph.**

# 9. SYSTEM EVALUATION

This section presents the findings for the evaluation of the existing sewer collection system. The evaluation included current and future flow conditions, covering planning horizons for near-term through build-out. Capacity deficiencies are addressed in this section and recommended infrastructure improvements detailed in Section 10. The evaluation used the 5-yr 24-hr design storm (described in Section 8.6) and was based on observed flow conditions during the monitoring period, as well as projected flow conditions for ultimate buildout based on the Future Land Use Plan, presented in Section 5.

The primary goals of this evaluation were to identify pipes approaching maximum capacity and manholes at risk of surcharging due to downstream constraints during wet weather conditions. It also identifies pipes with low velocities during maximum dry weather conditions, which could result in operational or maintenance issues. The criteria used to evaluate the system capacity are discussed in Section 8.

The Future Land Use Map was utilized to develop the ultimate capacity required for pipes to handle projected ultimate buildout flows, ensuring no surcharging SSOs. Intermediate horizons, not included in the FLUM Map, were developed using a modified version of the 10 States Standards by interpolating population values, as described in Section 5.2. This evaluation discusses the design flows predicted at the Bentonville WRRF for each design horizon.

## 9.1 Existing System Dry Weather Flow Evaluation

Under existing DWF conditions, the system has adequate capacity and does not experience surcharge conditions. Additionally, the maximum velocities were found to be below the recommended 2 ft/s minimum scouring velocity, primarily in the upstream segments of the modeled network, under both existing and 2050 dry weather design flows. Pipes with low velocities during current dry weather conditions are shown on Figure 14.

While the system does not surcharge under current dry weather conditions, it does shown signs of surcharging and SSOs under projected ultimate buildout dry weather conditions, particularly in the Shewmaker 1, South LS 1, 2, 3, and 4 Subbasins. These surcharging and SSOs are primarily due to capacity constraints, as shown on Figure 15 and Figure 16.

