FINAL

SANITARY SEWER ASSET INVENTORY AND ASSESSMENT

Master Plan Report

B&V PROJECT NO. 194803
B&V FILE NO. 42.6500

PREPARED FOR

Hendersonville Water Sewer

CITY OF HENDERSONVILLE

25 MARCH 2019

BLACK & VEATCH

Black & Veatch International Company
10715 David Taylor Dr. Suite 240
Charlotte, NC 28262 USA
+1 704-548-8461 | Firm License No. F-0794

Table of Contents

LIST OF TABLES

LIST OF FIGURES

APPENDICES

Acronym List

BSF	Base Sanitary Flow
CCTV	Closed-Circuit Television
CIP	Capital Improvement Plan
CIWEM	Chartered Institution of Water and Environmental Management
COF	Consequence of Failure
COH	City of Hendersonville
DWF	Dry Weather Flow
GIS	Geographic Information System
GWI	Groundwater Infiltration
I/I	Inflow and Infiltration
LOF	Likelihood of Failure
MGD	Million Gallons per Day
MMPF	Maximum Month Peaking Factor
NCAC	North Carolina Administrative Code
QA/QC	Quality Assurance/Quality Control
RDII	Rainfall Dependent Inflow and Infiltration
SL-RAT	Sewer Line Rapid Assessment Tool
SSAIA	Sanitary Sewer Asset and Inventory Analysis
SSO	Sanitary Sewer Overflow
TWC	Total Water Consumption
WaPUG	Wastewater Planning Users Group
WWTP	Wastewater Treatment Plant

Executive Summary

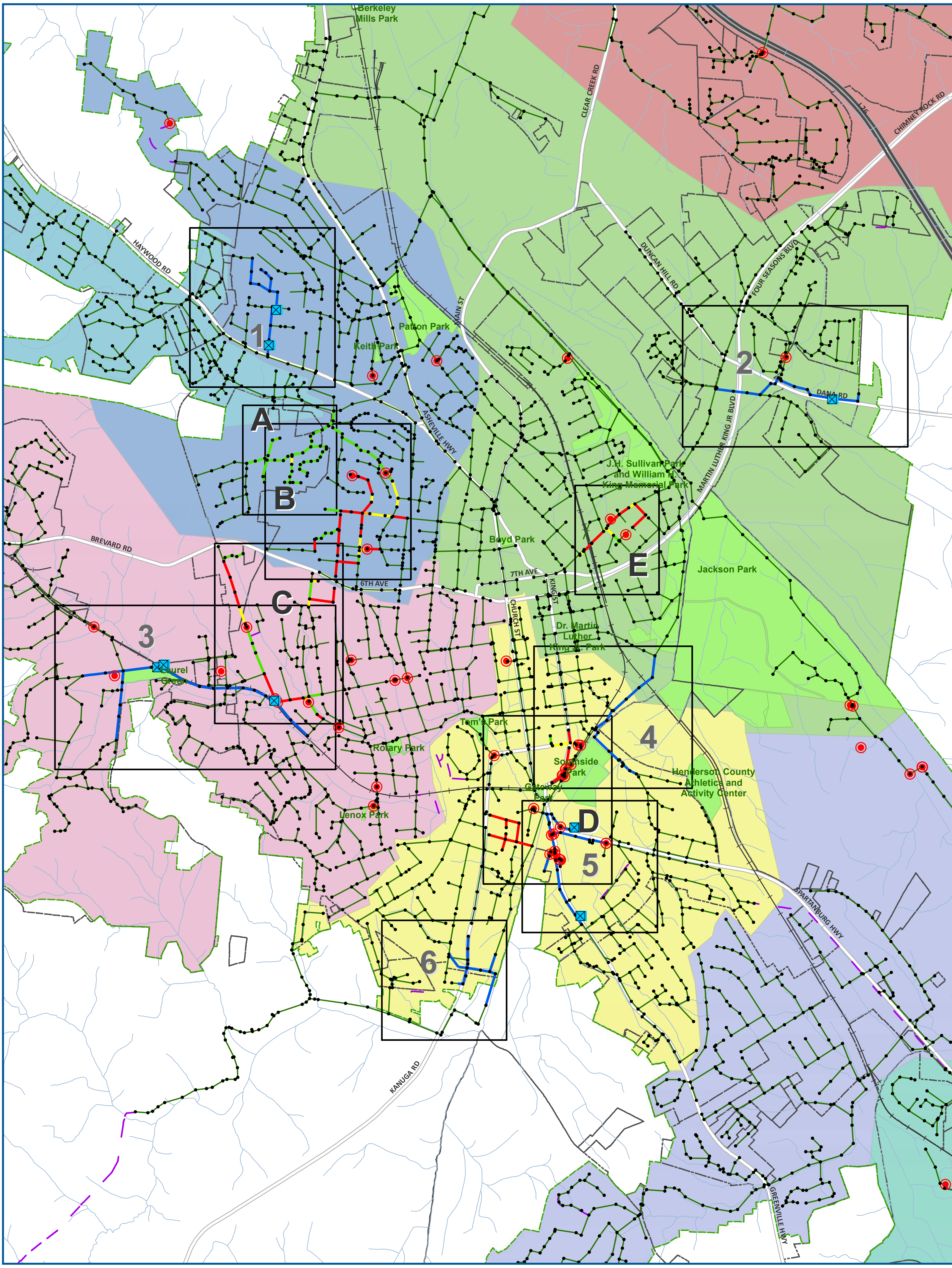
In 2017 and 2018, Black & Veatch worked with the City of Hendersonville (COH) to perform its Sanitary Sewer Asset Inventory and Analysis (SSAIA), which will serve as its master plan document. This master plan provides COH with a roadmap to maintain, improve, and expand its collection system so that the COH can operate a great utility for all its current and future customers.

The SSAIA included a condition assessment of the COH's sewer system, development and calibration of a dynamic hydraulic model, flow projections from the COH's service area through 2040, hydraulic capacity assessment of the sewer system, and finally, the development of a risk-based, prioritized capital improvement program (CIP).

CONDITION ASSESSMENT

The condition assessment work consisted of a review of available information as well as field inspections to collect information on the current conditions of the sewer pipes and manholes in the system. The results of these inspections were used to identify locations of potential infiltration and inflow (I/I) and the location of possible blockages and structural defects in the inspected areas so that the overall condition of the entire system could be estimated. More detailed inspections will be required to develop specific capital projects, but these inspections provided useful information in describing the general condition of the system. The field inspections, performed in two phases, included smoke testing, lift station visits, sewer pipe acoustic testing, and manhole inspections.

Figure ES-1 shows a map of the smoke and acoustic testing locations and results. Using the data from the Phase 1 and Phase 2 inspections, defects and blockages were identified in the sewer system that could contribute to sanitary sewer overflow (SSO) events. The field effort also identified areas where future SSOs could be prevented. The older portions of the system appear to be a primary source of the defects and source of maintenance requirements. COH should continue to improve the use of inspection results to direct the system maintenance and repairs to prevent future SSOs.



N

0 850 1,700
Feet
1 inch = 1,700 feet

Sewer System
- Manhole
- Historic Overflows

Gravity Mains
- Public Gravity Mains

Force Mains
- Public Force Mains
- Sewer Service Boundary

Accoustic Testing
- No Score
- Poor Condition
- Fair Condition
- Good Condition

Smoke Testing
- Defect Locations
- Smoke Inspection
- A Accoustic Areas of Inspection
- 1 Smoke Areas of Inspection

Flow Meter Basins
- FM 1
- FM 2
- FM 3
- FM 4
- FM 5
- FM 6
- FM 7
- FM 8

Basemap
- Streets
- Interstate
- Railroad
- Hydrology
- Municipal Boundary
- Parks

City of Hendersonville, NC

Overall Inspection Map

Sanitary Sewer Asset Inventory & Assessment

194803

Hendersonville Water Sewer

BLACK & VEATCH

Hendersonville_OverallInspection_11x17_Portrait_Scores January 03, 2019

The following recommendations, which are based on the condition assessment work, are intended to address the identified deficiencies and to maintain or improve the condition of the sewer pipes:

- Conduct closed-circuit television (CCTV) inspection of the segments with severe and moderate defects identified by the smoke testing and the segments with scores of blocked or poor from the acoustic testing and acoustic testing.
- Continue in-house smoke testing in areas identified in the Inspection Plan (discussed in Chapter 2) and as indicated by flow data.
- Complete the manhole inventory and inspection with a concentrated effort in the next year.
- Implement a program to inspect the 16 miles of pipelines in the system with a high likelihood of failure score within the next 3 years. This baseline inspection of these pipelines can be used to measure performance within the collection system in the future. The inspections can be smoke testing, acoustic testing, or CCTV, according to the prioritization of the pipeline. If in-house inspections have been completed of these priority pipelines, the work should have been within the past 5 years.
- Incorporate acoustic testing using the SL-RAT assessment tool used in Phase 2 as part of the inspection procedures.
- Continue to update the Inspection Tracker tool with new inspection data collected in the future.
- Complete the following maintenance needs identified from the lift station inspections performed by COH:
  - Support the slope at lift station 037 Carriage Park.
  - Repair/replace the check valve, update the disconnect, and repair or replace the pump rail system at 003 Garden Lane.

The force mains were not included in this work but should be inspected within the next 5 years to document their condition and determine if repair and replacement are required as part of the capital plan.

HYDRAULIC MODEL CALIBRATION

A hydraulic model of the COH collection system based was developed using Innovyze InfoSewer and COH's geographic information system (GIS). The skeletonized model was inclusive of all pipes with diameters 10 inches and larger. The model was calibrated for both dry and wet weather conditions. The wet weather calibration was performed for four distinct storm events recorded during the spring 2017 monitoring period. The calibration used eight flow meters and three rain gauges. The calibration noted that during large wet weather events, the flow in the 42 inch main line interceptor along Mud Creek backed up and caused flooding due to a restricted flow at the wastewater treatment plant (WWTP).

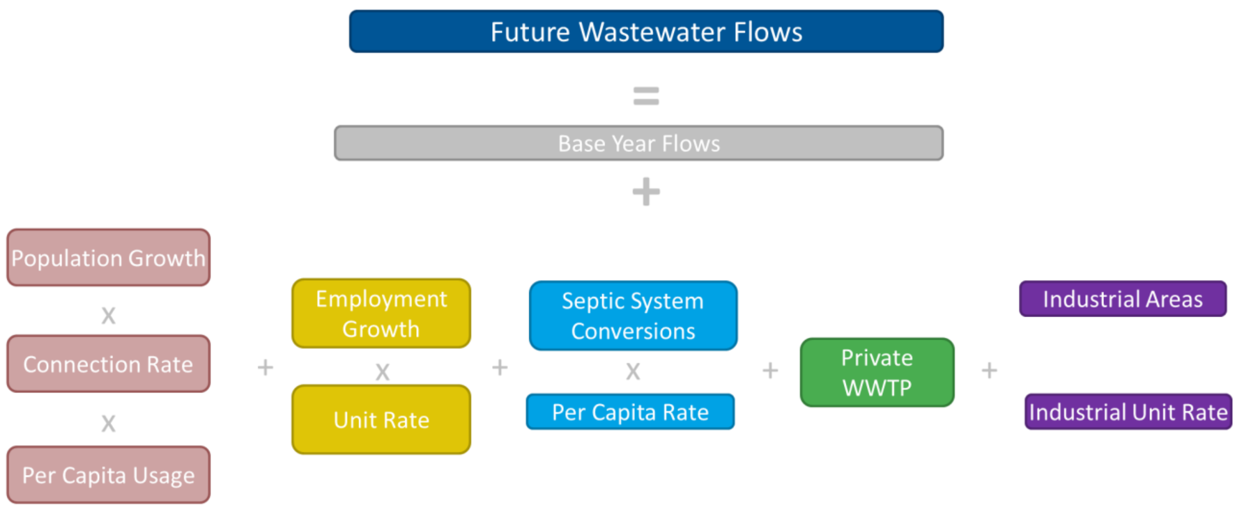
The calibration identified several areas in the older section of the city with higher rates of I/I. Overall, each flowmeter location was calibrated with a moderate to high degree of confidence. With confidence in the model, it is appropriate to use it as a tool for future system planning.

FLOW PROJECTIONS

The flow projections were developed for the base year (2017) and future planning years (2025 and 2040). Base and future flows from the COH service area were calculated using the following data:

- Future population and employment forecasts.
- Base year flow.
- Maximum month peaking factor (MMPF).
- Future I/I rates.
- Failing septic systems.
- Private WWTP flows.
- Interlocal agreement flow capacities.

Future flows can be calculated from the listed data using the following equation:



The 5-year annual average flow to the Hendersonville WWTP served as the base flow. This was 3.07 million gallons per day (MGD). The incremental flow projections for the 2040 service area were added to the base flow to determine the future average annual flows to the WWTP. The projected maximum month flows to the Hendersonville WWTP are based on the 5-year maximum month peaking factor of 1.30. A graphic representation of average flow projections for the WWTP in relationship to the plant's permitted capacity and discharge capacity is shown on Figure ES-2. The Hendersonville WWTP has a 4.8 MGD discharge permit that allows for system upgrades and discharges up to 6.0 MGD. The maximum month projections are shown against the permitted 6.0 MGD on Figure ES-2. The maximum month flow will surpass the plant capacity (4.8 MGD) in 2021 and the discharge permit capacity in 2028. The average flow surpasses the plant capacity (4.8 MGD) in 2030 and the discharge capacity in 2040.

Timing of plant expansions is dictated by the permit capacity and 15A NCAC 02T.0118, often referred to as the 80/90 Rule. The 80/90 rule states that prior to exceeding 80 percent of the wastewater treatment system's permitted hydraulic capacity based on average flow of the last calendar year, an evaluation on meeting future wastewater needs must be submitted to the State. Additionally, at 90 percent plant capacity, final plans and specifications for expansion must be submitted and approved. Based on the 80/90 Rule, COH should be ready to submit an evaluation of their future treatment needs and outline plans going forward by the time the average annual flow exceeds 80% of the permitted treatment capacity (3.84 MGD) in 2022.

However, it can be seen from Figure ES-2 that there is a possibility that the max month flows will exceed the plant capacity by 2021. This is sooner than the 80/90 rule. To reduce the risk of violating the permit during a single month, an expansion of the WWTP is recommended to occur by 2021. The max month flows are projected to exceed the 6.0 MGD discharge capacity by 2028.

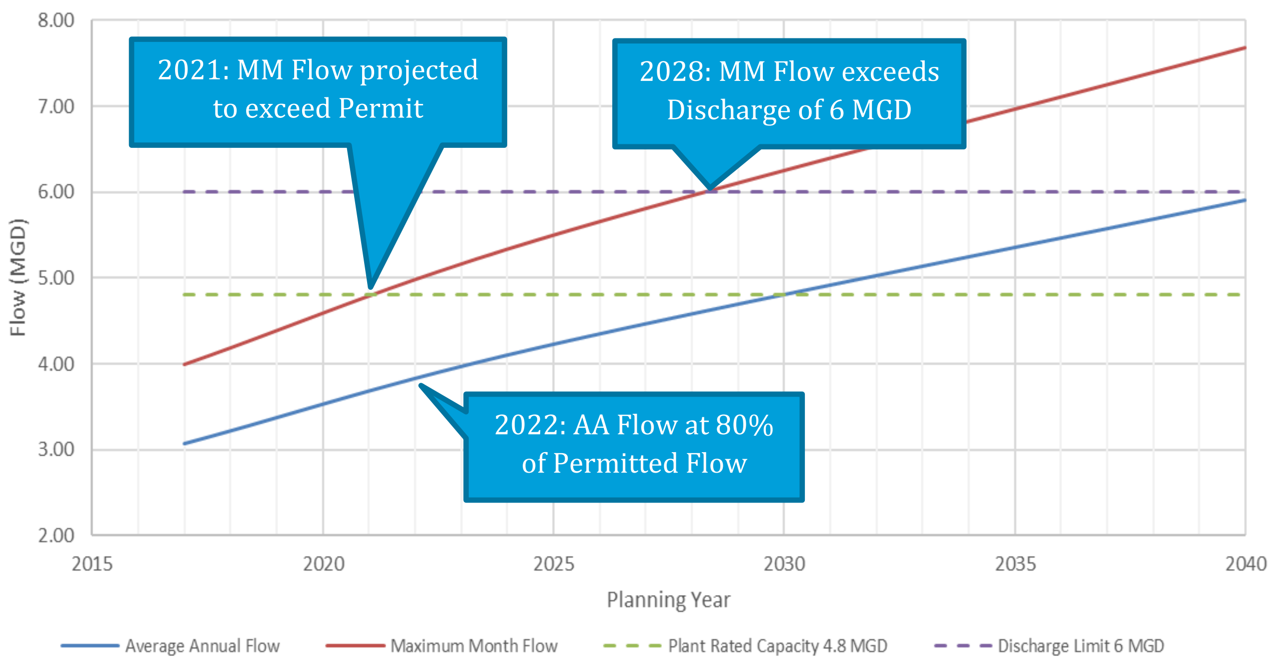


Figure ES-2 Hendersonville WWTP Flow Projections

CAPACITY ASSESSMENT

The flow projections were combined with the hydraulic model to evaluate existing and future collection system capacity. The design storms were evaluated according to the acceptable risk to be used as the design and trigger criteria. COH selected a 2-year return interval, 24-hour duration storm as the criteria that will trigger an improvement and the 10-year storm return interval, 24-hour duration storm to use as the design criteria. The existing collection system was modeled with the projected 2040 flows under the 2-year and 10-year design storms to determine the locations where capacity constraints occurred. Improvements were recommended for any identified capacity constraints.

PRIORITIZATION

The prioritization is an important step in the ranking of CIP projects. The identified improvements were prioritized using a classic risk-based approach. A set of the likelihood of failure (LOF) and consequence of failure (COF) criteria was selected to quantify the relative importance of each pipe segment, which is referred to as the risk. The risk is based on the agreed levels of service and impacts to the social, economics, health, and safety factors. The LOF and COF criteria are shown in Table ES-1.