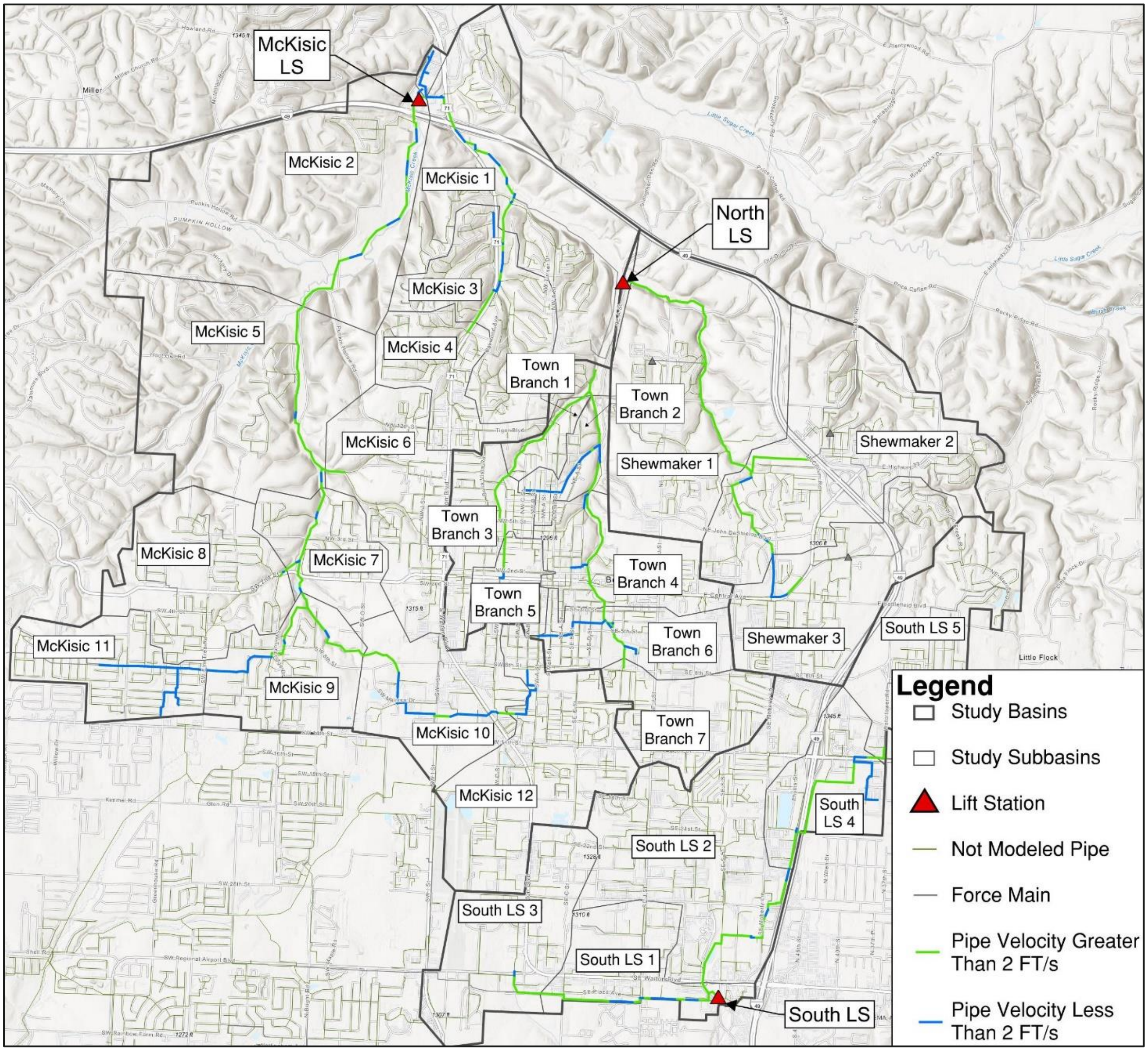



**Figure 14. Existing System Minimum Velocity Evaluation - Current Dry Weather Conditions.**

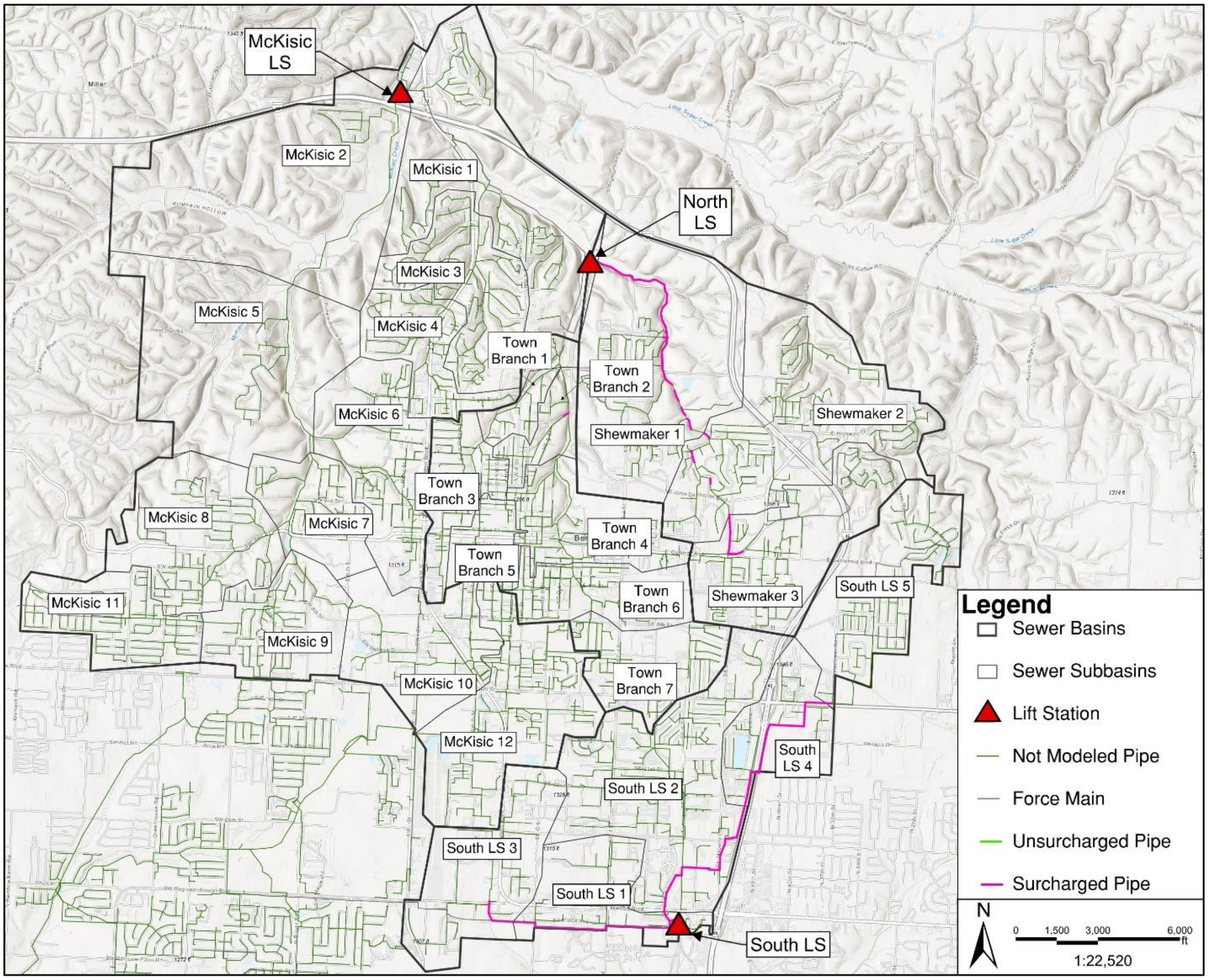



**Figure 15. Existing System Capacity Evaluation - 2050 Dry Weather Conditions.**

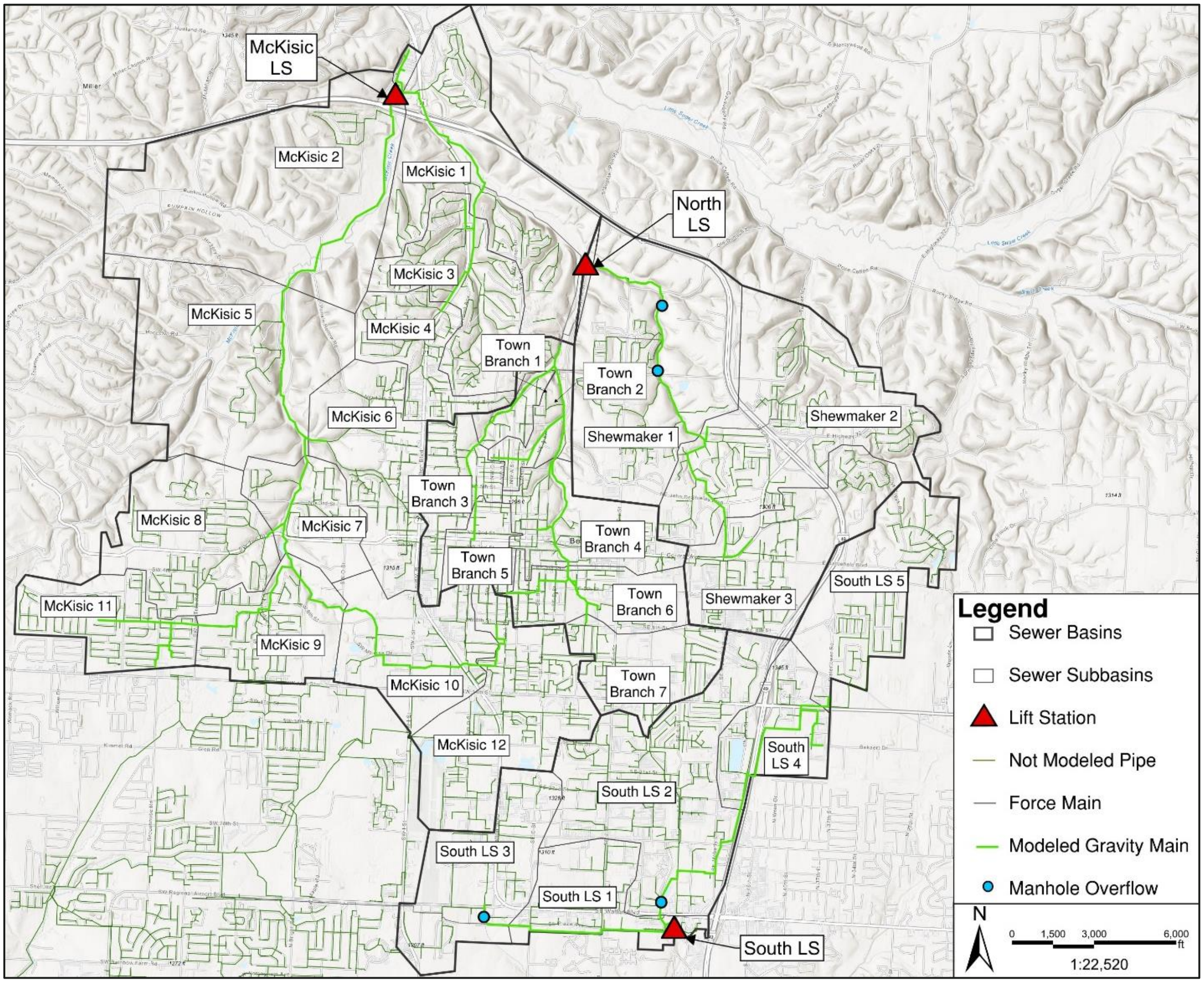



**Figure 16. Existing System Overflows – 2050 Dry Weather Conditions.**

## 9.2 Existing System Wet Weather Evaluation

When the current system and existing population experiences the 5-yr 24-hr design storm, the model predicts 130 surcharged pipes and 6 SSO's, summarized by basin in Table 15. Maps showing the locations of surcharged pipes and manholes can be found in Appendix C.

**Table 15. Current System with Existing Population - Wet Weather Results.**

| Subbasin | Surcharged Pipes | Predicted SSO's |
|---|---|---|
| **McKisic** | 18 | 0 |
| **Shewmaker** | 31 | 3 |
| **South Lift Station** | 51 | 0 |
| **Town Branch** | 30 | 3 |
| **Total** | 130 | 6 |

When the current system with the projected ultimate buildout population experiences the 5-yr 24-hr design storm, the model predicts 181 surcharged pipes and 28 SSO's, summarized by basin in Table 16. Maps showing the locations of surcharged pipes and manholes can be found in Appendix C.

**Table 16. Current System with 2050 Projected Population - Wet Weather Results.**

| Subbasin | Surcharged Pipes | Predicted SSO's |
|---|---|---|
| **McKisic** | 41 | 1 |
| **Shewmaker** | 42 | 11 |
| **South Lift Station** | 58 | 10 |
| **Town Branch** | 40 | 6 |
| **Total** | 181 | 28 |

## 9.3 WRRF Design Flow Projections

The City owns and operates one WRRF which receives flows from the McKisic and North lift stations in addition to the Town Branch basins that flow in by gravity. The South lift station sends its flows to the NACA WWTF. The existing WRRF has a current treatment capacity of 4 MGD and can pass a peak flow of 10 MGD. The city has improvements in progress that will increase the treatment capacity to 8 MGD with a peak flow of 30 MGD. The existing 4 MG of storage in the system at the McKisic Lift Station site is sufficient for current and future conditions.

The two 2-MG equalization tanks at the McKisic lift station are the only storage available in the system. During extreme wet weather events, McKisic storage is utilized when flows into the plant approach the treatment capacity of the WRRF. In order to achieve this, operators manually turn pumps on and off at the McKisic Lift Station forcing the storage basins to fill. Pumps are manually turned back on when the plant has available treatment capacity, and the McKisic lift station resumes normal operation.

The WRRF influent flow projections for the existing and ultimate buildout are presented in Table 17. The flows listed in the table below show the equalization tanks at McKisic Lift Station partially utilized during the 5-Year Design Storm when the flows into the WRRF surpassed an operational threshold. When evaluating the performance of the McKisic Lift Station under existing conditions, the current peak flow of 10 MGD was used, and the existing storage was approximately 70% utilized. When evaluated under the 2050 flow conditions, the future peak flow of 30 MGD was used, and the existing storage is approximately 40% utilized.

**Table 17. WRRF Influent Flow Projections by Design Horizon.**

| Flow Condition | Flow | Existing | Ultimate Buildout |
|---|---|---|---|
| **Dry Weather Flow** | Avg Daily Volume (MG) | 2.5 | 8.2 |
| | Peak Hour (MGD) | 3.6 | 10.7 |
| | Instantaneous Peak (MGD) | 5.3 | 12.6 |
| **5-YR Storm Wet Weather Flow** | Avg Daily Volume (MG) | 8.5 | 17.8 |
| | Peak Hour (MGD) | 15.2 | 27.6 |
| | Instantaneous Peak (MGD) | 15.9 | 29.5 |
| **McKisic Performance under 5-YR Storm** | Estimated Storage Required | 2.7 | 1.6 |
| | Daily Pumped Volume (MG) | 1.6 | 4.6 |
| | Peak Pumping Rate (MGD) | 4.4 | 6.1 |

## 9.4 I/I Reduction Evaluation

Design flows were calculated based on flow projections for every five (5) years from 2025 through 2050, using the 10 States Standards value of 100 gpcd. The peaking factor for the existing pipes was adjusted in the spreadsheet to reflect cases where the model predicted a peak wet weather response more than four times the average DWF.

Recommended projects are grouped based on similar flow triggers, which are determined by the wet weather response and average dry weather flow. As upstream population increases, the average DWF through a pipe also increases. The wet weather response of a pipe is directly related to the RDII of upstream connections, which can be addressed through I/I reduction efforts, as detailed in Section 7.

As BWU implements I/I reduction measures, flow monitoring should be performed to evaluate changes in the wet weather response in the areas of recommended projects. The revised wet-weather response will allow for higher DWF, thereby accommodating more growth upstream before the recommended improvements are required. Each recommended project includes the manhole ID of the pipe with the lowest capacity and its flow trigger during DWF, assuming no I/I reduction efforts. Any recommended projects downstream of I/I reduction efforts should be reevaluated before construction to determine if the flow trigger can be increased.

## 9.5 Lift Station and Infrastructure Evaluation

Three (3) major lift stations were included in the hydraulic model - McKisic, North, and South Lift Stations. The McKisic and North Lift Stations both pump and discharge directly to the WRRF and the South Lift Station discharges into a section of the City's sanitary sewer system that drains by gravity to the NACA WWTF.

The McKisic Lift Station has two (2) separate wet wells that drain different subbasins. The Turner side of McKisic receives flow from subbasins MK1, MK3, and MK4. The Turner side currently has four (4) 5-hp pumps that discharge through an 18-inch force main approximately 100 feet away into a gravity manhole that combines flow from the Turner side with flows from subbasins MK2, MK5, MK6, MK7, MK8, MK9, MK10, MK11, and MK12 before passing through a bar screen and entering the Dogwood side wet well. There are two (2) 2.0-million-gallon equalization basins that are connected to the Dogwood wet well and are used for storage during wet weather. The Dogwood side of McKisic has five (5) 85-hp pumps that pump directly to the WRRF via a 24-inch force main.

The Turner side of McKisic is expected to pass peak flows of 3.3 MGD in ultimate buildout, exceeding the current firm capacity of 1.5 MGD. Due to the lack of storage connected to the Turner side, improvements will be required to adequately pass peak flows without surcharging pipes or creating SSOs. The Dogwood side of McKisic is expected to pass peak flows of 8.2 MGD in 2050, exceeding the current firm capacity of 6.9 MGD. Based on the available storage at the McKisic Lift Station, the existing pumps in the Dogwood side are adequate to prevent SSOs and surcharging during a 5-yr 24-hr design storm when fully utilizing the available storage.

The North Lift Station receives all flow from the Shewmaker basin and uses four (4) 37-hp pumps that discharge directly to the WRRF via a 12-inch force main. The North Lift Station does not have any storage and an existing firm capacity of 3.17 MGD. Peak flows in ultimate buildout with the 5-yr 24-hr design storm are projected to be 6.8 MGD, requiring improvements to be made to the existing pumps and the associated force main. This lift station is being analyzed and improved as a part of another project and improvements will not be presented for it in this report.

The South Lift Station receives all flow from the South Lift Station basin and uses three (3) 36-hp pumps that discharge into a gravity manhole via a 16-inch force main and then flows south to the NACA WWTF. The South Lift Station does not have any storage and an existing firm capacity of 3.46 MGD. Peak flows in ultimate buildout during a 5-yr 24-hr design storm are projected to be 10.2 MGD, requiring improvements to be made to the existing facility. Two improvements options are recommended for the South Lift Station; One 2-part project that includes constructing a new flow equalization tank and improving the existing pumps, and one project that completely replaces the existing station and force main. In conversations with BWU staff, a single upsized force main was preferential to installing a dual main, the recommended improvements and estimated opinions of cost presented in Section 10 reflect this preference.

# 10. RECOMMENDED IMPROVEMENTS

The following section outlines the recommended improvements to the sanitary sewer system to be implemented by the city as part of the Capital Improvement Plan (CIP). These project improvements were developed using the hydraulic sewer model to eliminate surcharging in pipes for the 2050/Ultimate Buildout design horizon during a 5-yr 24-hr design storm. The recommended improvements are grouped by sewer basin into projects by similar maximum capacities.

To verify that all the recommended improvements will satisfy ADH requirements, the system was exported to a spreadsheet for desktop analysis using the 10 States Standards to establish design flows. This analysis allowed for adjustments or confirmations of recommended pipe sizes and project extents for the 2050/Ultimate Buildout design horizon. This spreadsheet also helped determine the specific horizon at which each project would be required.

Using the peak flows derived from the adjusted peak factors described in Section 9.4, design horizons were assigned to each recommended project based on the projected DWF trigger. Five (5) year intervals were used to approximate when to expect the DWF trigger would be met, based on the projected population growth for each horizon. These projected horizons are approximate, and flow monitoring should be conducted immediately upstream of the most capacity-constrained pipe in each project area to determine when field verified DWF conditions are present, prior to implementing any recommended project.

Appendix E contains a map showing location of all recommended CIP projects. Additionally, basin-specific maps are provided, along with estimated project schedules as well as the DWF trigger details.

## 10.1 Cost and Schedule Estimation Methodology

A comprehensive cost estimate, or Opinion of Probable Cost (OPC), was prepared for each recommended improvement project. Costs were estimated using AACE Class 3 guidelines. AACE Class 3 cost estimates are typically used for budget, authorization, or control and are estimated using semi-detailed unit costs with assembly level line items. The expected accuracy of an AACE Class 3 cost estimate ranges from 10% - 20% low or 10% - 30% high.

Each OPC includes construction costs, contingencies for design and construction, mobilization, contractor overhead and profit, escalation, engineering fees, and costs for easement acquisition, if required. Design, acquisition, bid, and construction services are estimated not to exceed a total of 15% for gravity line projects and a total of 18% for lift station projects. Table 18 provides a detailed breakdown of the values for each parameter used in this report, assuring transparency and accuracy in cost estimates.

**Table 18. Cost Estimate Assumptions.**

| Parameter | Value |
|---|---|
| **Temporary Easement Acquisition** | $5/square foot |
| **Real Estate** | $200/square foot |
| **Contingency** | 30% |
| **Design, Acquisitions, Bid and Construction Services** | 15% Gravity or 18% Lift Stations |
| **Project Escalation** | 5% compounded annually to each project horizon |

Recommended gravity sewer improvements are primarily estimated using a traditional dig-and-replace method. This construction approach involves bypass pumping, trench excavation to remove the old pipe, installation of a new, larger pipe, backfilling, and surface restoration. Alternatively, pipe bursting may be desirable in many locations throughout the city. Pipe bursting is a trenchless construction method that involves pulling a hydraulic or pneumatic expansion head through an existing pipe. As the expansion head is pulled, it breaks apart the existing pipe, creating space to pull new pipe in behind it. The main advantage of pipe bursting is the reduced surface disturbance, minimizing construction impacts to businesses, homeowners, and traffic. For early implementation of gravity projects in urban areas, Olsson recommends assessing the feasibility of pipe bursting by evaluating factors such as the number of service connections, soil type, conflicting utilities, and overall cost.

All recommended projects presented in this report have a 2025 dollar as well as an escalated total project cost. Regional construction pricing experienced a sharp uptick from 2021 to 2023 due to COVID and large demand in the area. Prices have stabilized but demand is still high and forecasted to stay high which will continue to escalate construction pricing. Due to these observations and assumptions, a 5% per year escalation is used in this study to project future project costs.