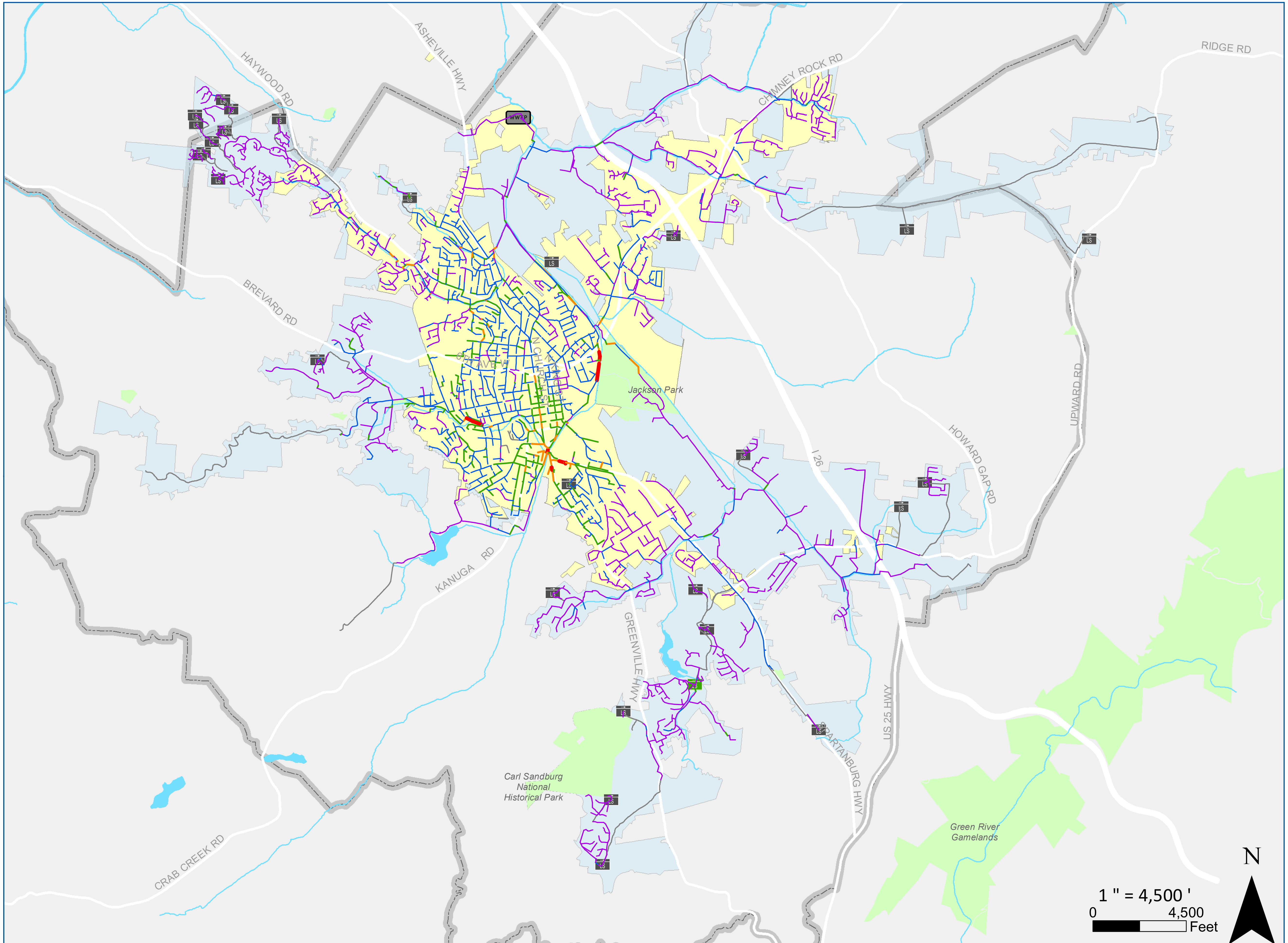
Table ES-1 Likelihood and Consequence of Failure Criteria

LOF	COF
Pipe Age	Alignment
Pipe Material	
Pipe Capacity	
Basin I/I Rate	Diameter
Condition Assessment Score	

All of the pipes were ranked on these criteria. Figure ES-3 is a heat map breakdown of all the COH pipes. The highest COF score (5) is shown in the top row, and the highest LOF scores are in the right most column. Approximately 16 miles of pipe have a high LOF score and are the highest priority for condition assessment. Figure ES-4 shows a map of the risk scores across the system.

COF_Category	1 LOF - Low	2 LOF - M. Low	3 LOF - Medium	4 LOF - M.High	5 LOF - High
5 COF - High	55 pipes, 2.5 Miles	356 pipes, 26.6 Miles	100 pipes, 4.2 Miles	9 pipes, 0.4 Miles	3 pipes, 0.1 Miles
4 COF - M.High	97 pipes, 3.1 Miles	503 pipes, 17.9 Miles	121 pipes, 4.8 Miles	1 pipes, 0.1 Miles	6 pipes, 0.4 Miles
3 COF - Medium	213 pipes, 9 Miles	1283 pipes, 45.2 Miles	138 pipes, 5.2 Miles	2 pipes, 0.1 Miles	11 pipes, 0.4 Miles
2 COF - M. Low	439 pipes, 13.9 Miles	683 pipes, 23.8 Miles	24 pipes, 1.2 Miles	3 pipes, 0.2 Miles	
1 COF - Low	334 pipes, 9.3 Miles	396 pipes, 11.7 Miles	5 pipes, 0.2 Miles		

Figure ES-3 Risk Scores of the Entire COH System



City of Hendersonville
SSAIA
Likelihood of Failure Risk Score Map
Figure ES - 4

Risk Score

- 1 - 5 - Low
- 6 - 10 - Medium Low
- 11 - 15 - Medium
- 16 - 20 - Medium High
- > 20 - High
- City Limits
- Existing Service Boundary
- 2040 Service Area

Hendersonville Water Sewer

BLACK & VEATCH

Template for Figures 6- March 18, 2019

CAPITAL IMPROVEMENT PLAN

The COH Wastewater Collection System 30-year CIP includes 13 capacity driven gravity sewer projects, recommended condition assessment work and recommendations for capacity at the treatment plant. The cash flow is front-loaded by Project G-06, which is a critical linear project on the Mud Creek Interceptor. Figure ES-5 shows the CIP cash flows. The total project cost of all the projects is $74.4 million. The majority of the cost ($43.3 million) are capacity projects while the rest are gravity extension projects and pump station abandonment projects ($31.1 million). A map of the improvement locations is shown in Figure ES-6.

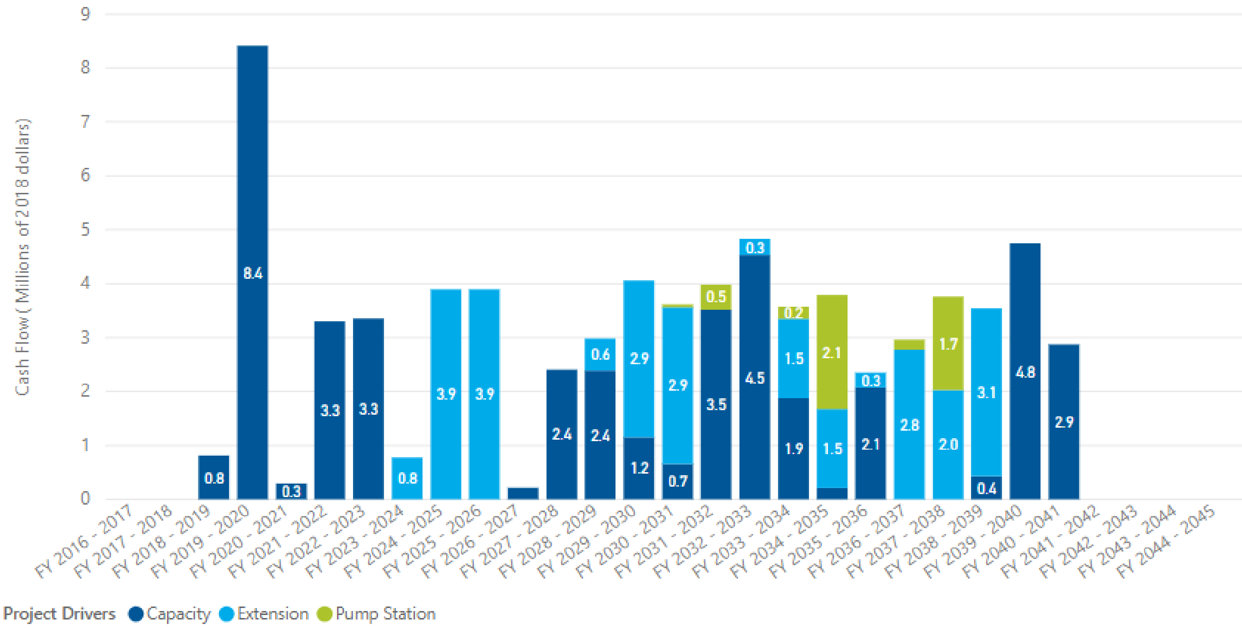
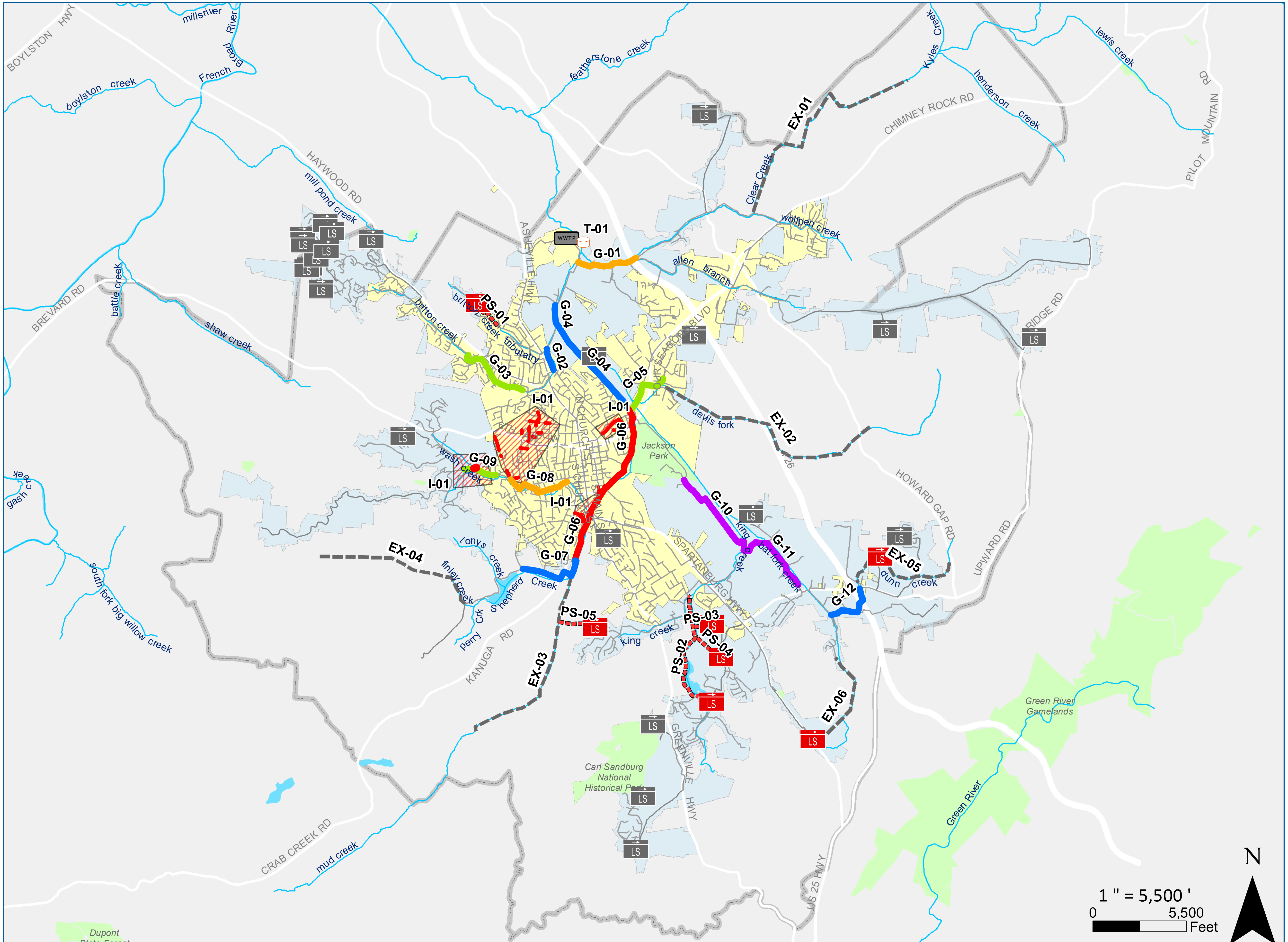


Figure ES-5 CIP Cash Flow



City of Hendersonville
SSAIA

Recommended
Improvement Projects
Figure ES-6

- EQ Tank
- Pump Station
- Treatment Plant
- Abandoned Pump Station
- Pump Station Abandonment
- Critical Projects (2019)
- 2020 - 2025
- 2026 - 2030
- 2031 - 2035
- 2036 -2040
- CCTV Investigation
- Gravity Extensions
- COH Collection System
- City Limits
- Existing Service Boundary
- 2040 Service Area

Hendersonville Water Sewer

BLACK & VEATCH

1.0 Introduction

The COH owns and operates sewer collection and treatment for Hendersonville as well as the town of Laurel Park and Village of Flat Rock and unincorporated portions of Henderson County. The collection system consists of approximately 160.4 miles of sanitary sewer, 29 sewer lift stations, 20.4 miles of force main, and 4,700 manholes. The City operates a 4.8 MGD WWTP that discharges to Mud Creek, a tributary of the French Broad River. A map of the system is shown on Figure 1-1.

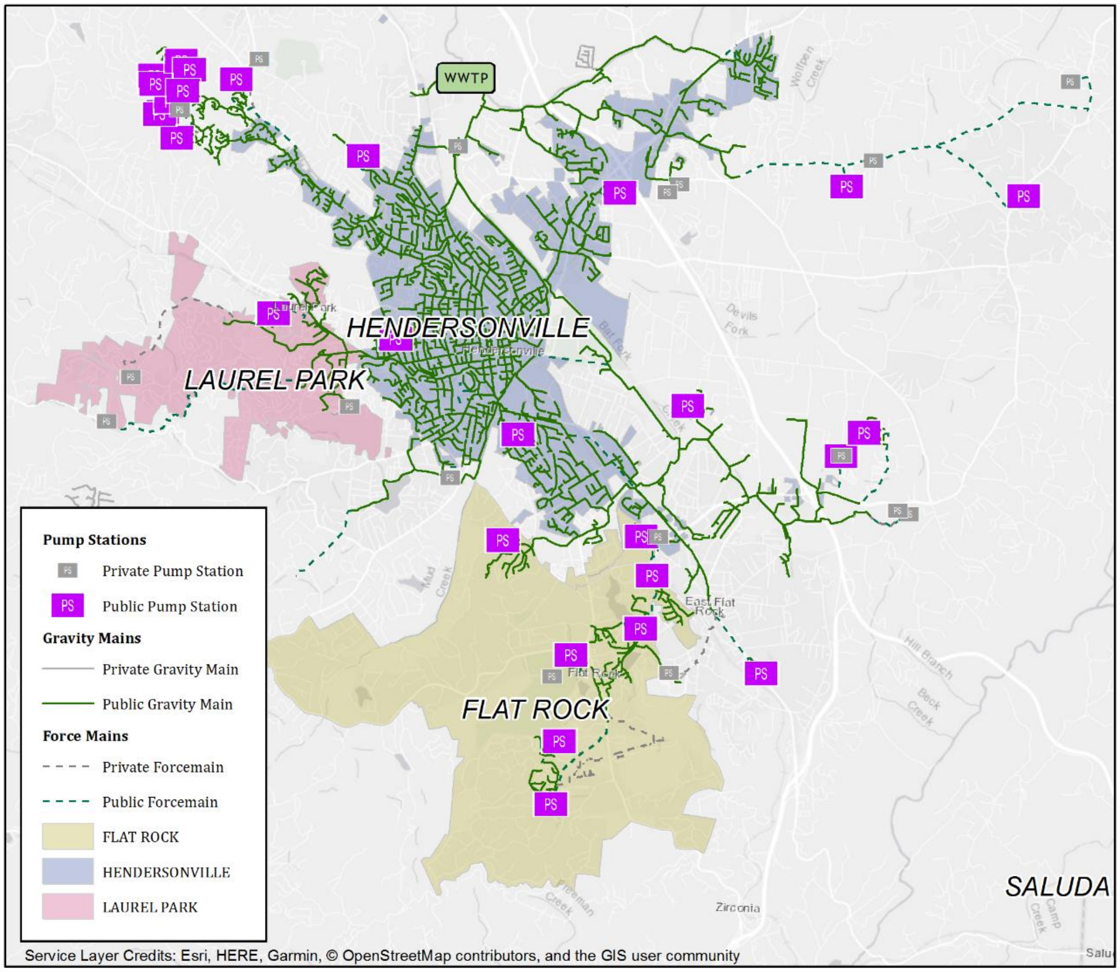


Figure 1-1 City of Hendersonville Collection System

In January of 2017, COH and Black & Veatch initiated the SSAIA. COH's main goals for the project included the following:

- A well-documented, forward-looking master plan.
- An overall assessment of the condition of the sewer system, as well as guidance for future repairs and maintenance.
- Prioritization of the recommended improvements to direct investments to the most significant projects.
- An expanded GIS database with more complete data.
- An interactive, easy-to-use planning tool.

The project was completed in two Phases. Phase 1 was a concentrated effort to gather data on the current state of the system, including condition and available capacity. The primary objectives of Phase 1 were to perform condition assessments of the existing system, as well as to develop and calibrate an InfoSewer hydraulic model of the collection system to evaluate capacity. In addition, Black & Veatch worked with COH to update the GIS schema and migrate its database to ESRI's local government information model. Phase 2 focused on planning for the future while continuing to collect system condition data. The main objectives for Phase 2 were to develop performance criteria and level of service goals for the COH's collection system, develop wastewater flow projections, and recommend collection system improvements in a prioritized and dynamic CIP. The master plan CIP is intended to (1) meet current and projected loadings through the 2040 planning horizon, (2) compile a detailed list of the sewer system assets and their condition, and (3) provide tools to continue monitoring and evaluating the system performance and expansion.

Various system evaluations, analyses, and assessments were conducted to meet the stated project objectives. The results of this work are detailed in this SSAIA report organized into the following chapters:

- Chapter 1: Introduction
- Chapter 2: Condition Assessment
- Chapter 3: Model Update and Calibration
- Chapter 4: Flow Projections
- Chapter 5: Capacity Assessment
- Chapter 6: Project Prioritization
- Chapter 7: Capital Improvement Plan and Recommendations

2.0 Condition Assessment

One of the objectives of this SSAIA was to perform condition assessments of the existing system and provide guidance for future maintenance and repair of COH's assets. This sewer condition assessment work consisted of a review of available information as well as field inspections to collect information on the current conditions of the pipes and manholes in the system. The results of these inspections were used to estimate the I/I into the system and to locate possible blockages and structural defects to evaluate the overall condition of the system. If necessary, capital and maintenance projects were proposed for areas with blockages or structural defects. These projects were included in the master plan recommendation or the CIP. More detailed inspections will be required in the future to develop other specific capital projects, but these inspections provided useful information in describing the general condition of the system.

2.1 SUMMARY OF WORK COMPLETED

The condition assessment work was completed in two phases. Phase 1 included both desktop evaluation and field investigation. The desktop evaluation included the review of existing GIS data, reports from previous inspections including CCTV, and locations of previous SSO. The Phase 1 fieldwork included smoke testing several areas in the oldest sections of Hendersonville and lift station visits.

Prior to starting the fieldwork, the Black & Veatch team set up a manhole inventory and inspection procedure in the "Inspection Tracker" database with the staff at COH. The Inspection Tracker database was built in Microsoft Access and was used to organize fieldwork completed prior to this project and the work completed under the SSAIA. Black & Veatch reviewed results from previous lift station inspections before visiting a few stations in the field for additional evaluations.

Phase 2 expanded upon Phase 1 work with acoustic testing of the pipelines and additional manhole inventory and inspections.

2.2 BACKGROUND

According to the GIS data provided, the COH's collection system consists of approximately 160.4 miles of sanitary sewer, 29 sewer lift stations, 20.4 miles of force main, approximately 4,700 manholes, over 9,500 service connections; the system is a tributary to a 4.8 MGD WWTP.

2.2.1 Inspection Methods

The COH has an ongoing inspection program that includes smoke testing, CCTV inspection, and a recently implemented comprehensive manhole inventory and inspection program. These inspection methods provide additional information that was used to evaluate and assess the condition of the collection system. The COH's recent CCTV inspection data were added to the Inspection Tracker database. Inspections completed as part of Phase 1 and Phase 2 of the SSAIA project were added to the final database to be delivered to COH at the end of this project.