The estimated project duration includes preliminary planning, design, bid, and construction phases calculated with the values shown in Table 19.

**Table 19. Schedule Development Parameters.**

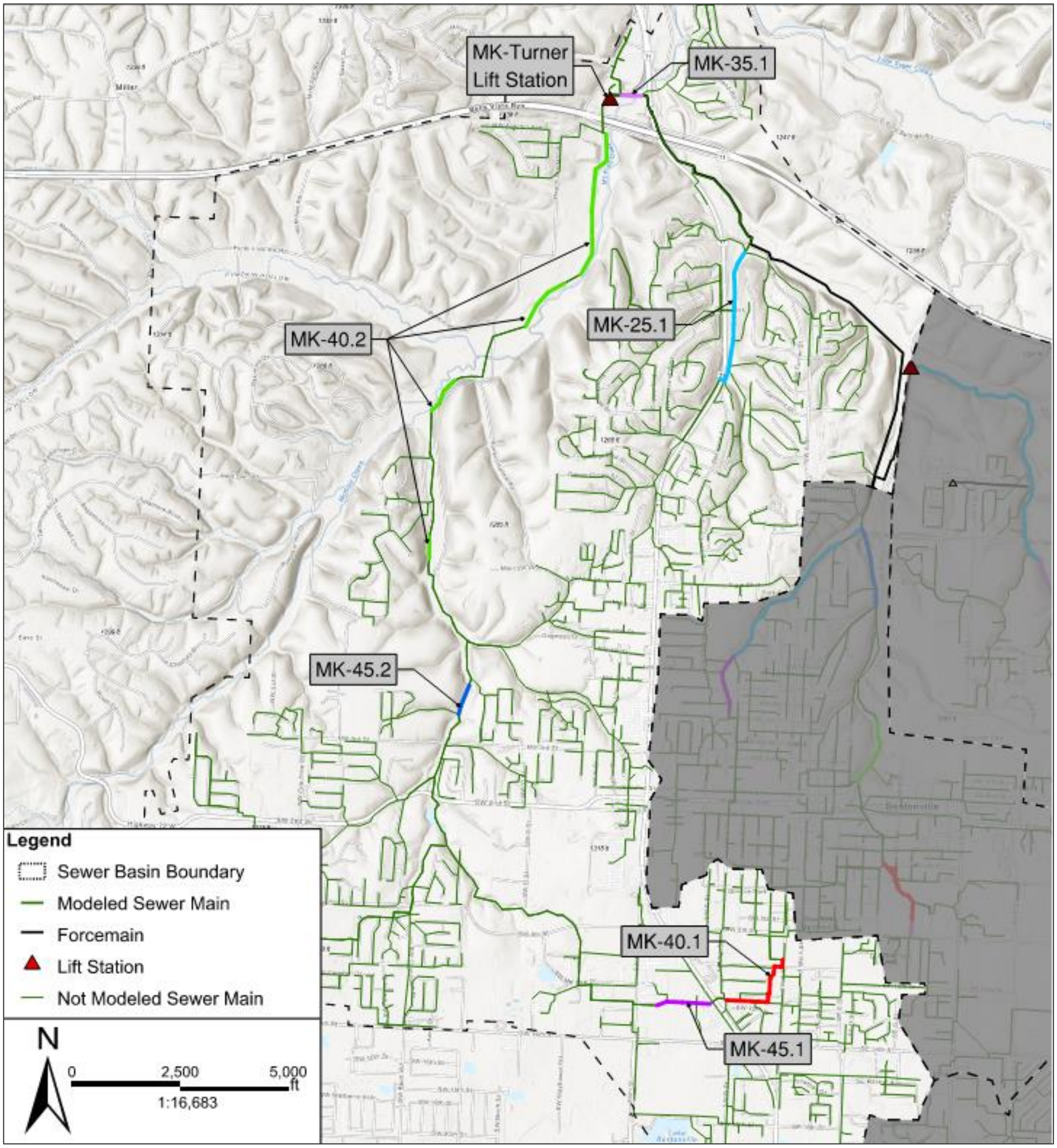
| Parameter | Value |
|---|---|
| **Initiation/Pre-Design** | Assumed at 3 Months |
| **Design** | Calculated at 1 Month per 800 feet of Improvement (or a Minimum of 5 Months) |
| **Bid/Construction** | Calculated at 1 Month per 600 feet of Improvement (or a Minimum of 6 Months) |

## 10.2 McKisic Basin – Recommended Improvements

The McKisic basin occupies the western and northern areas of Bentonville and drains by gravity to the McKisic Lift Station, where flows are pumped via force main to the WRRF. The McKisic basin has a current population of 19,971 and future flows were projected using a residential population of 35,399 and a commercial population of 23,742 as described in Section 5.2. A summary of recommended projects and costs in the McKisic Basin can be found in Table 20 and Figure 17.

**Table 20. McKisic Basin – Summary of Recommended Improvements.**

| Project ID | Total Length (Feet) | Diameter (Inch) | Project Duration | Total Project Cost | Escalated Total Project Cost |
|---|---|---|---|---|---|
| **MK-25.1** | 3,167 | 8/10 upsized to 12 | 18 Months | $3,327,000 | $3,852,000 |
| **MK-Turner LS** | - | - | 14 Months | $484,000 | $561,000 |
| **MK-35.1** | 462 | 18 upsized to 24 | 14 Months | $1,150,000 | $1,874,000 |
| **MK-40.1** | 2,214 | 12 upsized to 18 | 15 Months | $2,857,000 | $5,940,000 |
| **MK-40.2** | 6,179 | 24 upsized to 30 | 27 Months | $9,189,000 | $19,104,000 |
| **MK-45.1** | 1,237 | 18 upsized to 24 | 14 Months | $6,488,000 | $17,215,000 |
| **MK-45.2** | 717 | 24 upsized to 30 | 14 Months | $1,225,000 | $3,251,000 |
| **Total** | **13,975** | **-** | **-** | **$24,708,000** | **$51,783,000** |




[1]Projects are color coded by Project ID.

**Figure 17. Recommend Capital Improvement Projects[1]: McKisic Basin.**

Project MK-25.1 is located in subbasins MK1 and MK3, upstream of the Turner Lift Station. Potential surcharging of the pipes in this project is estimated to occur in the 2025 horizon. This project crosses and then runs parallel to N Walton Blvd through mostly open green space and includes upsizing 482 feet of 8-inch and 2,685 feet of 10-inch pipe to 12-inch. This recommended project has an estimated escalated cost of $3,852,000 and project duration of 18 months.

MK-Turner LS improvements are estimated to be needed in the 2025 horizon, with an estimated cost of $561,000 and project duration of 14 months. The existing firm capacity of the Turner side is 1.5 MGD and the estimated 2025 peak flow is 2.6 MGD. Recommended improvements include upsizing the four (4) existing 5-hp pumps with 15-hp pumps on the Turner side of the McKisic Lift station to pass the projected 2050 peak flows of 3.3 MGD, without surcharging pipes or creating SSOs.

Project MK-35.1 is located in subbasin MK1, directly upstream of the Turner Lift Station. Potential surcharging of the pipes in this project is estimated to occur during the 2035 horizon. This project is northwest of the Interstate 49/Highway 71 interchange and crosses McKisic Creek. This project includes upsizing 462 feet of 18-inch pipe to 24-inch. This recommended project has an estimated escalated cost of $1,874,000 and project duration of 14 months.

Project MK-40.1 is located in subbasins MK12 and MK10, in southwest, downtown Bentonville. Potential surcharging of the pipes in this project is estimated to occur during the 2040 horizon. This project is in a residential neighborhood and includes upsizing 2,214 feet of 12-inch to 18-inch with an estimated escalated cost of $5,940,000 and project duration of 15 months. This project should consider pipe bursting as an alternative to open trench replacement during design to help limit the impact of construction.

Project MK-40.2 is located in subbasins MK5 and MK2, upstream of the Dogwood Lift Station. Potential surcharging of the pipes in this project is estimated to occur during the 2040 horizon. This project runs along McKisic Creek and includes upsizing four areas of existing 24-inch to 30-inch, totaling 6,179 feet, with an estimated escalated cost of $19,104,000 and project duration of 27 months.

Project MK-45.1 is located in subbasin MK10, slightly downstream of Project MK-40.1 in a commercial/residential mixed-use area. Potential surcharging of the pipes in this project is estimated to occur during the 2045 horizon. This project includes upsizing 1,237 feet of 18-inch pipe to 24-inch, with an estimated escalated cost of $17,215,000 and duration of 14 months.

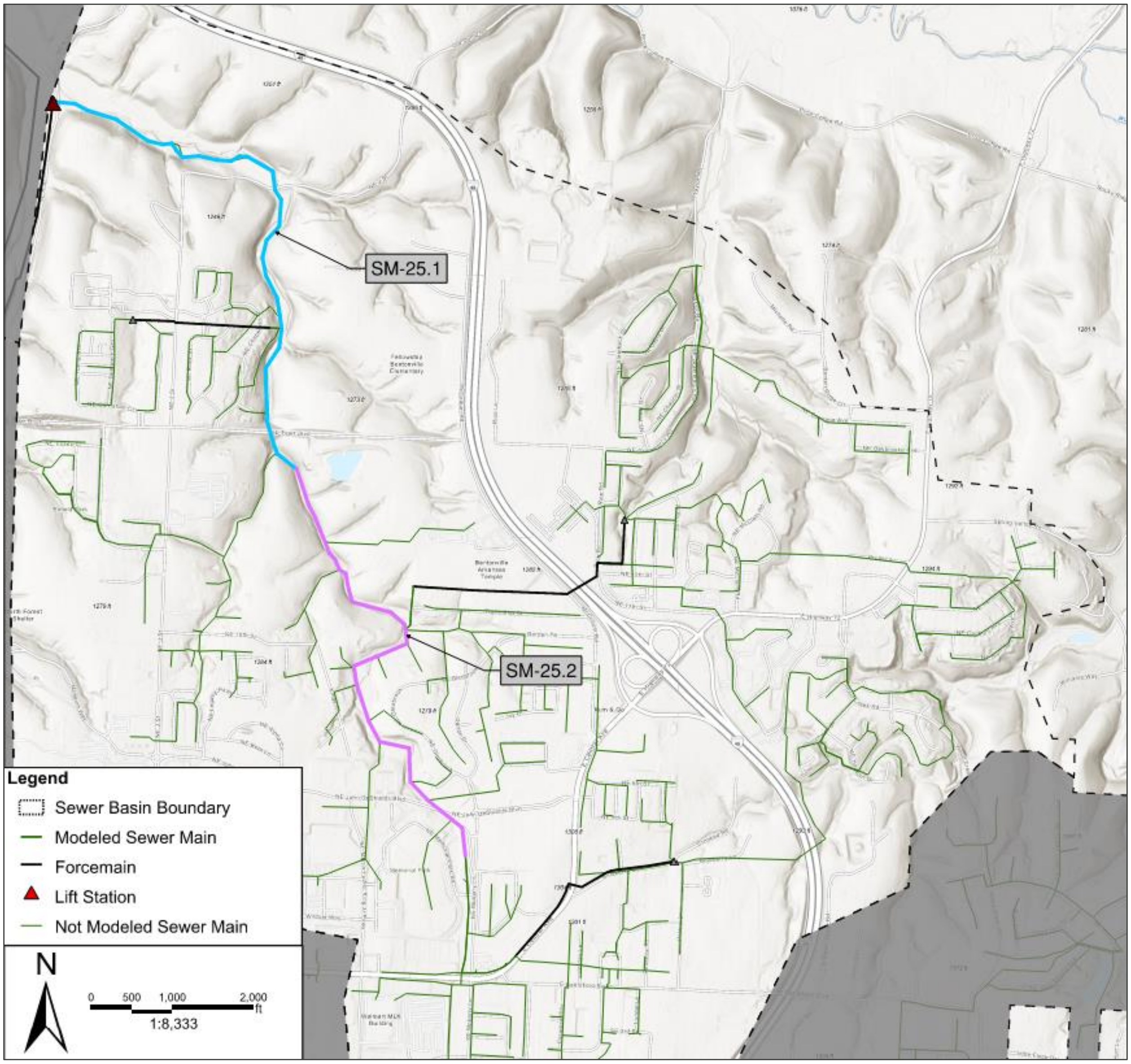
Project MK-45.2 is located in subbasin MK7, located along Applegate Trail in Coler Mountain Park. Potential surcharging of the pipes in this project is estimated to occur in the 2045 horizon. This project includes upsizing 717 feet of 24-inch pipe to 30-inch with an estimated escalated cost of $3,251,000 and project duration of 14 months. This project should consider pipe bursting as an alternative to open trench replacement during design to help limit the impact of construction.

## 10.3 Shewmaker Basin – Recommended Improvements

The Shewmaker basin occupies the northeastern area of Bentonville and drains by gravity to the North Lift Station, where flows are pumped via force main to the WRRF. The Shewmaker has a current population of 7,318 and future flows were projected using a residential population of 25,493 and a commercial population of 31,265 as described in Section 5.2. A summary of recommended projects and costs in the Shewmaker Basin can be found in Table 21 and Figure 18. The Shewmaker Basin is currently being studied more in depth by another Engineering firm, who will be recommending improvements to the North Lift Station as a part of their project.

**Table 21. Shewmaker Basin – Summary of Recommended Improvements.**

| Project ID | Total Length (Feet) | Diameter (Inch) | Project Duration | Total Project Cost | Escalated Total Project Cost |
|---|---|---|---|---|---|
| **SM-25.1** | 6,915 | 12/18 upsized to 24 | 29 Months | $10,199,000 | $11,807,000 |
| **SM-25.2** | 6,449 | 10/12 upsized to 18 | 28 Months | $7,294,000 | $8,444,000 |
| **Total** | **13,364** | **-** | **-** | **$17,493,000** | **$20,251,000** |




[1]Projects are color coded by Project ID.

**Figure 18. Recommended Capital Improvement Projects[1]: Shewmaker Basin.**

Project SM-25.1 is located in subbasin SM1 and is immediately upstream of the North Lift Station. Potential surcharging of the pipes in this project are estimated to occur in the 2025 horizon. This project includes upsizing 6,578 feet of 12-inch pipe to 24-inch and 337 feet of 18-inch to 24-inch, with an estimated escalated cost of $11,807,000 and project duration of 29 months.

Project SM-25.2 is located in subbasins SM1 and SM2 and is immediately upstream of Project SM-25.1. Potential surcharging of the pipes in this project are estimated to occur in the 2025 horizon. This project includes upsizing 3,949 feet of 10-inch pipe with 18-inch and 2,500 feet of 12-inch pipe with 18-inch, with an estimated escalated cost of $8,444,000 and project duration of 28 months.