Fieldwork during Phase 1 and Phase 2 included smoke testing, lift station visits, pipeline acoustic testing, and manhole inspections.

2.2.2 Inspection Planning

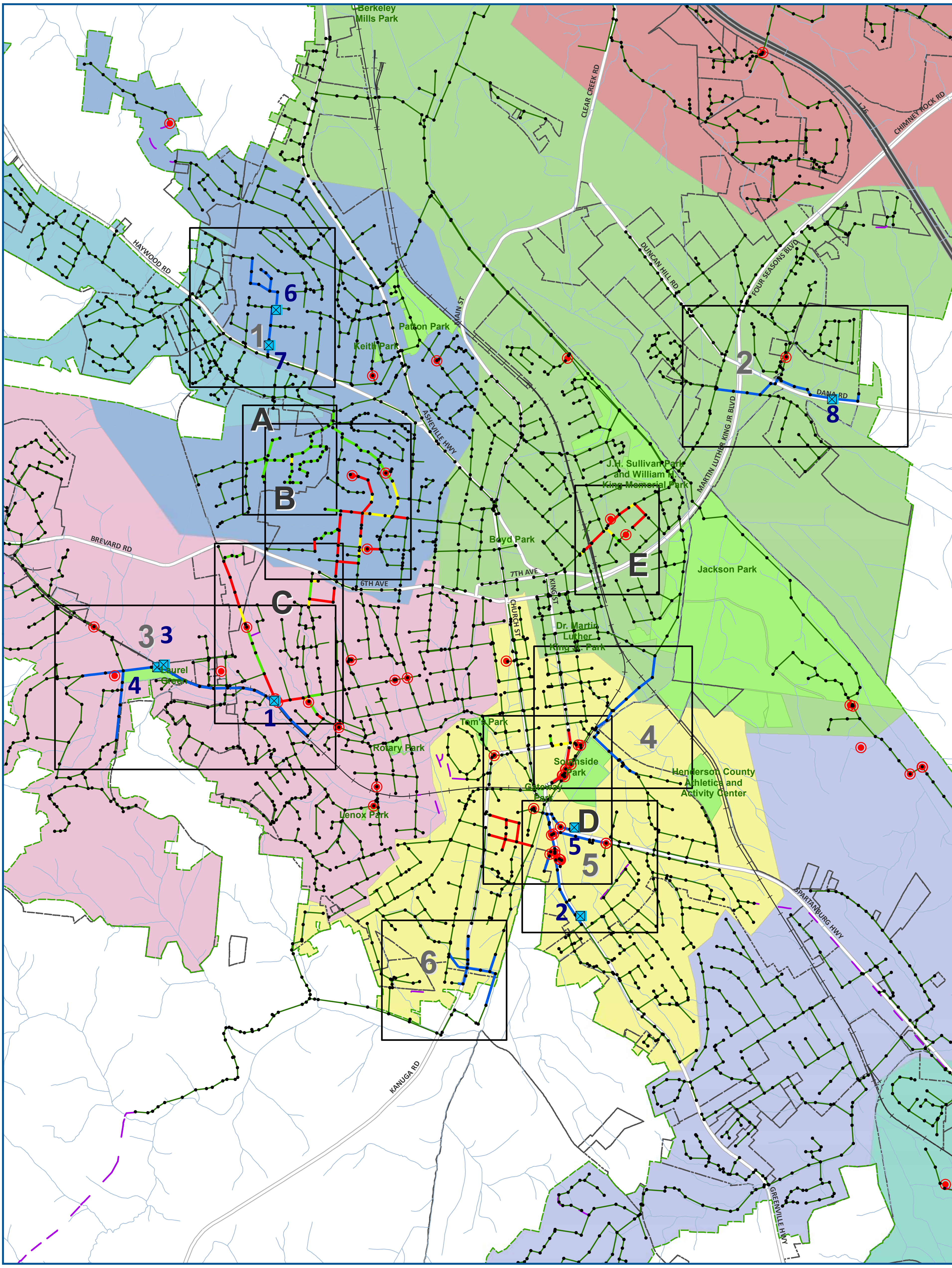
The work required for gathering the data from each inspection method was described in an Inspection Plan that was developed and approved prior to the beginning of the work. The plan served as a basis for coordinating the work with the subcontractors and included maps showing the segments planned to be inspected by each method. The complete Inspection Plan for Phase 1 smoke testing is included as Appendix E and the Inspection Plan for the Phase 2 and overall collection system is included as Appendix H.

The plan included contact information for the COH, Black & Veatch, and the contractor, Frazier Engineering, to provide for open lines of communication. The plan addressed notification of the public and police, fire and safety concerns, and guidance to mitigate hazards.

During the implementation of the work, the plan was modified in the field depending on the ability to locate or not locate some manholes, and some access was restricted by overgrown areas. In addition, some segments were deleted from the inspection because they had recently been installed and were not likely to have defects.

2.2.3 Overview of Inspection Locations

The factors used in selecting segments to inspect included recent flow metering results, locations of SSO events, creek crossings, the proximity to storm drainage, previous overflows, and experience with the various pipe materials. In addition, the locations of previous smoke testing were reviewed to avoid areas that had recently been tested. The results from the COH's CCTV inspection and SSO events were used to identify possible areas for smoke testing in Phase 1 and acoustic testing in Phase 2. These locations are shown on Figure 2-1. Discussions with the COH's Operations staff were used to refine the areas for inspections. The inspection areas were finalized using this information, and the specific details on each area are in the inspection plans.



Sewer System

- Manhole
- Historic Overflows

Gravity Mains

- Public Gravity Mains

Force Mains

- Public Force Mains
- Sewer Service Boundary

Accoustic Testing

- No Score
- Poor Condition
- Fair Condition
- Good Condition

Smoke Testing

- Defect Locations
- Smoke Inspection
- A Accoustic Areas of Inspection
- 1 Smoke Areas of Inspection

Flow Meter Basins

- FM 1
- FM 2
- FM 3
- FM 4
- FM 5
- FM 6
- FM 7
- FM 8

Basemap

- Streets
- Interstate
- Railroad
- Hydrology
- Municipal Boundary
- Parks

0 850 1,700
Feet
1 inch = 1,700 feet

City of Hendersonville, NC

Overall Inspection Map

Sanitary Sewer Asset Inventory & Assessment

194803

Hendersonville Water Sewer

BLACK & VEATCH

Hendersonville_OverallInspection_11x17_Portrait_Scores January 03, 2019

2.3 SMOKE TESTING RESULTS

The purpose of smoke testing is to identify cross connections in the sewer pipe or other defects that allow I/I by forcing smoke into the pipe. A blower is used to seal a manhole and force the smoke into the pipe as shown on Figure 2-2. Smoke is then forced out of the pipe at cross connections with storm drains or cracks in the pipe joints or wall. The best results are obtained when the soil surrounding the pipe is dry because it will allow the smoke to surface through the voids or cracks in the ground. The ground conditions during testing were not the most ideal because of recent rain events. However, infiltration from the rain events did not saturate the ground to prevent the smoke from identifying sources of potential inflow.

Figure 2-2 Smoke Testing Blower System

In the areas affected by the smoke testing, the public was notified using door hangers distributed by Frazier Engineering two days prior to the smoke testing work. Black & Veatch provided a list of property owners' names and addresses from the GIS data around the inspection areas for Frazier Engineering to use in contacting the property owners and informing them of the work in the area. Frazier Engineering contacted the local fire and police departments through the non-emergency dispatch to inform them of the work on a daily basis; the Deputy Fire Chief was also directly contacted each day of the inspection as needed.

The City staff aided with locating manholes and identification of access during the inspection. During Phase 1, smoke testing was performed on 20,000 feet of sewer, as shown in the Inspection Plan. The smoke testing areas and pipe segments are shown on Figure 2-1. Detailed maps of each location are included in Appendix E.

The results of the smoke testing identified areas of potential inflow. Figure 2-3 shows a typical location of smoke from a defect. There are no published standards to rate smoke testing results but based upon Black & Veatch's extensive experience assessing such results, the defects were rated severe, moderate, or light. The rankings were given to each defect based to the amount of inflow estimated from the location of the defect, estimated defect opening size, and type of defect.

Severe defects were those located in the drainage path in such a way that inflow is likely and the opening from the defect would allow water to easily enter the pipe. This type of defect could be an open lateral cleanout or a possible broken pipe with a cross connection to a storm drain.

Moderate defects were located close to a drainage path that has the potential for high flows to enter, and the opening could be a cracked cleanout or broken pipe. This type of defect is typically a missing cleanout cap or indicated by smoke surfacing along the pipe alignment by a storm drain.

Light defects were located anywhere along the pipeline where smoke was detected, but they were not in a location that would potentially allow inflow. Light defects are typically broken service cleanouts or those with cracks in the lids and are not in the path of surface runoff.

Figure 2-3 Potential Inflow Locations can be identified by Smoke Testing

The results of the smoke testing identified eight defects. Three defects were rated severe, three rated moderate, and two rated light. The locations and identification of each defect are shown in Table 2-1. The sketch of the location and complete results from the field testing are attached in Appendix F. The locations of the defects are also shown on Figure 2-1.

Table 2-1 Smoke Testing Defect Results

SMOKE SKETCH NUMBER (REFER TO APPENDIX F)	EASTING	NORTHING	MANHOLE	DOWNSTREAM MANHOLE	ADDRESS	DEFECT TYPE	INFLOW POTENTIAL	PHOTO NUMBER	COMMENTS
1	963392.8	587075.1	MH-183	MH-182	175' Downstream of MH-183	Storm Drain	Severe	111-1068/ 111-1069	Heavy smoke from storm drain catch basin and storm pipe.
2	968831.7	583257.5	MH-532	MH-2048	110A Greenville Highway	Service Cleanout	Light	111-1072	4" polyvinyl chloride (PVC) cleanout cap and insert missing at grade.
3	961314.1	587668.7	MH-1726	MH-1727	120' Downstream of MH-1726	Mainline Quick Entry	Severe	111-1065	Heavy smoke from the mainline sewer at the creek. Vitrified Clay Pipe (VCP) aerial.
4	961424.8	587716.9	MH-1727	MH-1731	66' Downstream of MH-1727	Mainline Multiple	Severe	111-1066	Heavy smoke from multiple sinkholes over the mainline sewer. The line being crushed by railroad tracks.
5	968726.1	584828.8	MH-1763	MH-1764	610 Spartanburg Highway	Service Cleanout	Moderate	111-1071	Two 4" PVC services are open and exposed 2' below grade.
6	963294.9	593386.7	MH-2309	MH-2310	204A Morris Lane	Service Cleanout	Light	111-1061	4" PVC cleanout standpipe broken 12" below grade in vault. Light inflow potential.
7	963428	594016.4	MH-2311	MH-2307	220 Morris Lane	Service Cleanout	Moderate	111-1060	4" PVC cleanout cap missing 4" below grade near storm ditch.
8	973299.8	592430.5	MH-3917	MH-3835	216 Dana Road	Service Cleanout	Moderate	111-1062	4" PVC cleanout cap and insert missing 1" above grade in low lying area.

The severe defect between MH 182 and 183 is shown on Figure 2-4. The defect is a potential cross connection between the storm drain and the collection system.

Figure 2-4 Severe Defect between MH 182 and MH 183

The severe defect between MH 1726 and MH 1727 is shown on Figure 2-5. This defect is located in the creek crossing and has the potential to be a major source of inflow into the collection system.

Figure 2-5 Severe Defect between MH 1726 and MH 1727

The severe defect between MH 1727 and MH 1731 is shown on Figure 2-6. The defect is located along the railroad tracks and is a potential source of inflow directly into the system.

Figure 2-6 Severe Defect between MH 1727 and MH 1731

2.3.1 Smoke Testing Observations

The 20,000 feet of pipeline inspected with smoke testing is a small percentage of the total length of pipe in the system. These testing locations were selected in areas of older pipes where flow metering indicated higher rates of I/I. Three severe defects were detected within a small percentage of pipes which can indicate that more defects were in the COH system. Although, these areas might have higher rates of defects than newer areas with less I/I, it is still important to create an inspection program to maintain those low I/I rates throughout the COH system. Therefore, it is recommended that COH continue its in-house smoke testing investigations by selecting pipes in high priority areas, notifying customers and completing field work. This work should be planned for the dryer summer months when smoke testing is more effective. In areas where the defects were found and could not be resolved, further CCTV investigation should be conducted.

The smoke testing was also effective in locating defects that impact the flow capacity of the system. The eight defects identified in Table 2-1 in Section 2.3 indicate a significant potential for additional defects in the system. The detailed results of the smoke testing are included in Appendix F.

2.4 LIFT STATION VISIT RESULTS

The COH operates and maintains 29 lift stations as part of the collection system. All lift stations consist of submersible pumps in a wetwell and range in age from 3 to 40 years. COH initiated an inspection program of the lift stations in 2016 and inspected all the stations in 2017. The inspection included a pump test to validate the operation of the pumps. The results of the COH lift station inspections are included in Appendix G. The COH continues to inspect and maintain the lift stations on a regular basis.

Black & Veatch reviewed the results of the 2016 and 2017 routine COH inspections. The inspections identified the following defects:

- 003 - Garden Lane: The check valve was rusting and should be repaired or replaced. In addition, the vault does not appear to drain properly. The rail system for the pump appeared to be rusting and should be replaced or repaired.

In February 2017, as part of the Phase 1 field inspections, Black & Veatch visited lift stations 011, 012, and 019 for general observations. The COH assisted in these observations and opened the wetwell so the piping, control floats, and site conditions could be observed. According to the staff, the piping in lift station 012 is typical for all the lift stations and is shown on Figure 2-7.

Figure 2-7 Piping at Lift Station 012

As part of the Phase 2 fieldwork, seven additional lift stations were reviewed in June 2018. The seven stations reviewed included those with defects identified from the review of the COH inspection results. The lift stations reviewed were 003, 008, 016, 018, 024, 037, and 038. Based upon the visual inspections of the lift stations, there were no major defects observed. The following observations were made during the field visit:

- 003 – Garden Lane: The wetwell is elevated and difficult to access with only a ladder on the side of the wetwell. There is no generator, and the disconnect has not been updated.
- 008 – Browning Avenue: The valve vault was not accessible. Therefore, it was not possible to verify the condition of the piping and valves.
- 016 – Kenmure Driving Range: The lift station has a rain gauge that can be used to correlate rainfall with the wetwell levels. The drain from the wetwell allowed grease into the valve vault.
- 018 – Kenmure Brookwood: The wetwell is fiberglass with no valve vault.
- 024 – Shaws Creek Farm: The discharge pipe is galvanized steel, which has a potential for corrosion depending on the soil characteristics. There was erosion on the road leading to the station.
- 037 – Carriage Park: The wetwell piping appeared to have some corrosion, and the holding tanks have the potential for odor concerns. The hillside was observed to be sliding into the fence and was pressing on the gas meter for the generator, as shown on Figure 2-8. The bank above the station is undercut from the slope sliding down.
- 038 – Carriage West: There was no generator, but the disconnects appeared to be upgraded. There is no valve vault, so the piping was not visible.

Figure 2-8 Slope Sliding at Lift Station 037

The force mains from the lift stations range from 2 inch galvanized to an 8 inch ductile iron pipe. There is one 4 inch PVC force main. The force mains are not included in the inspection because they are not visible past the discharge pumping.

2.4.1 Lift Stations Summary

The ten lift stations observed in April 2017 and June 2018 did not reveal any visible defects or issues that would indicate the potential for failure of the piping or wetwell. The major defects identified in the regular inspection of the lift stations include installation of valve vaults, updates to SCADA systems and alarms should be included in capital projects. Therefore, capital projects that address these defects were included in Section 7. It is recommended that COH continue its program of regular inspections to identify, maintain, and address any serious potential issues.

The force mains were not included in this inspection but should be considered for future assessment work within the next 5 years.

2.5 ACOUSTIC TESTING RESULTS

In Phase 2, Frazier Engineering conducted a “pilot” of the acoustic technology SL-RAT (sewer line rapid assessment tool) that utilizes acoustic technology to quickly assess the degree of blockage in a sewer line. An acoustic transmitter is located in one manhole, and a receiver is located in an adjacent manhole as depicted on Figure 2-9.The sound wave propagates in the air gap above the wastewater flow up to 800 feet. The strength of the received signal serves as an indication of the percent of blockage and can be measured in less than 3 minutes. The results from the acoustic testing are reported in a color-coded rating system from 0 to 9, with 0 being a total blockage and 9 being no blockage.

Figure 2-9 Typical Setup of SL-RAT Inspection

SL-RAT technology was used to inspect 24,200 feet of sewer from various areas identified in the Inspection Plan between September 24 and 27, 2018. The inspection locations are shown on Figure 2-1. The complete results are in Appendix F. The work was conducted with 134 setups at individual manholes. The test is based on the acoustic monitoring and the scoring is shown in Table 2-2.

Table 2-2 Acoustic Test Scoring

PIPE SEGMENT CONDITION	SCORE RANGE
Good	7 to 10
Fair	4 to 6
Poor	1 to 3
Blocked	0

The results of the testing indicate the following shown on Figure 2-10.

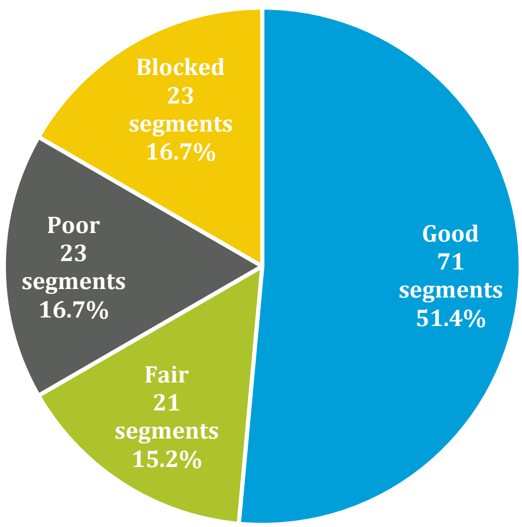


Figure 2-10 Acoustic Testing Results

The indication that a line is blocked or receives a "poor" score can be caused by several factors, including a sag in the line, surcharging at the time of the test, the buildup of debris, or roots blocking the line, which can be confirmed via CCTV investigations.

2.5.1 Acoustic Testing Observations

The acoustic testing identified that 33 percent of the inspected pipes were in "poor" or blocked condition. This is a high percentage but is not necessarily an indication of the total system since these pipes were selected on the basis of suspected or possible defects. This is an indication that these areas should be scheduled for CCTV inspection to validate the cause of the defect and identify corrective action.

2.6 MANHOLE INSPECTION RESULTS

The COH developed an "in-house" inspection form that was used to efficiently gather the data on the manholes. All data from the inspection forms were recorded in the Inspection Tracker database. Based on the total number of manholes inspected it appears the plans to collect data on manholes as part of routine maintenance has not produced many inspections. The focused efforts have been able to collect data on about 200 manholes. According to the focused efforts, the average rate of collecting data is about 10 minutes per manhole. At this rate, the completion of the inspection and inventory of the manholes would require about 125 additional days, at 6 hours per day, with a two-man crew, to inspect the remaining manholes in the system.

The manhole inspection rates the overall condition and provides condition assessments on the various components. The depth to the invert is measured for all connections to the manhole. Photographs document the condition and allow for later review. The minimum information included on photographs is as follows:

- Manhole number on the whiteboard.
- Manhole location (in the street or easement).
- Manhole ring, lid, and cover.
- Manhole cone and invert.