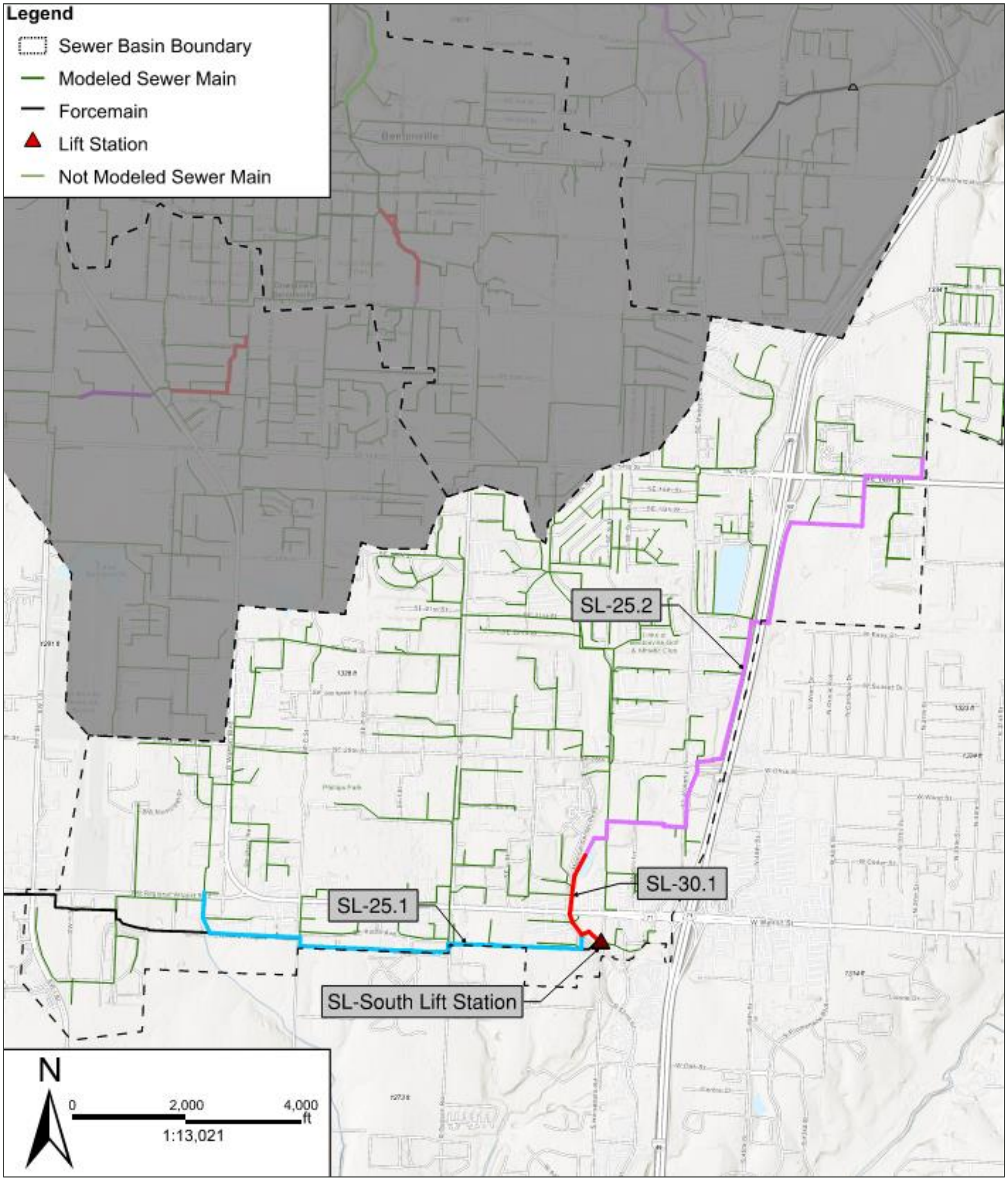
# 10.4 South Lift Station Basin – Recommended Improvements

The South Lift Station basin occupies the southern and southeastern areas of Bentonville and drain by gravity to the South Lift Station, where flows are pumped via force main to gravity mains that drain to NACA WWTF. The South Lift Station Basin has a current population of 7,122 and future flows were projected using a residential population of 35,815 and a commercial population of 56,156 as described in Section 5.2. A summary of recommended projects and costs in the South Lift Station Basin can be found in Table 22 and Figure 19. Two options are presented for increasing capacity at the South Lift Station, but only Option 2 is included in the total costs. Olsson recommends Option 2 due in part to its lower overall cost as well as reduced impact downstream of the improvements.

**Table 22. South Lift Station Basin – Summary of Recommended Improvements.**

| Project ID | Total Length (Feet) | Diameter (Inch) | Project Duration | Total Project Cost | Escalated Total Project Cost |
|---|---|---|---|---|---|
| **SL-25.1** | 7,633 | 12 upsized to 18/24 | 31 Months | $14,037,000 | $16,250,000 |
| **SL-25.2** | 11,916 | 8/12/18 upsized to 18/24 | 43 Months | $17,431,000 | $20,179,000 |
| **SL-South Lift Station Option 1[1]** | 18,300 | 24 | 36 Months | $48,320,000 | $55,937,000 |
| **SL-South Lift Station Option 2, Part 1** | - | - | 30 Months | $30,140,000 | $34,891,000 |
| **SL-30.1** | 1,872 | 18 upsized to 24/30 | 15 Months | $3,528,000 | $4,503,000 |
| **SL-South Lift Station Option 2, Part 2** | - | - | 23 Months | $5,638,000 | $9,184,000 |
| **Total** | **21,421** | **-** | **-** | **$70,774,000** | **$85,007,000** |

[1] SL-South Lift Station Option 1 is not included in length or cost totals.




[1]Projects are color coded by Project ID.

**Figure 19. Recommended Capital Improvement Projects[1]: South Lift Station Basin.**

Project SL-25.1 is located in subbasins SLS1 and SLS3 and contains the western portion of the modeled pipe in the basin. Potential surcharging of the pipes in this project is estimated to occur in the 2025 horizon. This project includes upsizing 4,915 feet of 12-inch pipe to 18-inch and upsizing 2,718 feet of 12-inch pipe to 24-inch with a total estimated escalated cost of $16,250,000 and project duration of 31 months. An approximately 1,800 feet section located between a residential and commercial development of this project would be a good candidate for pipe bursting to reduce the impact of construction.

Project SL-25.2 is located in subbasins SLS2, SLS4, and SLS5, and contains most of the northern portion of the modeled pipe in the South Lift Station Basin. Potential surcharging of the pipes in this project is estimated to occur in the 2025 horizon. This project includes upsizing 1,320 feet of 8-inch and 9,740 feet of 12-inch pipe to 18-inch and upsizing 856 feet of 18-inch pipe to 24-inch with an estimated escalated cost of $20,179,000 and project duration of 43 months. The portion of the project that is upstream of the Interstate 49 crossing is mostly in open green space and suitable for open trench replacement. The portion of the project that crosses Interstate 49 and continues downstream enters into commercial and residential areas where pipe bursting should be considered as a construction alternative.

The South Lift Station has existing firm capacity of 3.46 MGD and the 2025 estimated peak flow coming into the station is 4.6 MGD, indicating immediate needs for improvements to prevent surcharging or causing SSOs. Both options presented address the immediate needs while also being sized to pass the estimated 2050 peak flow of 10.2 MGD. Further study will be required at the receiving manhole and downstream gravity pipes that will be affected by increased capacity of the pump station if either option is selected.

SL-South Lift Station Option 1 includes upsizing the pumps and force main associated with the South Lift Station. The recommended improvements include a new lift station with a firm capacity of 10.25 MGD with four (4) 65-hp pumps along with upsizing the existing 16-inch force main to a 24-inch force main using the existing alignment. If this option is selected, its estimated horizon is 2025 with an estimated escalated project cost of $55,937,000 and project duration of 36 months.

SL-South Lift Station Option 2 involves two parts: Part 1 includes the addition of a 1-MG flow equalization tank to store the excess peak flows, located just north of the existing lift station under a parking lot and needed in the 2025 horizon. The proposed equalization tank is limited by the size of the parking lot and soil stability during excavation. Part 1 has an estimated project cost of $34,891,000 and project duration of 30 months. Part 2 includes upsizing the three (3) lift station pumps from 35.75-hp pumps to 55-hp pumps to increase flow rate through the existing force main and is estimated to be needed in the 2035 horizon. The improvements recommended in Part 2 have an estimated escalated project cost of $9,184,000 and project duration of 23 months. Olsson recommends Option 2 due to the lower overall cost as well as reduced impacts downstream since the force main is not being upsized.