The photographs are saved on the server in a folder for manholes. The inspection form is submitted to Engineering so the data can be entered into the GIS database and Inspection Tracker, and GPS locating can be completed.

2.6.1 Manhole Inspection and Inventory

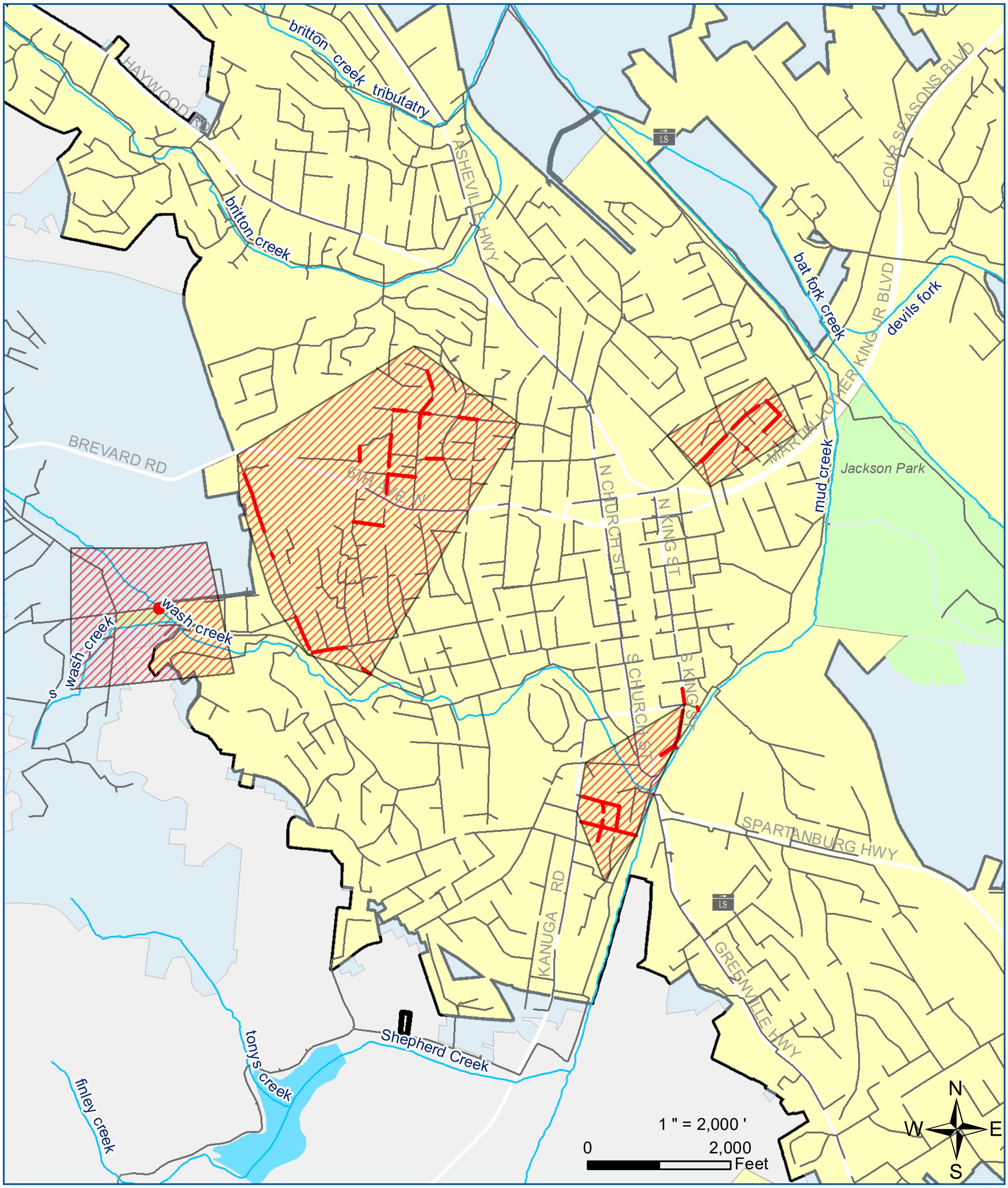
The inspection and inventory of the manholes is a significant undertaking and will require a diligent effort over an extended period of time. The operations staff currently collects the data on manholes associated with other work orders when possible. However, this results in a few inspections being completed.

For the manhole data to be gathered in a condensed period of time, a concentrated effort would be required. At the time of writing this master plan, Black & Veatch had assisted COH with the inspection of 122 manholes.

2.7 DISCUSSION OF FIELD INSPECTION RESULTS

Through Phase 1 and Phase 2, approximately 8.3 miles, or five percent, of the COH's collection system had been inspected. The results of Phase 1 and Phase 2 inspections indicate the collection system has areas with the potential for blockage or high I/I that could create overflows. The routine maintenance program continues to make progress in addressing these areas as they are identified.

Through the inspection work conducted for this project, areas have been identified where future SSOs could be prevented. The older portions of the system appear to be a primary source of the defects and source of maintenance requirements. The high percentage of defects found in the inspection results indicates that these areas should continue to be inspected and maintenance conducted. Pipes recommended for CCTV based on defects detected with the smoke and acoustic testing are shown below in Figure 2-11. The COH should continue to improve the use of inspection results to direct the work of maintenance to prevent SSOs in the future. The continued use of inspections and prioritization of the work supports the progress the COH is making toward reducing the number of SSOs. A collection system prioritization was completed as part of the SSAIA and is presented in Section 6 of this report. The future inspection data should also be used to identify segments for replacement or rehabilitation in future capital improvement planning. Repairs to defects and blockages identified will be required for the COH to continue to reduce SSOs and I/I.



City of Hendersonville
SSAIA

Mud Creek Basin
Addtional Investigation Areas
Figure 2-11

CCTV Investigation
Smoke Inspection Defects
Accoustic Testing Defects
City Limits
Existing Service Boundary

Hendersonville Water Sewer

BLACK & VEATCH

2.8 RECOMMENDATIONS

Recommendations derived from Phase 1 and Phase 2 work are to address the deficiencies noted during the inspections and to maintain or improve the condition of the piping. Continued inspection and repair programs are also recommended in order to maintain low rates of I/I throughout the COH system. The following recommendations are made:

- Conduct CCTV inspection of the segments with severe and moderate defects identified by the smoke testing and the segments with scores of blocked or poor condition from the acoustic testing and enter the data into GIS.
- Continue to update the GIS with information from the field to develop more accurate maps.
- Continue in-house smoke testing in areas identified in the Inspection Plan and as indicated by flow data.
- Complete the manhole inventory and inspection with a concentrated effort in the next year.
- Incorporate acoustic testing using SL-RAT used in Phase 2 as part of the inspection procedures.
- Continue to update the Inspection Tracker tool with new inspection data collected in the future.
- Complete the following maintenance needs identified in the lift station inspections performed by COH:
  - Support the slope at lift station 037 Carriage Park.
  - Repair or replace the check valve, update the disconnect, and repair or replace the pump rail system at 003 Garden Lane.
  - Continue to make repairs based upon regular inspections as shown in Appendix G.

The force mains were not included in this work but should be inspected within the next 5 years to document their condition and determine if repair and replacement are required as part of the capital plan. A force main inspection plan would include the following:

- Develop an inventory of the pipe material, age, diameter, and length from the GIS.
- Prioritize the force mains using a risk analysis approach that uses the likelihood of failure multiplied by the consequence of failure to create a risk-based ranking. A preliminary ranking is included in Section 6.
- Identify inspection technologies (i.e. leak detection, ultrasonic testing for wall thickness, or electromagnetic testing) for gathering data on the condition of the force mains.
- Conduct inspections of the force mains according to the prioritized rankings. The higher ranking force mains would be inspected in more detail than the lower ranking force mains.

3.0 Model Development and Calibration

3.1 CITY OF HENDERSONVILLE SEWER SYSTEM

The COH is located in western North Carolina in Henderson County. The COH's sewer system serves Hendersonville, Laurel Park, Flat Rock, and some unincorporated areas of Henderson County. The sanitary sewer collection system consists of approximately 29 pump stations, 160.4 miles of gravity sewer, 20.4 miles of force main, and 4,700 manholes. Figure 3-1 illustrates the configuration of the Hendersonville Sewer System.

The collection system was modeled using Innovyze's InfoSewer software version 7.6 to assess the system's hydraulic capacity. The model network was constructed using a combination of GIS data, survey data, and as-built drawings. The model included all 10-inch and larger sewers. In addition, critical 8-inch pipes were included that were relevant for connectivity in the model or acted as trunk line sewers. A "skeletonized" model is appropriate for sewer system evaluations as most 8-inch collector sewers have more than adequate capacity. Though a smaller size would be sufficient from a capacity standpoint, an 8-inch sewer is usually used as a local collector to prevent localized problems with blockages from affecting individual customers. Field evaluations, like the smoke testing and acoustic testing described in Section 2, are the best method for maintaining the level of service on 8-inch sewers. The Bonclarken pump station was the only pump station and force main included in the hydraulic model since it contributes significant flows and was connected to larger diameter upstream sewers.

The model was calibrated using field data collected in the spring of 2017. Model calibration is necessary to verify that the tool replicates field conditions optimally. Peak flows during wet weather events drive the sizing of sewers to prevent SSOs. In a sanitary system, the rainfall-dependent inflow and infiltration (RDII) is driven by a myriad of factors including the following:

- Age and condition of the system.
- Construction practices at the time of installation.
- Prevalence of direct (illicit) stormwater connections to the sanitary system.
- Maintenance of the system.
- Antecedent moisture conditions (the saturation of the ground around the sewers).
- Groundwater elevation.

Model calibration ensures that the model produces an accurate representation of how all the above factors combined affect the rate and volume of RDII. An accurate model is a powerful tool for determining current and future capacity constraints, predicting SSOs, and identifying capital improvements.

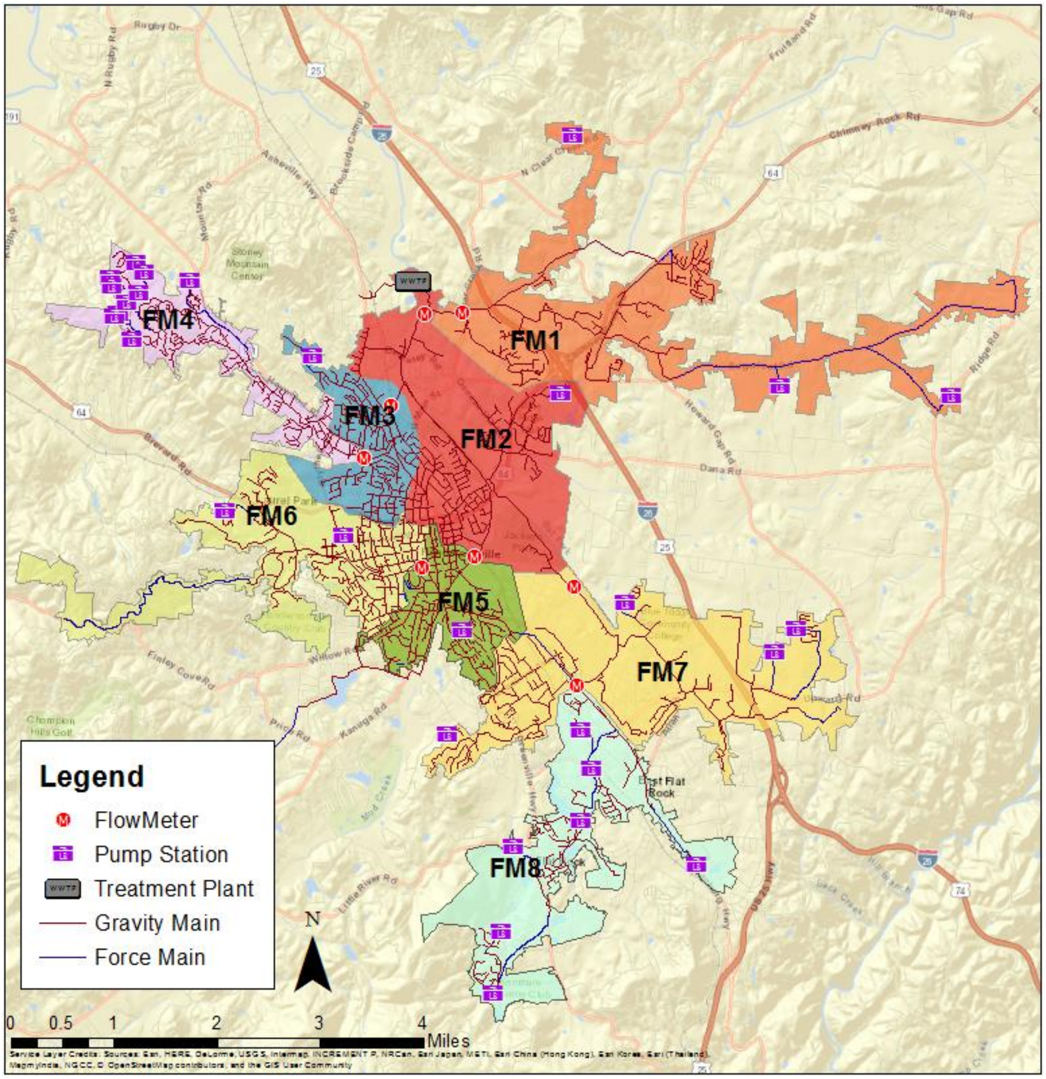


Figure 3-1 City of Hendersonville's Sewer System

The system was divided into eight sub-basins as shown on Figure 3-1 and Figure 3-2. The eight sub-basins are FM1 (orange), FM2 (red), FM3 (blue), FM4 (purple), FM5 (green), FM6 (light green), FM7 (yellow), and FM8 (light blue). The meters were placed to capture all flows in the COH system. Meter locations were selected to capture flows from major drainage basins and to break up the system into similarly sized catchment areas. The temporary meters installed by Frazier Engineering were Sigma 920 meters with submerged area-velocity sensors. Additional information about the meter installation is included in the Flow Monitoring Report (Appendix J) by Frazier Engineering. All eight meters were used in the dry and wet weather calibration.

One unmetered 10-inch sewer is located on the northwest of the WWTP. The unmetered 10-inch sewer serves a couple of businesses on Asheville Hwy and receives pumped flow from an Industrial Park in the County's system. This drainage area was assigned the "WWTP" settings in the model which used dry weather and wet weather parameters using the average of the eight meter sub-basins. The manhole location for each meter is shown in Table 3-1.

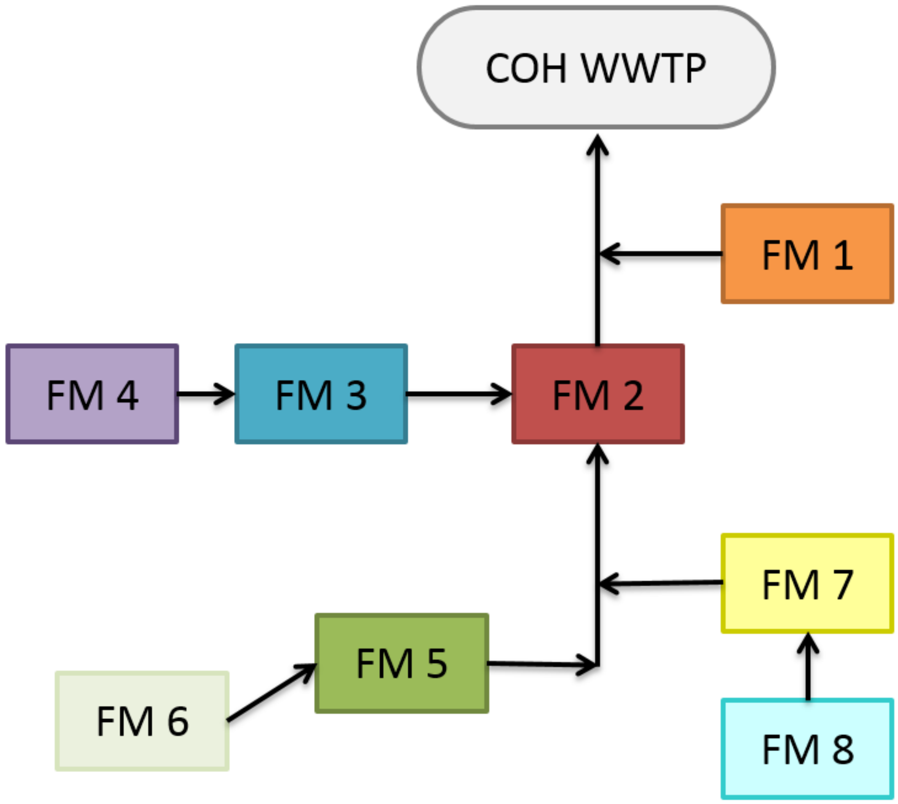


Figure 3-2 Hendersonville Sewer Basin Flow Schematic

Table 3-1 Flowmeter Manhole Locations

FLOWMETER	MANHOLE	PIPE DIAMETER (IN.)
FM1	MH-3836	18
FM2	MH-196	42
FM3	MH-2008	24
FM4	MH-1476	12
FM5	MH-2278	24
FM6	MH-917	18
FM7	MH-2773	24
FM8	MH-3792	18

3.2 DATA COLLECTION AND ASSUMPTIONS

Table 3-2 lists the data sources used and assumptions made to build the hydraulic model.

Table 3-2 Data Sources and Assumptions

MODEL INPUT	SOURCE / ASSUMPTION
Hydraulic Model Base	GIS database dated March 28, 2017.
Manhole/Pipe Inverts	GIS data and survey data received July 13, 2017.
Total Water Consumption (TWC)	The TWC for each sub-basin was determined from the geocoded water consumption for December 2016-February 2017. Average indoor water consumption is used to allocate tributary flows within a meter basin, however, the actual total flows will be based on the wastewater flow metering. Winter is used to capture average usage without the impact of irrigation.
Base Sanitary Flow (BSF)	Initially, the BSF was assumed to be 80% of the TWC and was adjusted to calculate an appropriate groundwater infiltration flow.
Groundwater Infiltration (GWI)	GWI was determined from the flowmeter data during the dry weather period (March 2, 2017-March 9, 2017) and TWC provided via GIS. NOTE: The GWI cannot be larger than the minimum nighttime flows during dry weather.
Contributing Area	Contributing area was developed using a 200 foot buffer around the existing sewers to estimate the area where rainfall depths will impact the sewer system.

3.2.1 Flowmeter Data Review

Gravity flowmeters measure the flow depth and the average velocity. The flow is then calculated via the continuity equation. The recorded depth and velocities can be graphed against each other to develop a “scatter” plot. The resulting graph should yield an increasing relationship consistent with the Manning equation. The data trend should extrapolate back to the graph’s origin. If sediment were present in the pipe, the trend line would intersect the y-axis at a positive depth corresponding to the sediment depth.

The monitoring data were reviewed to identify potential issues as a part of the quality assurance and quality control (QA/QC) program. Frazier Engineering provided final data for the entire flow monitoring period.

The QA/QC was accomplished by analyzing the time series plots and the scatter plots for each meter. The scatter plots represent the relationship of the depth data to the velocity data at each meter. Gravity pipe flow scatter plots can indicate specific conditions within the pipe. Under normal operation, the points on the scatter plot form a "comma" shaped curve that can be defined using the Manning equation. Other patterns can suggest surcharge, backwater conditions, overflows, pump station operations, siphons, drifting sensors, sediment, or debris. Scatter plots of the final data for each meter are included in Appendix A. Examples of typical scatter plots are available from http://www.adsenv.com/scattergraphs. These scatter plots clearly indicate that the following flowmeters surcharged:

- FM1 - Pipe diameter is 18 inches, maximum recorded depths of approximately 122.4 inches (10.2 feet) on April 3, 2017. The manhole depth at this location is 11.95 feet. The scatter plot shows significant depth at low velocities with a "flat top" that could indicate a downstream SSO. Based on the scatter plot, there is likely a downstream manhole with a lower rim elevation (similar in elevation to the maximum depth of 10.2 feet) where an SSO occurred. A wet weather SSO was recorded downstream of this location at 99 Balfour Road on March 31, 2017, when the depth at FM1 reached 122.2 inches.
- FM2 - Pipe diameter is 42 inches, maximum recorded depths of approximately 139 inches (11.6 feet) on April 24, 2017. The manhole depth at this location is 12.6 feet. The scatter plot shows significant depth at low velocities with a "flat top" that could indicate a downstream SSO. Based on the scatter plot, there is likely a downstream manhole with a lower rim elevation (similar in elevation to the maximum depth of 11.6 feet) where an SSO occurred. A wet weather SSO was recorded downstream of this location at 99 Balfour Rd on March 31, 2017, when the FM2 depth ranged from 133 inches to 136 inches. The depth at this manhole exceeded 120 inches (10 feet) during 5 storm events: March 31st, April 3rd, April 6th, April 24th, and May 5th.
- FM3 - Pipe diameter is 24 inches, maximum recorded depths of approximately 79 inches (6.6 feet) on April 24, 2017. The manhole depth at this location is 11.3 feet. The high depths in the FM3 scatter plot indicate surcharge and backwater from a downstream restriction.
- FM5 - Pipe diameter is 24 inches, maximum recorded depths of approximately 101 inches (8.4 feet) on April 24, 2017. The manhole depth at this location is 12.2 feet. The high depths in the FM5 scatter plot indicate surcharge at this location.
- FM6 - Pipe diameter is 18 inches, maximum recorded depths of approximately 54 inches (4.5 feet) on April 3, 2017. The manhole depth at this location is 8.6 feet. The high depths in the FM6 scatter plot indicate surcharge at this location.
- FM7 - Pipe diameter is 24 inches, maximum recorded depths of approximately 98 inches (8.2 feet) on April 24, 2017. The manhole depth at this location is 12.3 feet. The high depths in the FM7 scatter plot indicate surcharge and backwater from a downstream restriction.

Figure 3-3 illustrates the data collected at the FM1 metering site. This shows a pattern consistent with backup and surcharge. The velocity in the pipe is reduced because of the backup and the depth rose, overtopping the 18-inch pipe. The depths leveled off just above 120 inches indicating a possible downstream SSO. In fact, one wet weather SSO was recorded downstream of this location at 99 Balfour Rd on March 31, 2017, when the depth at FM1 reached 122.2 inches.

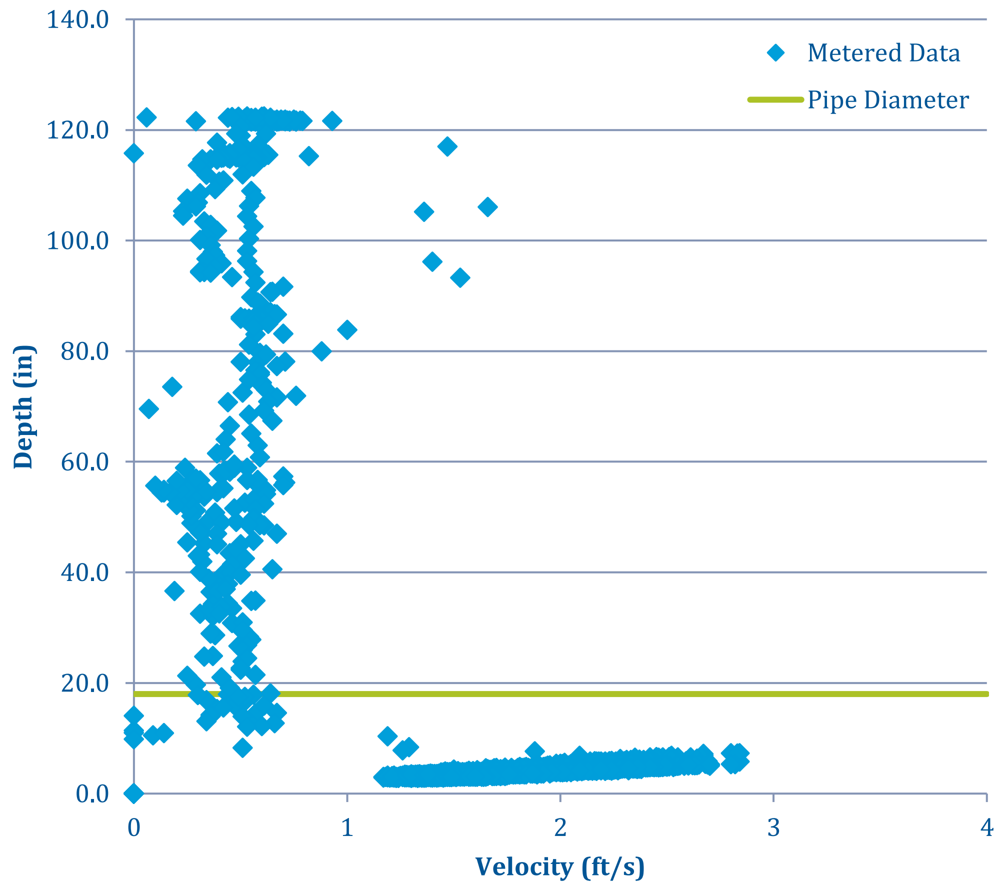


Figure 3-3 FM1 Flowmeter Scatter Plot

The data was reviewed to identify five types of issues common in flowmetering data:

- Data accuracy (reasonable depth – velocity relationships).
- Data drops.
- Proximity to pump stations.
- Flow balance (downstream meters recorded greater flow volumes than the upstream meters).
- Sediment deposition.

In general, the scatter plots showed good relationships between depth and velocity. No large data drops were observed in the data. Pump station operation was observed in the FM8 data, but the variability was not expected to impact the calibration.

3.2.1.1 Flow Balance

No flow imbalances were identified in the data during the dry weather period. Flow imbalances occur where downstream meters measured smaller flow volumes than upstream meters.

3.2.1.2 Sediment Deposition

The recorded depths and velocities are graphed against each other to develop the scatter plots in Appendix A. The resulting graph should yield an increasing relationship consistent with the Manning equation. The data trend should extrapolate back to the graph's origin. If sediment were present in the pipe, the trend line would intersect the y-axis at a positive depth corresponding to the sediment depth. The depth-velocity relationships for flowmeters FM5 and FM6 potentially demonstrate sediment deposition. These observations were compared with the site inspections performed by Frazier Engineering; at installation, the sediment was noted as negligible at every site. Sediment depth cannot be explicitly modeled in InfoSewer. Instead, depths were calibrated by increasing the roughness coefficient of the pipes.

3.2.2 Rain Gauge Sites

Three temporary rain gauges were installed in the sewershed. Rainfall data was being collected from the three sites during the 3 month monitoring period from February 20, 2016, to May 30, 2017. A cumulative rainfall plot is shown on Figure 3-4. The rain gauges show significant rainfall during the observation period. The model should be considered calibrated to saturated conditions. The three rain gauges recorded between 21 and 25 inches over the 3 month period.

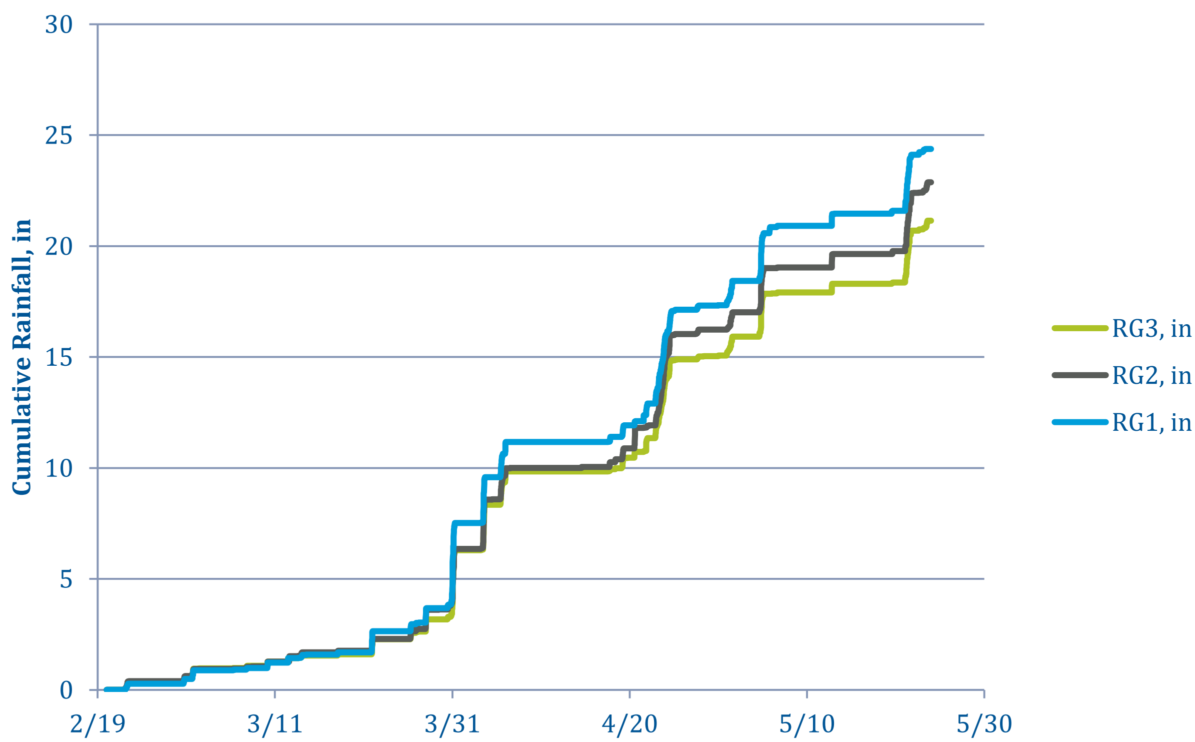


Figure 3-4 Cumulative Rainfall Plot for Rain Gauges in the COH Service Area

Four storm events were selected for the wet-weather calibration. Table 3-3 shows the date, rainfall depth, and duration of the selected storms.

Table 3-3 Selected Calibration Storms

STORM EVENT DATE	CUMULATIVE DEPTH (IN.)			PEAK INTENSITY (IN./HOUR)	DURATION (HR:MIN)
	min	mean	max		
March 31, 2017	2.55	3.07	3.68	1.92	11:15
April 3, 2017	2.03	2.09	2.2	1.84	07:30
May 4, 2017	1.93	2.02	2.16	1.88	11:30
May 21, 2017	1.86	1.90	1.95	0.80	10:15

The temporary gauges were positioned throughout the collection system to cover representative portions of the modeled network. The locations of the three rain gauges and the areas assigned to each gauge are shown on Figure 3-5.

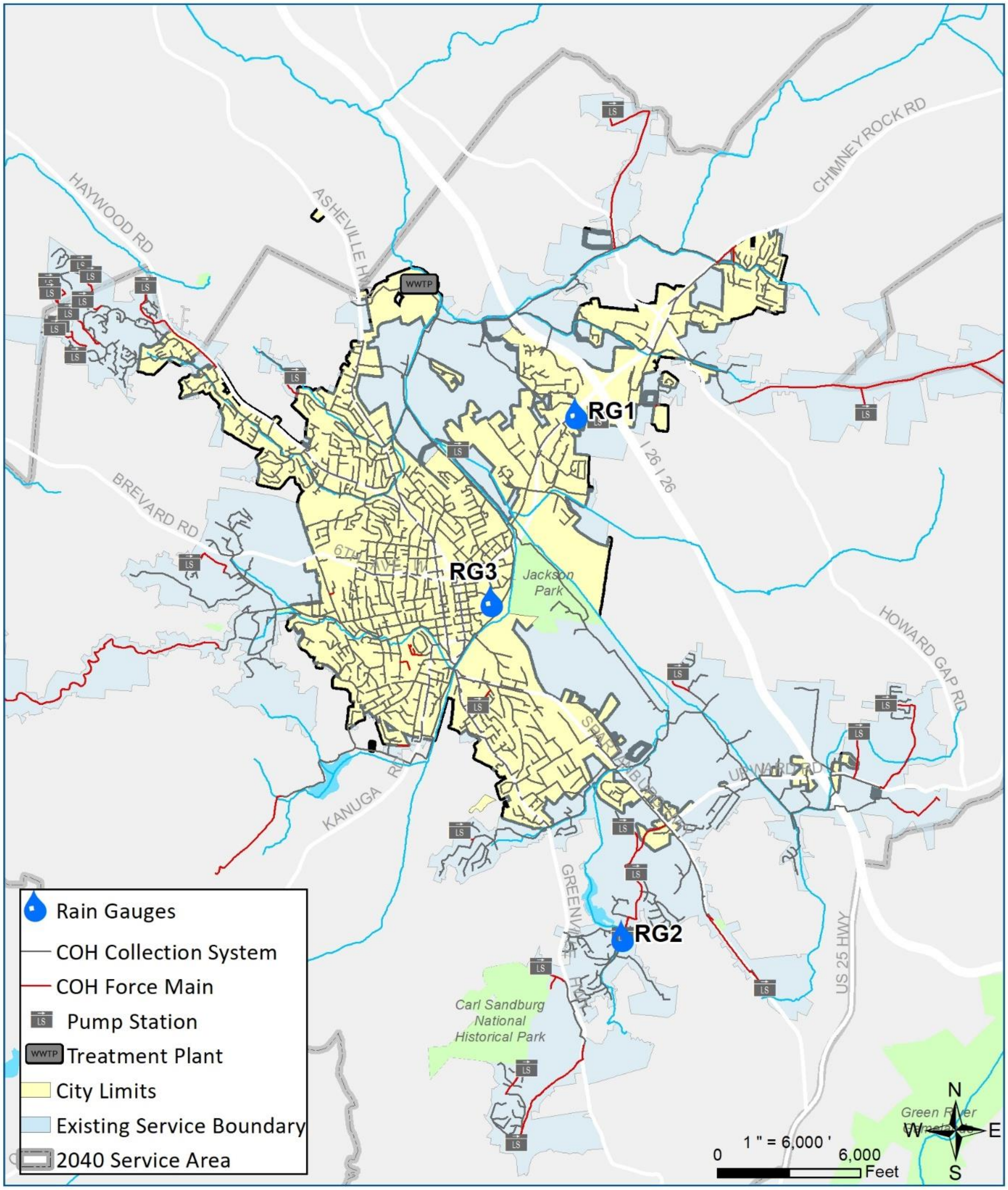


Figure 3-5 Temporary Rain Gauge Locations

Having several gauges across the basin provides accurate measurement of the rainfall for storms that have a wide spatial variation of rainfall depth. Also, multiple gauges allow for redundancy in case of a gauge malfunction. Missing data caused by malfunctioned rain gauges can be filled using data from other nearby functioning gauges if necessary. The name/location and recorded storm depth of each rain gauge are shown in Table 3-4.

Table 3-4 Hendersonville Rain Gauges and Cumulative Storm Depth

RAIN GAUGE	LOCATION	STORM DEPTH (IN.)			
		31 MAR	3 APR	4 MAY	21 MAY
RG1	Eastside Booster Station	3.68	2.05	2.16	1.95
RG2	Bonclarken Pump Station	3.00	2.03	1.93	1.86
RG3	Operations Building	2.55	2.20	1.98	1.9

3.3 DRY WEATHER CALIBRATION

3.3.1 Dry Weather Calibration Period

According to data from the temporary rain gauges, little to no rainfall occurred from March 2-March 9, 2017. Therefore, this time period was selected as the dry weather calibration time frame. This time period was also chosen since it did not follow any of the larger rain events that can cause elevated flows in the sewers for several days following the event. Figure 3-6 illustrates the rainfall for February 19-May 4, 2017. The figure shows the 15-minute interval rainfall depths.

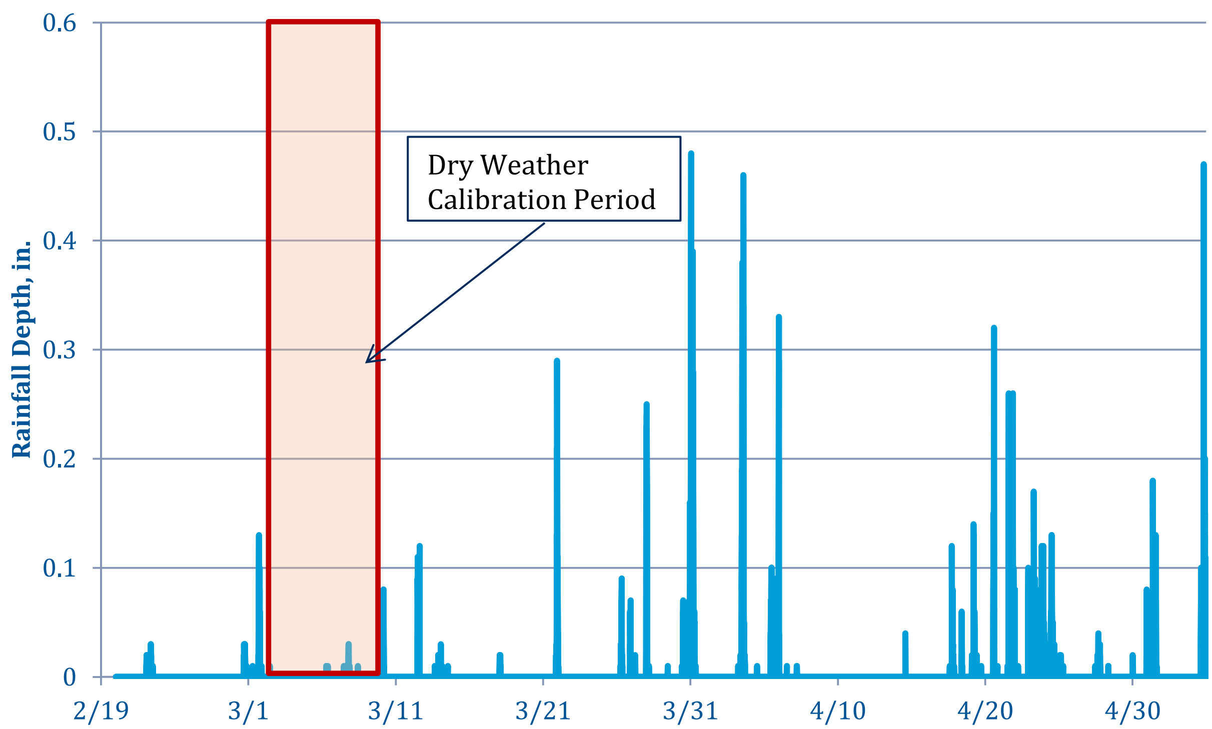


Figure 3-6 Rainfall Depth February–April 2017

3.3.2 Dry Weather Loadings

The flowmeter data was analyzed to determine the dry weather loadings for the sub-basins. The dry weather flow (DWF) includes contributions from all customers (base sanitary) in the collection system as well as GWI into the collection system. The DWF is separated into three components to describe magnitude and variation:

- Base sanitary flow (BSF).
- GWI.
- Diurnal patterns.