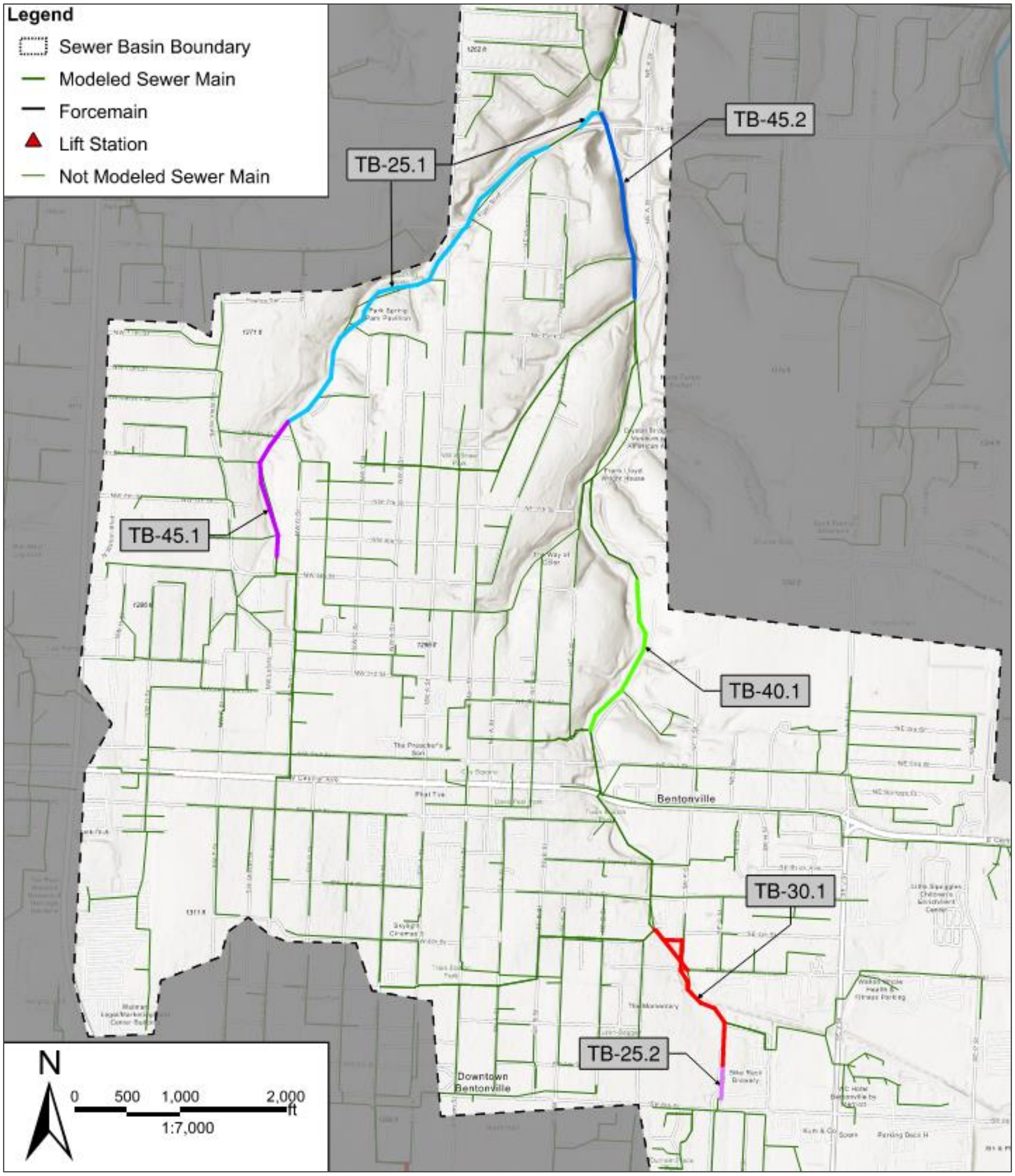
Project SL-30.1 is located in subbasins SLS1 and SLS2 and is immediately upstream of the South Lift Station. Potential surcharging of the pipes in this project is estimated to occur in the 2030 horizon. This project includes upsizing 1,544 feet of 18-inch pipe to 24-inch and 328 feet of 18-inch pipe to 30-inch with an estimated escalated cost of $4,503,000 and project duration of 15 months.

## 10.5 Town Branch Basin – Recommended Improvements

The Town Branch basin occupies central Bentonville and drains by gravity directly into the WRRF. The Town Branch Basin has a current population of 5,554 and future flows were projected using a combined population of 19,801 residents and 38,490 employees as described in Section 5.2. A summary of recommended projects and costs in the Town Branch Basin can be found in Table 23 and Figure 20.

**Table 23. Town Branch Basin – Summary of Recommended Improvements.**

| Project ID | Total Length (Feet) | Diameter (Inch) | Project Duration | Total Project Cost | Escalated Total Project Cost |
|---|---|---|---|---|---|
| **TB-25.1** | 4,019 | 12 upsized to 18 | 21 Months | $5,472,000 | $6,335,000 |
| **TB-25.2** | 317 | 12 upsized to 18 | 14 Months | $629,000 | $729,000 |
| **TB-30.1** | 2,043 | 12/15 upsized to 18 | 15 Months | $3,429,000 | $4,377,000 |
| **TB-40.1** | 1,595 | 18 upsized to 24 | 14 Months | $1,990,000 | $4,138,000 |
| **TB-45.1** | 1,378 | 12 upsized to 18 | 14 Months | $1,884,000 | $4,999,000 |
| **TB-45.2** | 1,780 | 24 upsized to 30 | 14 Months | $3,103,000 | $8,234,000 |
| **Total** | **11,132** | **-** | **-** | **$16,507,000** | **$28,812,000** |




[1]Projects are color coded by Project ID.

**Figure 20. Recommended Capital Improvement Projects[1]: Town Branch Basin.**

Project TB-25.1 is located in subbasin TB1 slightly upstream of the WRRF. Potential surcharging of the pipes in this project are estimated to occur in the 2025 horizon. This project includes upsizing 4,019 feet of 12-inch pipe to 18-inch with an estimated cost of $6,335,000 and project duration of 21 months.

Project TB-25.2 is located in subbasin TB7 and is the uppermost reach of the Town Branch trunk line. Potential surcharging of the pipes in this project is estimated to occur in the 2025 horizon. This project includes upsizing 317 feet of 12-inch pipe to 18-inch with an estimated cost of $729,000 and project duration of 14 months.

Project TB-30.1 is located in subbasin TB6 and immediately downstream of project TB25.2. Potential surcharging of the pipes in this project is estimated to occur in the 2030 horizon. This project includes upsizing 1,543 feet of 12-inch and 500 feet of 15-inch pipe to 18-inch with an estimated escalated cost of $4,377,000 and project duration of 15 months.

Project TB-40.1 is located in subbasin TB4 in the wooded area south of Crystal Bridges. Potential surcharging of the pipes in this project is estimated to occur in the 2040 horizon. This project includes upsizing 1,595 feet of 18-inch pipe to 24-inch with an estimated escalated cost of $4,138,000 and project duration of 14 months.

Project TB-45.1 is located in subbasin TB3 and is immediately upstream of Project TB-35.1. Potential surcharging of the pipes in this project is estimated to occur during the 2045 horizon. This project includes upsizing 1,378 feet of 12-inch pipe to 18-inch with an estimated escalated cost of $4,999,000 and project duration of 14 months. Due to this project's location along a creek with steep terrain, pipe bursting should be considered during design.

Project TB -45.2 is located in subbasin TB2 slightly upstream of the WRRF. Potential surcharging of pipes in this project is estimated to occur during the 2045 horizon. This project includes upsizing 1,780 feet of 24-inch pipe to 30-inch with an estimated escalated cost of $8,234,000 and project duration of 14 months. Due to this project's location along a creek with steep terrain, pipe bursting should be considered during design.

## 10.6 Recommended Pipe and Manhole Rehabilitation

Where recommended capacity improvements overlapped with rehabilitation recommendations, shown in Figure 21 and Figure 22, the following exclusions were made from TREKK's rehabilitation recommendations:

- If the recommended capacity improvement was estimated to be needed in 2025, priority 1 and 2 rehab recommendations were excluded.
- If the recommended capacity improvement was estimated to be needed in 2030, only priority 2 rehab recommendations were excluded.
- If the recommended capacity improvement was estimated to be needed after 2030, no rehab recommendations were excluded.