The rain gauge and flowmeter data were reviewed for weekday and weekend periods not influenced by rainfall events. The period between March 2 and March 9, 2017, was selected for the dry weather loads analysis. March 4 - March 5 were used to generate typical weekend diurnal flow patterns per meter, while the rest of the period was used for the weekday analysis. Figure 3-7 shows the average weekday flow at FM1 during the calibration period.

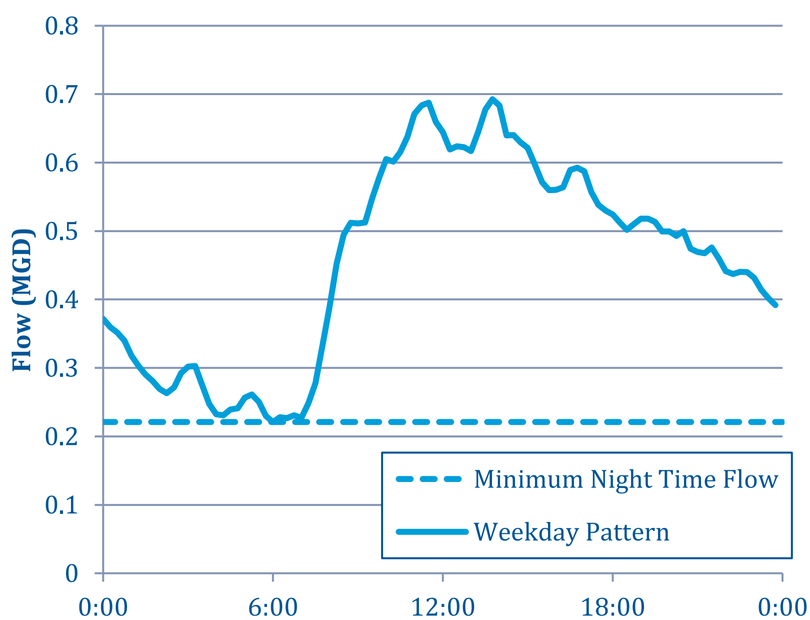


Figure 3-7 Average Weekday Flows through the FM1 Flowmeter for March 2-9, 2017

To determine the magnitude of the DWF, the average flows recorded at the monitoring locations during the dry weather period were separated into the BSF and GWI. The BSF is the loading directly contributed by the utility's customers. The GWI is assumed to be a constant, non-varying flow contribution entering the sewers from the groundwater table through defects in the sewer system during periods without rainfall influence.

The BSF is usually determined from geocoded water consumption data, starting with an assumption of an 80 percent return ratio to account for the portion of drinking water not returned to the wastewater collection system. It should be noted that this assumption was modified as a calibration parameter. For the COH system, the meters showed higher base sanitary flows than expected from the average winter consumption rates. The maximum GWI was assumed to be less than the minimum nighttime flow during the dry week. Then the BSF was estimated as the difference between the DWF and the GWI. The resulting BSF were larger than the average water consumption in the Winter of 2016-2017. For seven of the eight basins, the return ratio was greater than 100%, indicating that COH treated more wastewater than was indicating that COH treated more wastewater than was included in the water billing records. This issue can be caused by higher than normal seasonal fluctuations in usage, poor water metering data, illegal dumping, illicit connections or unmetered usage. The calibrated rates are shown in Table 3-5.

Table 3-5 summarizes the total dry weather flow, the incremental flow, the base sanitary loading, and the GWI for each meter basin. The total for flow for all basins represented the flow treated at the WWTP during the monitored DWF calibration week. Overall, COH received higher DWF on weekdays and about 20% of the dry weather flow during the calibration period was due to groundwater infiltration.

Table 3-5 GWI and BSF per Sub-Basin

FLOWMETER	WEEKDAY		WEEKEND		GWI (MGD)	GWI/ BSF	TWC (MGD)	WATER RETURN %
	DRY WEATHER FLOW (MGD)	BASE SANITARY FLOW (MGD)	DRY WEATHER FLOW (MGD)	BASE SANITARY FLOW (MGD)				
FM1	0.450	0.405	0.420	0.375	0.045	0.112	0.329	123%
FM2	1.008	0.893	0.929	0.813	0.115	0.129	0.319	280%
FM3	0.097	0.094	0.075	0.072	0.003	0.027	0.157	60%
FM4	0.398	0.215	0.414	0.230	0.184	0.855	0.095	225%
FM5	0.353	0.287	0.373	0.307	0.066	0.229	0.205	140%
FM6	0.513	0.344	0.484	0.315	0.169	0.492	0.196	175%
FM7	0.400	0.338	0.381	0.319	0.062	0.183	0.205	165%
FM8	0.240	0.210	0.215	0.184	0.031	0.148	0.145	145%
Total	3.459	2.786	3.291	2.615	0.675		1.651	

The BSF was applied to the model spatially throughout the collection system based on the geocoded water meters. GWI is associated with leaks along the gravity mains rather than customer location. Therefore, GWI was distributed throughout the collection system based on the diameter-length product (in-mile) of the gravity mains. The planar area of each pipe segment was calculated by multiplying the pipe diameter (inches) by the length of the pipe segment (miles). The GWI is divided by the sum of the pipe area for each sub-basin to calculate a normalized GWI loading which is then applied to the downstream node of each pipe segment and can be used to spatially distribute the GWI in each sub-basin.

*Inch Diameter Mile (in-dia*mile, idm) = Diameter (in-dia) x Length of pipe (mile)*

Normalized GWI Loading (gpd/idm) = Basin's GWI (gpd) / Basin's Sum of Inch Diameter Mile (idm)

GWI per Pipe (gpd) = Normalized GWI Loading (gpd/idm) x Inch Diameter Mile (idm)

3.3.3 Diurnal Patterns

The variation in the DWF to match the typical diurnal variation observed in municipal wastewater systems was accomplished by applying a dimensionless unit pattern, known as the diurnal pattern, to the BSF. Each flowmeter basin had its unique diurnal pattern. The diurnal patterns for the eight metered sub-basins were determined for the BSF (GWI was subtracted) using the 15-minute flow data provided for March 2–9, 2017. The average flow for each time step was used to determine the peaking factor for the corresponding time step by normalizing to the average weekday BSF for the week-long calibration period.

Figure 3-8 illustrates the weekday and weekend diurnal patterns for sub-basin FM1. The remaining diurnal patterns are included in Appendix B. The average of the weekend pattern was 0.93. This sub-basin has higher weekday flows than weekend flows since the weekend average is less than 1.

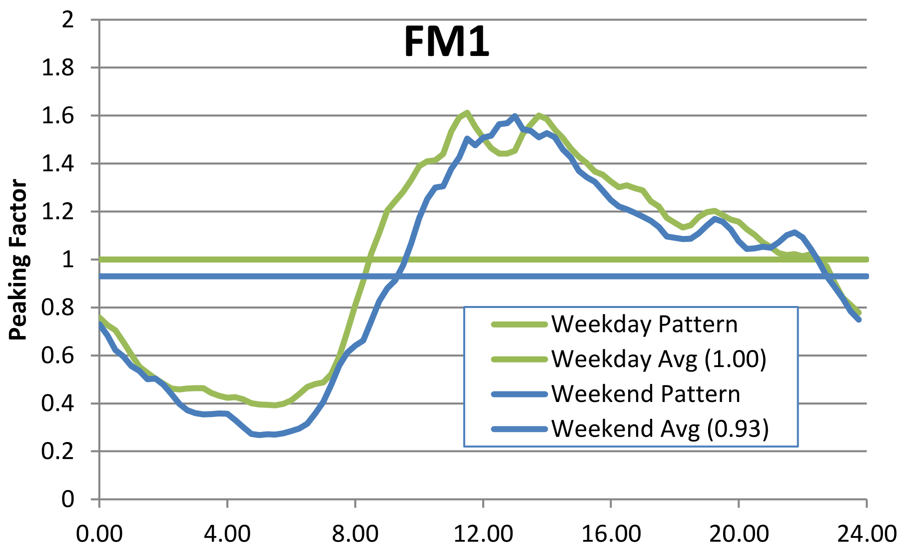


Figure 3-8 Diurnal Patterns for FM1

3.3.4 Dry Weather Calibration

The model was calibrated under DWF conditions to a dry weather period starting on March 2, 2017, and ending on March 9, 2017. Qualitative and quantitative comparisons were used as metrics for assessing dry weather calibration.

The dry weather goals were developed based on guidelines from the UK's Wastewater Planning Users Group (WaPUG), now organized as the Urban Drainage group under the Chartered Institution of Water and Environmental Management (CIWEM) and are shown in Table 3-6. When specific goals cannot be met in all cases, it can be due to a variety of reasons such as metering equipment failures, unsatisfactory meter location, and accuracy, system repairs, system blockages, rainfall variability, short-term system anomalies, etc. The qualitative comparisons (shape and timing) are the primary goals for assessing the match between the model and metered data. Only after the qualitative goals are met, are the quantitative comparisons determined to verify calibration.

Table 3-6 Dry Weather Model Calibration Goals

METRIC	DRY WEATHER CALIBRATION GOALS (WAPUG)
Shape	The shape of the modeled and metered curves should be similar for depth and flow
Timing	The timing of the peaks, troughs, and recessions of the modeled and metered curves should be similar for depth and flow
Peak Flow	±10% of measured values, or ±0.1 MGD for low flows
Volume	±10% of measured values, or ±0.1 MGD for low flows
Peak Depth	± 0.3 foot at non-surcharged locations or –0.3 to +1.5 feet at surcharged locations

The diurnal patterns and wastewater loadings produced in the DWF development analyses were input into the hydraulic model to generate DWF. The model results were compared to the observed flowmeter data, and the hydrographs were iteratively adjusted in order to reasonably match the typical dry weather flow pattern of each sub-basin. The dry weather calibration adjusted the water-to-sewer return rates and diurnal curves to match observed meter flow records. The dry weather calibration verified that the dry weather loadings are distributed appropriately throughout the model and confirmed the model routes the flow through the system appropriately.

Once the model was deemed calibrated to the flow conditions, the model depth results were compared to the depth data recorded by the flowmeters. The observed records were plotted over time for comparison to the model results. For meters with non-conforming depth and velocity results, scatter graphs were evaluated for better system understanding, such as potential backwater events or low flows that could cause such discrepancies.

Figure 3-9 and Figure 3-10 are sample calibration plots showing the match between the model results and the metered data for the calibration period for depth and flow. Specifically, the calibration plot is for the FM1 metering site. Each graph has the model results (red) and the meter data (black) to illustrate the agreement of the model results to the observed data. Appendix C contains all of the dry weather calibration plots for each of the metering sub-basins as well as detailed calibration statistics for DWF.

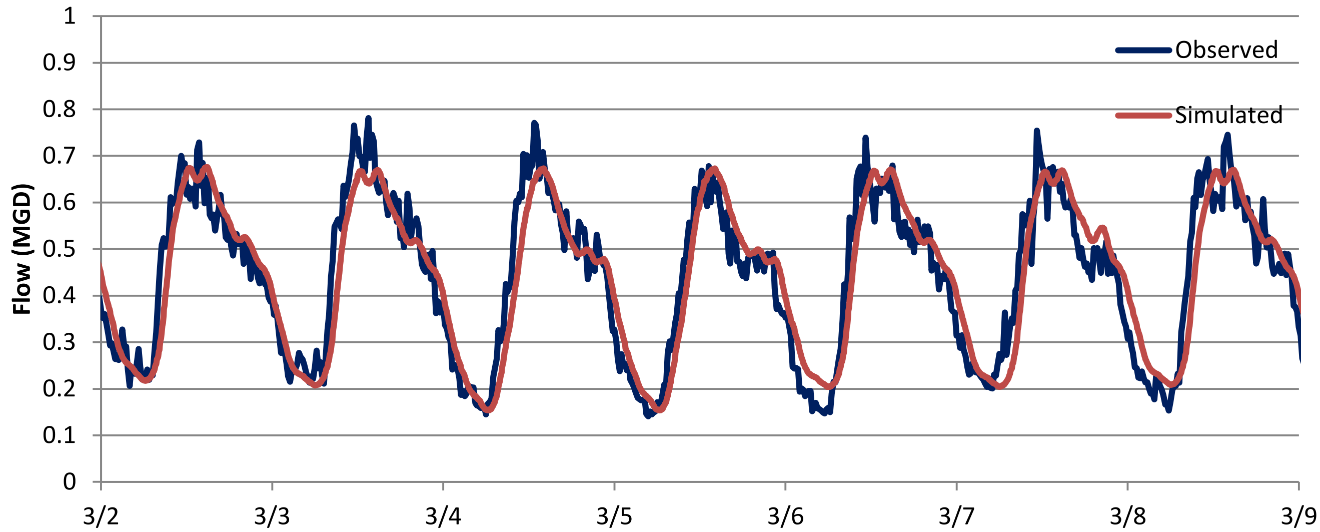


Figure 3-9 Sample Dry Weather Flow Calibration Plot (FM1)

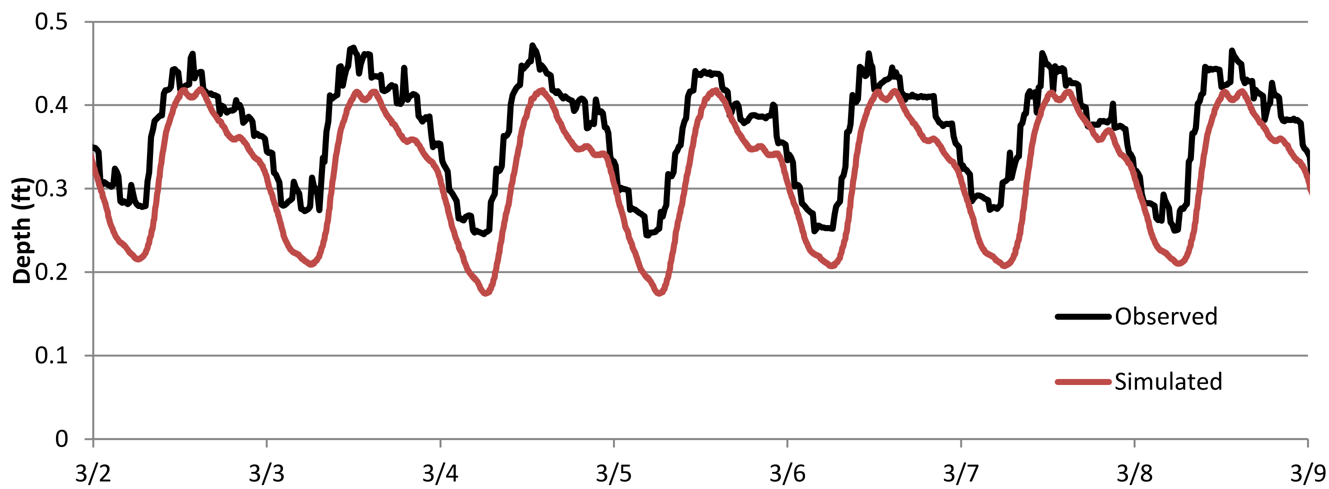


Figure 3-10 Sample Dry Weather Depth Calibration Plot (FM1)

Table 3-7 summarizes the overall dry weather calibration results for volume, peak flow, and peak depth at all metering locations. As shown, the calibration results meet the WaPUG guidelines for peak flow, volume, and peak depth with only a couple of exceptions. Values colored red are results that are outside of the calibration goals.

Table 3-7 Dry Weather Calibration Results

SUB-BASIN		PEAK DEPTH (FT) * GOAL: ± 0.3FT AT NON-SURCHARGED LOCATIONS OR –0.3FT TO +1.5FT AT SURCHARGED LOCATIONS			VOLUME (MG) * GOAL: ±10% OF MEASURED VALUES, OR ±0.1 MGD FOR LOW FLOWS			PEAK FLOW (MGD) * GOAL: ±10% OF MEASURED VALUES, OR ±0.1 MGD FOR LOW FLOWS		
		OBS	SIM	DEVIATION	OBS	SIM	DEVIATION	OBS	SIM	DEVIATION
Weekday	1	0.47	0.42	-0.05	2.25	2.24	-0.3%	0.74	0.67	-9.3%
	2	0.79	0.58	-0.22	15.04	15.12	0.6%	3.91	4.16	6.5%
	3	0.43	0.33	-0.10	2.47	2.53	2.5%	0.71	0.77	8.9%
	4	0.35	0.27	-0.08	1.99	2.00	0.4%	0.65	0.59	-9.2%
	5	1.32	1.49	0.17	4.33	4.44	2.5%	1.34	1.34	0.0%
	6	0.50	0.62	0.12	2.56	2.65	3.4%	0.77	0.86	12.2%**
	7	0.54	0.35	-0.20	3.20	3.24	1.2%	1.08	0.99	-8.2%
	8	0.49	0.30	-0.19	1.20	1.21	0.7%	0.63	0.59	-6.1%
Weekend	1	0.47	0.42	-0.05	0.84	0.83	-0.6%	0.74	0.67	-8.6%
	2	0.79	0.56	-0.23	5.74	5.71	-0.5%	4.00	3.98	-0.6%
	3	0.44	0.31	-0.13	0.98	0.97	-0.4%	0.74	0.71	-4.7%
	4	0.35	0.28	-0.07	0.83	0.82	-0.4%	0.65	0.63	-2.7%
	5	1.32	1.48	0.17	1.71	1.71	-0.5%	1.32	1.30	-1.4%
	6	0.50	0.58	0.07	0.97	0.97	0.3%	0.73	0.73	-0.7%
	7	0.51	0.32	-0.19	1.19	1.19	-0.1%	0.85	0.85	0.1%
	8	0.48	0.28	-0.20	0.43	0.43	0.8%	0.60	0.53	-11.3%**

* Negative values mean that the simulated values were less than the metered values.
** Complied with the low flow calibration goal: ±0.1 MGD.

All of the calibration goals were met at each meter for peak depth and volume. For peak flow, two meters were just outside the 10 percent goal; however, the parameters met the low flow goal of ±0.1 MGD. The dry weather flow plots show a good visual match between the observed and simulated results for all the meters.

Figure 3-11 through Figure 3-13 summarize the overall agreement between the metered and the modeled results for peak depth, peak flow, and volume for the meter locations. The data are presented in a 1:1 scatter plot comparison of the model results data (y-axis) with the observed data (x-axis), where the 1:1 line (solid blue line) represents an exact match between model and monitored data. The figure also shows dashed lines to represent the percent difference or absolute ranges defining the dry weather calibration goals. As shown on these figures, the model matches the observed data within the acceptable range of calibration.