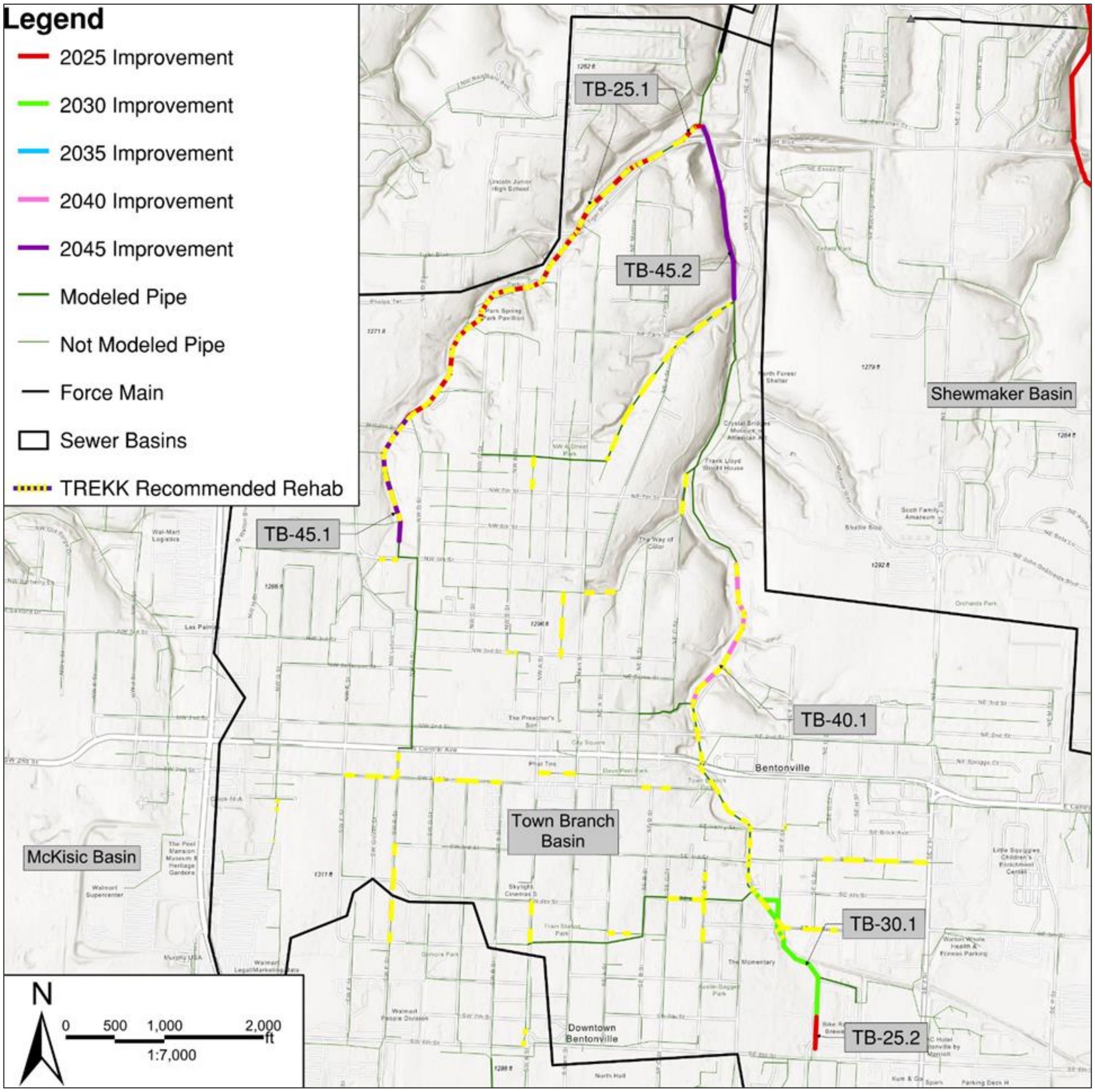



Figure 21. TREKK Recommended Pipe Rehab Overlay.

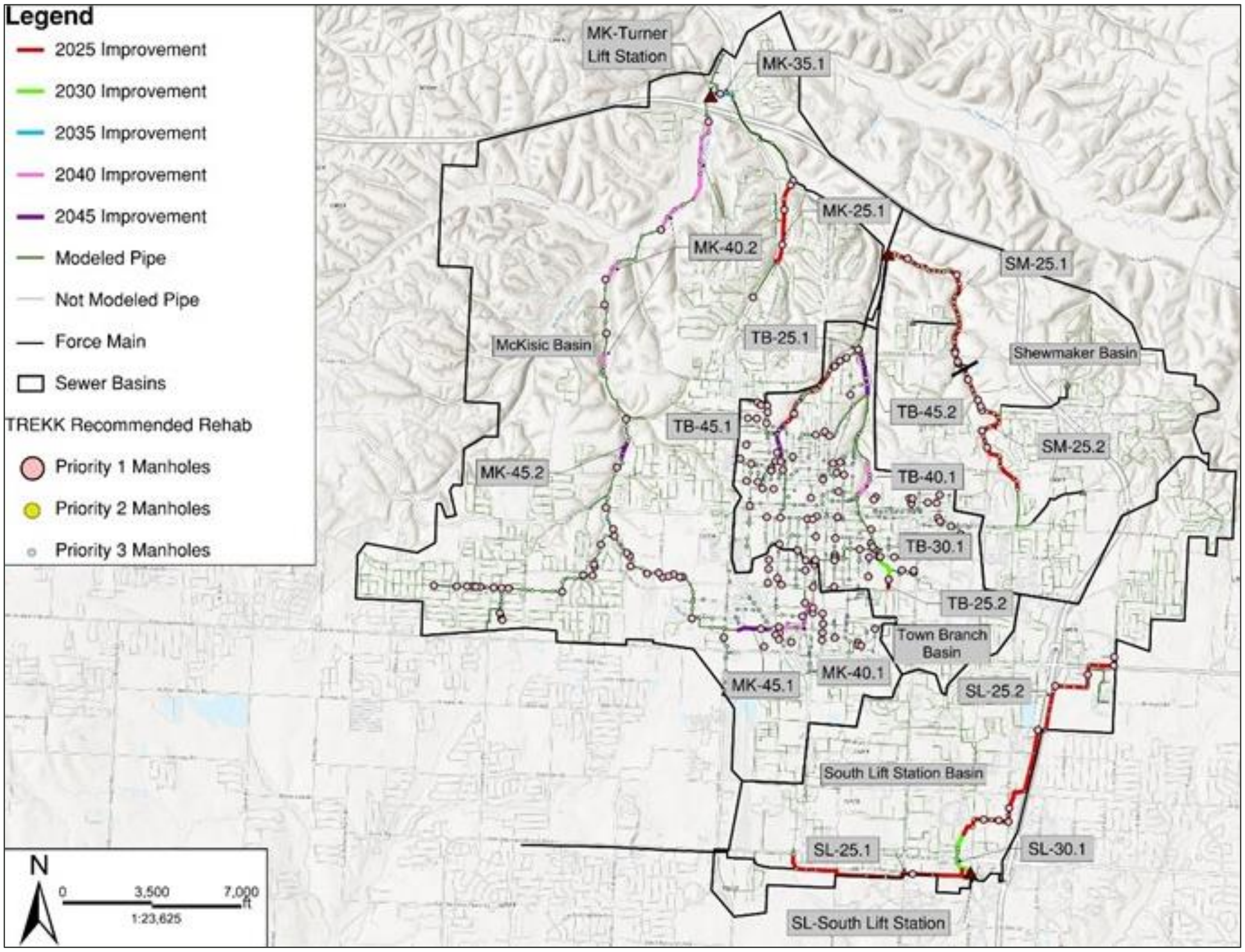


**Figure 22. TREKK Recommended Manhole Rehab Overlay.**

The costs associated with the SSES Part II recommendations, along with the manhole recommendations from SSES Part I (completed in 2019) along with the costs after are summarized in Table 24. Costs associated *With Listed Exclusions* have the priority exclusions listed above removed from TREKK's recommendations and the costs associated with *No Capacity Improvement Overlap* exclude any rehabilitation recommendation where there is also a recommended capacity improvement, regardless of estimated horizon.

Further details on the SSES can be found in the Technical Memo and Exhibits included in Appendix B of this report.

**Table 24. TREKK Recommended Improvement Cost Summary.**

| Study Area Improvements | TREKK's Original Recommendations | With Listed Exclusions | No Capacity Improvement Overlap |
|---|---|---|---|
| Private-Sector I/I Abatement Program[1] | $232,000 | $232,000 | $232,000 |
| Part 1 Manhole Rehabilitation Program (Priority 1 and 2)[2] | $1,670,000 | $1,208,000 | $1,044,000 |
| Part 2 Manhole Rehabilitation Program (Priority 1 and 2)[2] | $674,000 | $674,000 | $674,000 |
| Pipeline Rehabilitation Program (Priority 1 and 2)[1] | $1,544,000 | $1,365,000 | $1,105,000 |
| **Total Cost** | **$4,120,000** | **$3,479,000** | **$3,055,000** |

[1]Costs include 20% contingency.
[2]Costs include 30% contingency.

# BENTONVILLE BASELINE SANITARY SEWER CAPACITY STUDY – PART II

Bentonville, Arkansas - 2025

March 2025

Olsson Project No. 020-23210

CERTIFICATE OF AUTHORIZATION
Olsson Inc.
No.1010
ARKANSAS ENGINEER

STATE OF
ARKANSAS
REGISTERED
PROFESSIONAL
ENGINEER
★ ★ ★
03/13/2025
No. 14497
CHRIS DOUGHERTY