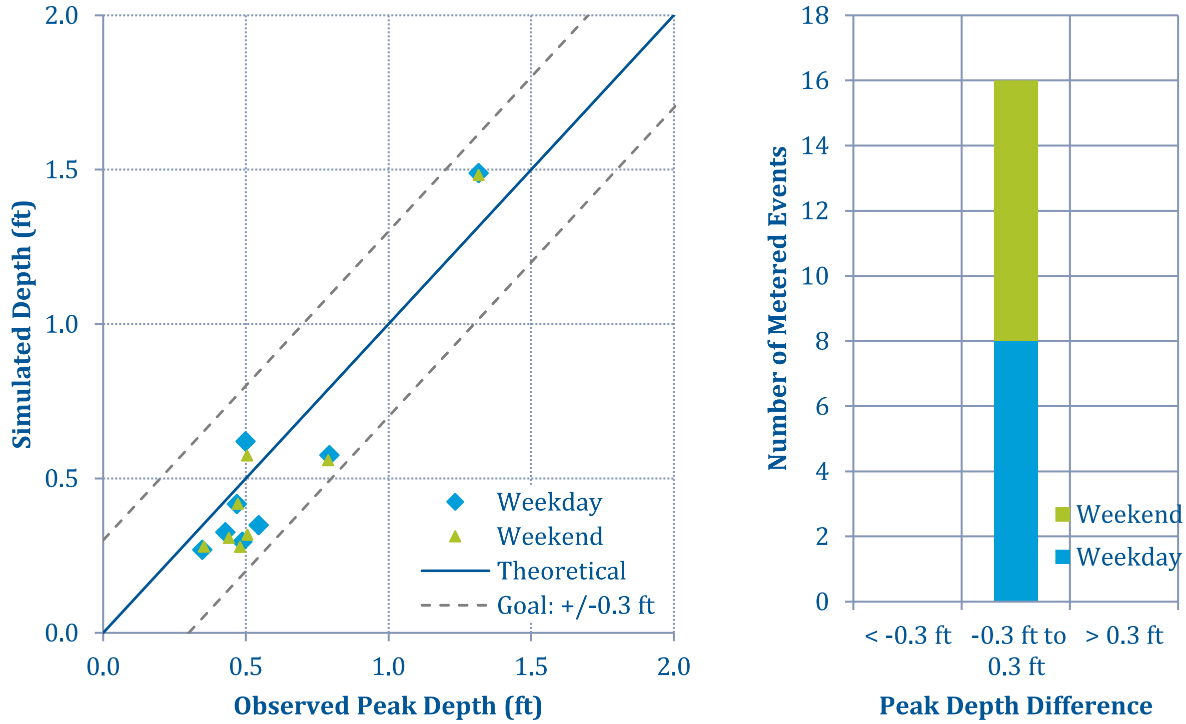


Figure 3-11 Peak Depth Scatter Plot – Dry Weather

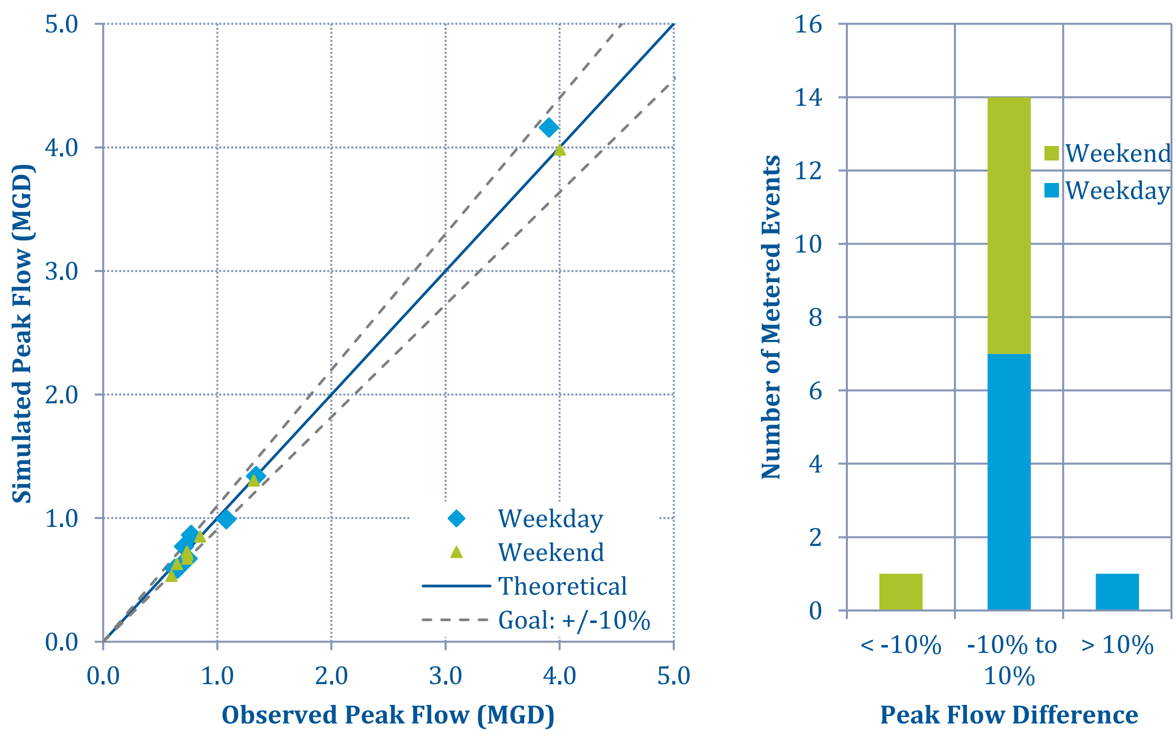


Figure 3-12 Peak Flow Scatter Plot – Dry Weather

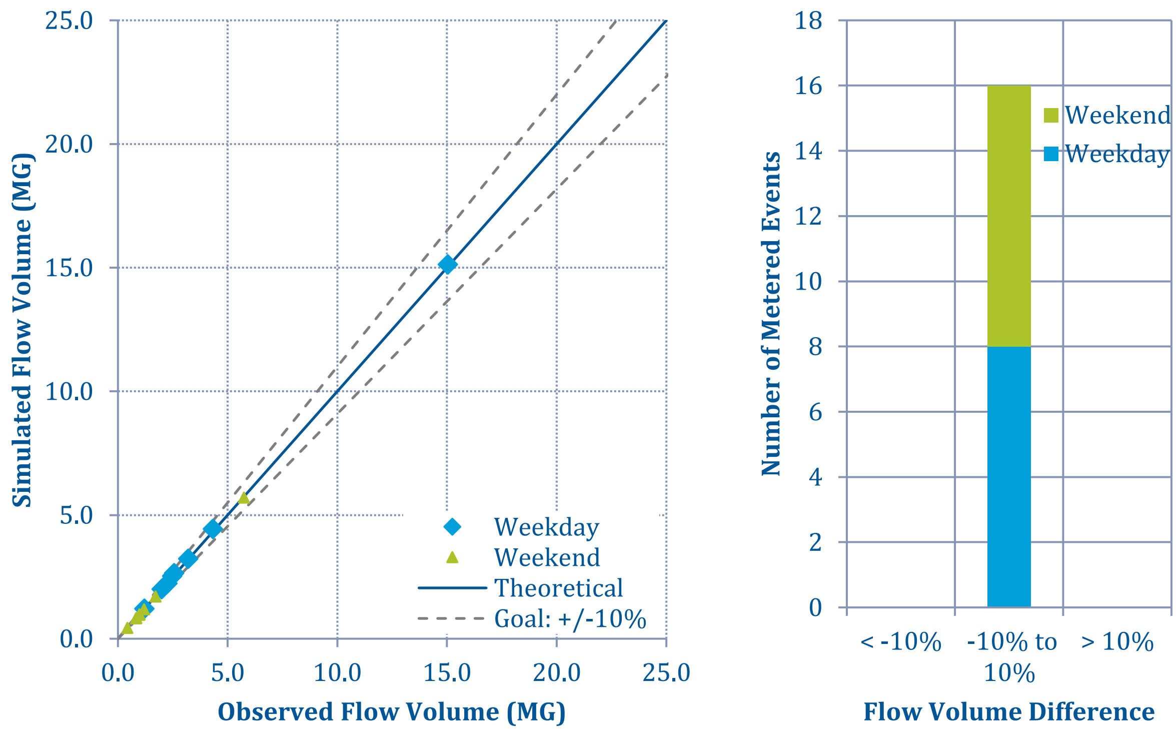


Figure 3-13 Volume Scatter Plot – Dry Weather

It should be noted that the meter locations where the low flow calibration goal was used are shown outside of the calibration goals for the flow scatter plot (Figure 3-12). These events should be considered within the calibration goals, but the scatter plot compares the relative percent difference, which is greater than 10 percent for these sites.

3.4 WET WEATHER CALIBRATION

The COH model was also calibrated to wet weather conditions. To perform a wet weather calibration, significant storm events need to be identified for use in the wet weather calibration of the sewer model.

3.4.1 Calibration Events

Significant storm events were identified by collecting rainfall data from the three temporary rain gauges and comparing that data to the flow data recorded at each flow meter location. Figure 3-5 illustrates the location of each of the rainfall gauges in relation to the monitoring basins discussed in previous sections. The storm events that had a greater response at the flow meter location were chosen as significant storm events for the wet weather calibration. The process of choosing a significant storm event required the following:

- Rainfall gauge collected data with a significant volume.
- Sewer flow data recorded an observable wet weather response at specific locations within the system.

The following rainfall events were selected for the wet weather calibration events:

- March 30, 2017 (3.68 inches).
- April 3, 2017(2.20 inches).
- May 4, 2017 (2.16 inches).
- May 21, 2017 (1.95 inches).

3.4.2 Wet Weather Flows

Increased flows observed in the sewer system during periods of rainfall are caused by RDII, which is extraneous groundwater or stormwater entering the collection system. Inflow is the direct connection of stormwater to the sewer collection system through sources such as manholes, cleanout lids, roof downspouts, and catch basins; whereas infiltration is characterized by leaky pipes and manholes allowing groundwater to infiltrate the collection system. In order to analyze the collection system's response to rainfall and develop initial model inputs, the RTK Unit Hydrograph Method was used. In a sanitary system, the RDII is driven by a myriad of factors including the following:

- Age and condition of the system.
- Construction practices at the time of installation.
- Prevalence of direct (illicit) stormwater connections to the sanitary system.
- Maintenance of the system.
- Antecedent moisture conditions (the saturation of the ground around the sewers).
- Groundwater elevation.

The RTK Unit Hydrograph Method uses three unit hydrographs to account for fast, medium, and slow RDII responses. R is the fraction of rainfall volume entering the sewer system, T is the time to peak flow, and K is the ratio of the time of recession to T. Figure 3-14 illustrates the calculation of each of the three unit hydrographs as well as the total runoff. The three unit hydrographs can be used to differentiate between direct runoff, rapid infiltration, and slower infiltration. By comparing R, T, and K factors, it is possible to rank relative sewershed RDII responses and prioritize sewer system rehabilitation efforts. In addition, the RDII reduction from selected rehabilitation methods can be estimated by applying reduction factors to the R, T, and/or K factors.

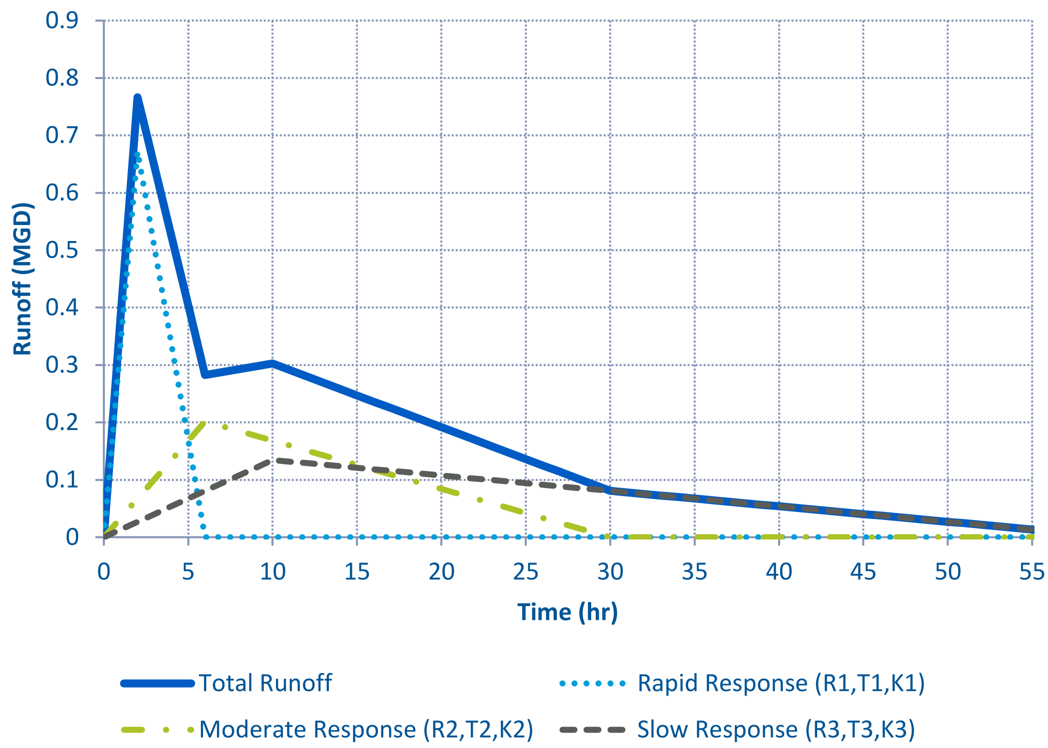


Figure 3-14 RTK Runoff Calculation Process

A detailed flow data analysis was conducted for the three identified storm events at each flowmeter. In this analysis, initial R, T, and K factors were input to the model for an initial run, and adjustments were made after comparing model results with the actual monitored flow data for each metering location. Observed and predicted peak flows and flow volumes are displayed graphically in the model during the calibration process. The appropriate combination of R, T, and K values is determined iteratively by adjusting the various coefficients to find the best match between the simulated and the observed RDII hydrographs. Table 3-8 summarizes the range of typical timing parameters (T and K) used per RTK factor to represent each of the three RTK unit hydrographs. As mentioned earlier, the R factor represents the fraction of rainfall entering the sewer as RDII, which can vary depending on the metering basin while the T and K are the triangular hydrograph shape and timing factors.

Table 3-8 RTK Timing Ranges

RDII RESPONSE	T (HOURS)	K
Fast	0.1-3	0.1-2
Medium	3-6	2-4
Slow	6+	4+

The model also contains a hydraulic engine to route the flow through the collection system, determine the depth of flow, and account for flooding (and as a result, water lost to the system). The model used the “dynamic wave” for its flow attenuation routing method with a 60 second time step that varies to maintain stability.

3.4.3 Wet Weather Calibration Results

A sample calibration plot is illustrated below on Figure 3-15 and Figure 3-16. The figures below are for the FM1 meter site during the March 31 and April 3 storm events. Appendix D contains the calibration plots for each of the calibration storm events. The R, T, and K values for the simulated model results (red) are adjusted until they closely match the metered data (black).

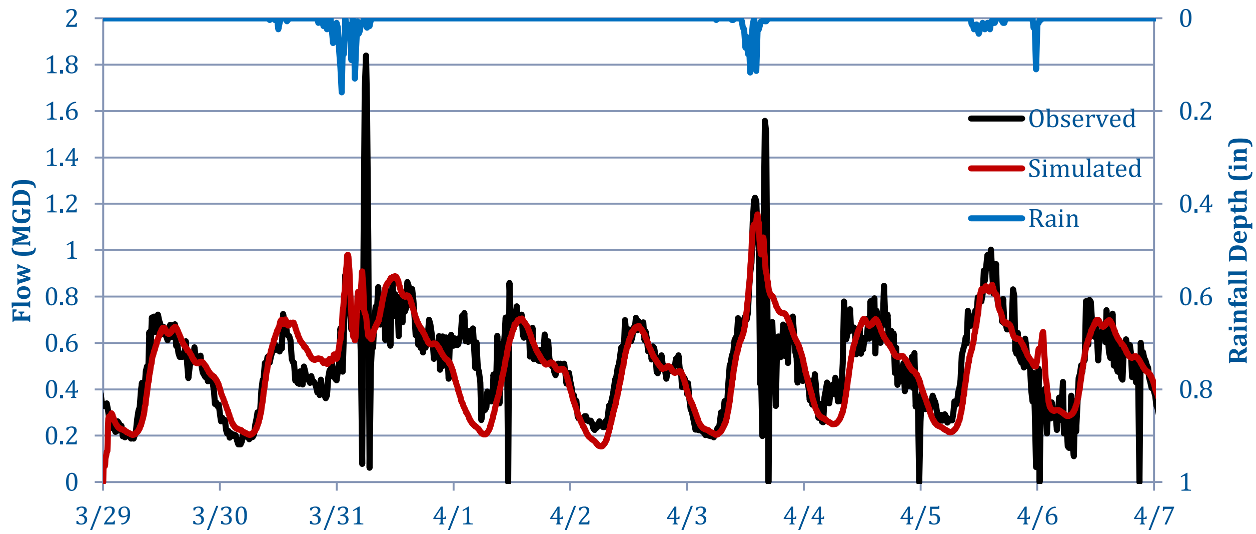


Figure 3-15 Sample Wet Weather Flow Calibration Plot (FM1)

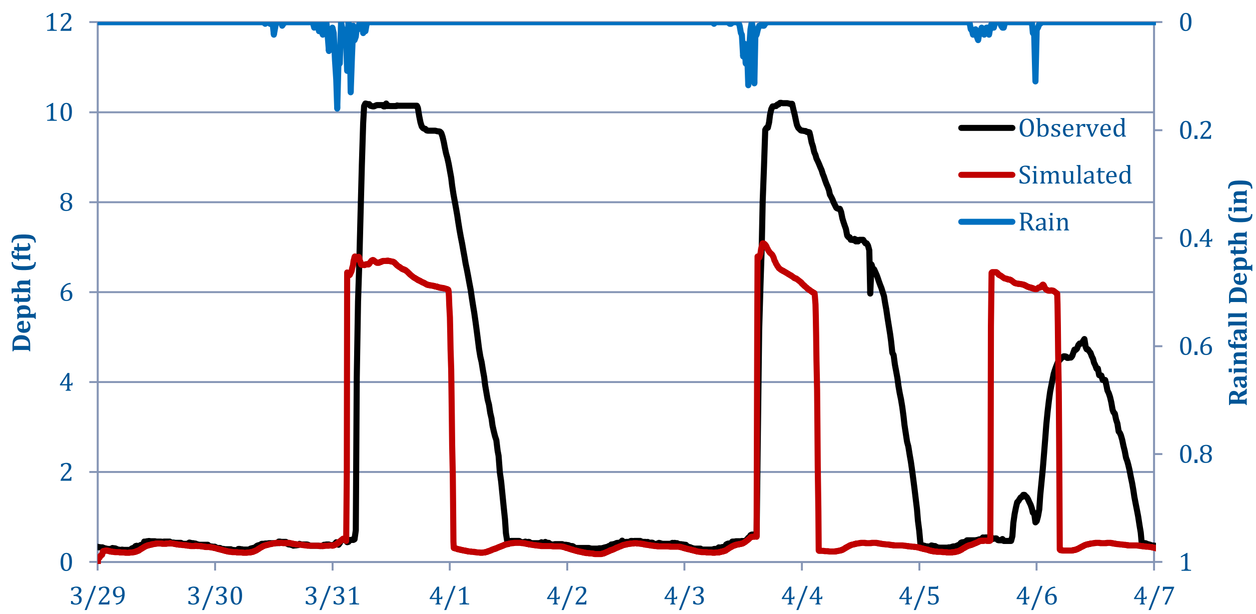


Figure 3-16 Sample Wet Weather Depth Calibration Plot (FM1)

The goal of the calibration was to reasonably represent the flows and depths of each storm for each meter during its monitoring period and develop a tool that can be utilized for predicting the collection system performance under a variety of conditions, including more intense design storm events used for capital improvement planning. This was accomplished by adjusting various modeling input parameters. The RTK parameters were adjusted to best match the peak flows and volumes of the observed data, while the pipe roughness coefficients were adjusted in order to meet the depths of the observed data.

A single set of modeling parameters was developed that adequately predict sewer system response caused by each of the rainfall events while maintaining realistic modeling parameter values. Similar to the dry weather calibration, the quantitative wet weather goals were developed using the WaPUG guidelines shown in Table 3-9. Typically, when specific goals cannot be met, it can be due to a variety of reasons, such as metering equipment failures, unsatisfactory meter location and accuracy, system repairs, system blockages, rainfall variability, and short-term system anomalies.

The qualitative goals (shape and timing) are the primary calibration goals for model calibration. These goals are assessed visually by comparing the depth and flow time series for both the model and meter data against each other. After the visual comparison of these results demonstrates agreement, quantitative comparisons are performed to determine the calibration accuracy. During the visual comparison, the relative model response can be compared to the meter data to confirm that peak flow, volume, and peak depth generally agree with the monitoring data. It is also crucial to make sure that any differences between the model and meter data are balanced between the storms meaning that there are a relatively equal number of meter events that over and underpredict.

Table 3-9 Wet Weather Model Calibration Goals

METRIC	WET WEATHER CALIBRATION GOALS (WAPUG)
Shape	The shape of the modeled and metered curves should be similar for depth and flow.
Timing	The timing of the peak, troughs, and recessions of the modeled and metered curves should be similar for flow and depth.
Peak Flow	-15% to +25% of measured values, or ±0.1 MGD for low flows.
Volume	-10% to +20% of measured values, or ±0.1 MG per day for low flows.
Peak Depth	-0.3 feet to +1.5 feet at surcharged locations. ±0.3 feet at non-surcharged locations.

The most crucial parameter in model calibration is the percentage of the runoff area relative to the contributing area. This value provides a measure of the amount of rainfall that is converted into sewer system flow. The final calibrated runoff percentages were summed up for each of the responses (fast, medium, slow, etc.) and summarized in Table 3-10. For most of the basins, the total percentages are lower than what is typically seen in municipal collection systems indicating that RDII is not entering the system at excessive rates. A total runoff area greater than 5 percent is considered excessive in a separate sanitary sewer system. The highest R-value from the model calibration was 5.70 percent in Flowmeter Basin 5, in the older section of the city. Figure 3-17 shows the runoff percentages for each FM basin. FM5, FM3, and FM6 had the highest I/I rates.

Table 3-10 Calibrated RTK Parameters

	1	2	3	4	5	6	7	8	AVERAGE
Total R	0.41%	0.00%	3.74%	1.19%	5.70%	3.69%	1.47%	0.22%	1.71%
R1	0.14%	0.00%	0.77%	0.34%	0.93%	0.74%	0.25%	0.09%	0.36%
R2	0.14%	0.00%	1.03%	0.34%	1.45%	1.48%	0.31%	0.08%	0.51%
R3	0.14%	0.00%	1.94%	0.51%	3.32%	1.48%	0.91%	0.05%	0.84%
T1	0.5 hr	1 hr	1 hr	1 hr	1.5 hr	1 hr	0.5 hr	1 hr	1 hr
T2	4 hr	4 hr	3 hr	4 hr	4 hr	4 hr	3 hr	4 hr	4 hr
T3	8 hr	8 hr	8 hr	6 hr	8 hr	8 hr	8 hr	6 hr	8 hr
K1	1	1	1.5	1	1.5	1	1	1.5	1
K2	2	2	2.5	2	3	4	2	2	2
K3	5	5	8	5	8	8	8	4	5

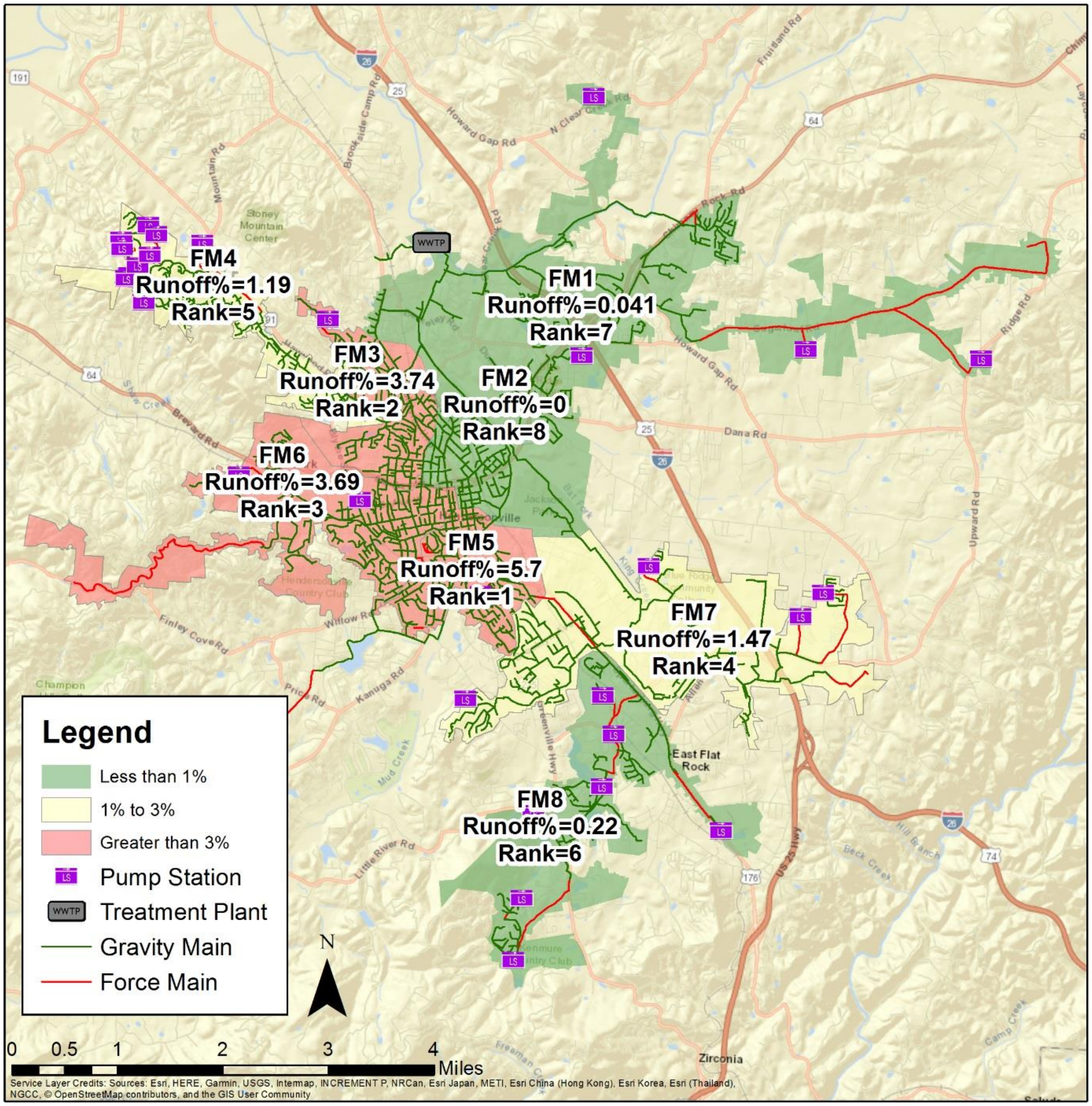


Figure 3-17 Calibrated Total Runoff Percentage

Table 3-11 provides the quantitative measurements for the calibration goals for each storm event for each meter; values outside calibration criteria are highlighted in red. Some of the causes for variation from calibration goals were variable rainfall data caused by scattered storms, periods of low dry weather loadings, and erroneous meter spikes.

Table 3-11 Wet Weather Calibration Results

METER	EVENT	PEAK DEPTH (FT) GOAL: ± 0.3 FT OR –0.3 FT TO +1.5 FT AT SURCHARGED LOCATIONS			VOLUME (MG) GOAL: -10% TO 20% OF MEASURED VALUES			PEAK FLOW RATE (MGD) GOAL: -15% TO 25% OF MEASURED VALUES		
		obs	sim	s-o	obs	Sim	% diff	obs	Sim	% diff
1	3/31	10.192	6.793	**-3.399***	1.944	1.877	-3.5%	0.851	0.980	15.1%
1	4/3	10.206	7.085	**-3.121***	2.056	2.082	1.3%	1.194	1.154	-3.3%
1	5/4	5.926	6.664	0.738*	1.534	1.513	-1.4%	1.002	1.014	1.2%
1	5/21	3.815	6.708	**2.894***	1.030	1.076	4.4%	1.072	0.952	-11.2%
2	3/31	11.515	11.922	0.407*	15.790	18.744	18.7%	12.205	12.534	2.7%
2	4/3	11.547	12.102	0.555*	18.857	20.914	10.9%	9.147	13.668	**49.4%**
2	5/4	10.539	11.838	1.299*	12.485	13.924	11.5%	10.601	11.812	11.4%
2	5/21	9.702	11.649	**1.947***	8.126	11.623	**43.0%**	7.089	9.933	**40.1%**
3	3/31	5.607	1.600	**-4.007***	3.466	3.853	11.2%	3.796	4.170	9.9%
3	4/3	4.325	2.183	**-2.142***	4.349	4.223	-2.9%	4.005	3.497	-12.7%
3	5/4	0.809	1.125	**0.316**	2.963	2.759	-6.9%	2.734	3.081	12.7%
3	5/21	0.728	0.672	-0.056	2.079	2.435	17.1%	2.225	2.444	9.8%
4	3/31	0.479	0.462	-0.017	1.936	1.929	-0.3%	1.407	1.458	3.6%
4	4/3	0.493	0.425	-0.068	2.369	2.134	-9.9%	1.335	1.275	-4.5%
4	5/4	0.373	0.415	0.042	1.439	1.514	5.2%	0.879	1.218	**38.6%**
4	5/21	0.406	0.383	-0.023	1.066	1.179	10.6%	1.031	1.072	4.0%
5	3/31	7.654	6.373	**-1.281***	6.826	7.357	7.8%	5.580	6.646	19.1%
5	4/3	6.910	5.843	**-1.067***	8.844	7.970	-9.9%	5.680	5.719	0.7%
5	5/4	3.141	2.785	**-0.356***	5.247	4.793	-8.6%	5.539	4.954	-10.6%
5	5/21	2.136	2.506	0.370*	3.743	4.601	**22.9%**	3.960	4.620	16.7%
6	3/31	4.132	2.141	**-1.990***	3.717	4.338	16.7%	3.997	4.569	14.3%
6	4/3	4.474	1.739	**-2.735***	4.951	4.782	-3.4%	4.112	3.577	-13.0%
6	5/4	1.384	1.664	0.280	3.342	3.065	-8.3%	3.513	3.361	-4.3%
6	5/21	1.187	1.551	**0.364**	2.337	2.755	17.9%	2.548	3.005	17.9%
7	3/31	7.014	5.282	**-1.732***	4.188	4.050	-3.3%	3.511	3.406	-3.0%
7	4/3	5.608	6.766	1.158*	5.241	4.787	-8.7%	4.073	3.917	-3.8%
7	5/4	0.819	3.289	**2.470**	3.242	3.134	-3.3%	2.979	3.240	8.8%
7	5/21	0.702	1.893	**1.191**	2.173	2.744	**26.3%**	2.274	2.444	7.5%
8	3/31	0.749	0.449	-0.300	1.171	1.125	-3.9%	1.263	1.069	**-15.3%**
8	4/3	0.768	0.518	-0.250	1.330	1.358	2.1%	1.369	1.367	-0.1%
8	5/4	0.625	0.489	-0.135	1.005	0.922	-8.2%	1.016	1.244	22.4%
8	5/21	0.595	0.364	-0.231	0.743	0.744	0.1%	0.726	0.813	12.0%

*Surcharged

During large storm events, the WWTP only operates one 4,500 gpm pump at the influent pump station. The flow into the plant is limited in order to maintain the solids concentration in the clarifiers. The 42 inch interceptor backs up from the WWTP into the sewer system. Many of the manholes along this stretch were marked as sealed in the COH's GIS. The high levels and sealed manholes cause the system to operate under pressurized conditions. No RDII flow was estimated in the FM2 basin because the majority of the pipes were full flowing during a storm event, which prevented any additional infiltration.

The volume calibration goal was met for the majority of the meter events. The results were balanced between the different storms where the models slightly under-predicted for one storm and slightly over-predicted for another. As a result, the model response is considered to be balanced; meaning, on average, the model will accurately represent the wet weather volumes entering the collection system for a variety of storm events. In three instances, the calibration was outside of model parameters. FM2 generally measured lower flows than the sum of the upstream meters, which could indicate overflows in the basin. FM5 experienced a meter drop during the event that caused a problem with the total volume recorded. The May 21 event also recorded lower flows at FM7, but that lower flow was balanced by higher flows during the other three storm events.

Errors in the peak depth calibration were mostly traced to the WWTP operation. Additionally, discrepancies in which manholes were sealed could impact the location and severity of predicted overflows. The unsealed manholes in the FM2 basin were likely to overflow during storm events, which affected the simulated depth at upstream locations.

The peak flow calibration goals were met for the majority of the storm events. The meter response is considered to be balanced because there were results that tended to slightly over- and under-predict the wet weather response, giving the city confidence that the model can predict a wide variety of storm events accurately. Again, the peak flows at FM2 tended to be over-predicted with just the sum of the upstream meters. The calibration could be improved in the future if the backwater from the WWTP could be mitigated. Two other events were outside of calibration guidelines: May 4 at FM4 and March 31 at FM8. These two events saw much different I/I responses than the other three recorded events. The difference could be caused by variability in the rainfall in the area or problems at upstream pump stations.

Figures 3-18, 3-19, 3-20, and 3-21 present the 1:1 scatter plots for the wet weather calibration for peak depth (surcharged and un-surcharged), peak flow, and volume during each of the storm events. These figures compare the predicted or modeled results (y-axis) to the monitored data (x-axis), where the 1:1 line represents an exact match between the model and monitored data. The figures also include dashed lines to represent the percent difference or absolute ranges defining the wet weather calibration goals. These calibration comparisons were calculated using the monitoring data and model results of each of the calibration storms. Each point represents the modeled and observed data for each meter.

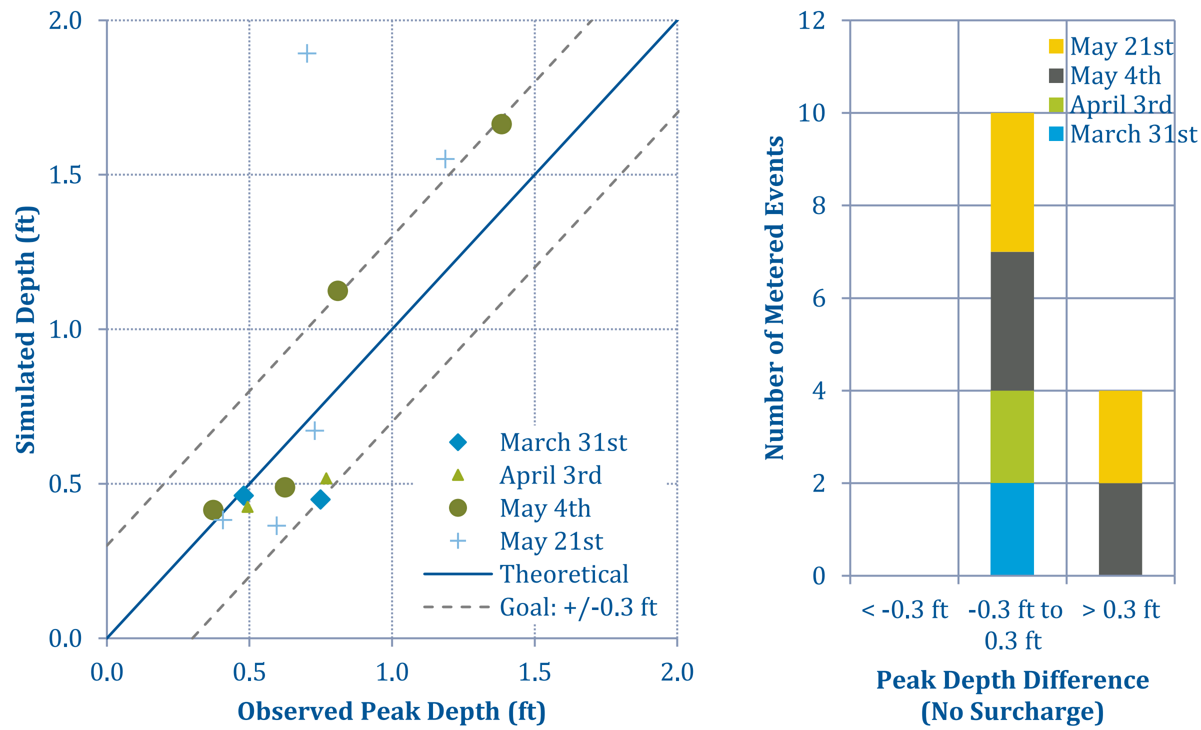


Figure 3-18 Peak Depth Scatter Plot – Wet Weather, No Surcharge

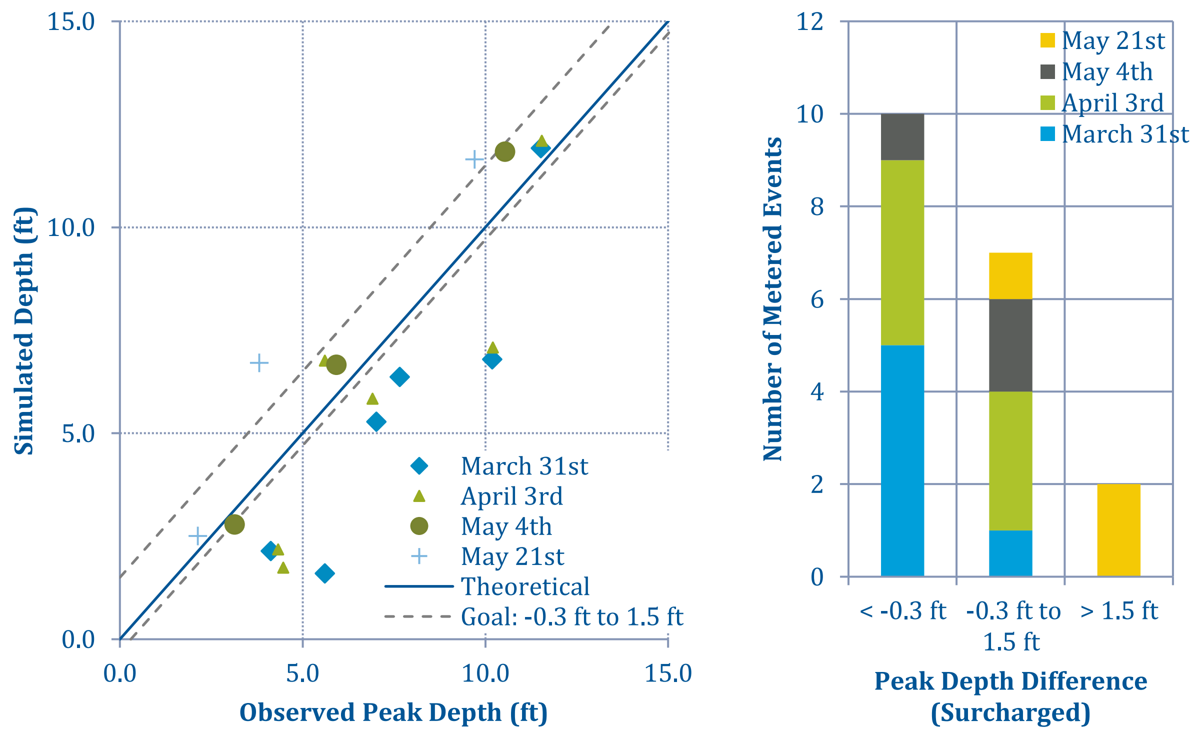


Figure 3-19 Peak Depth Scatter Plot – Wet Weather, Surcharge

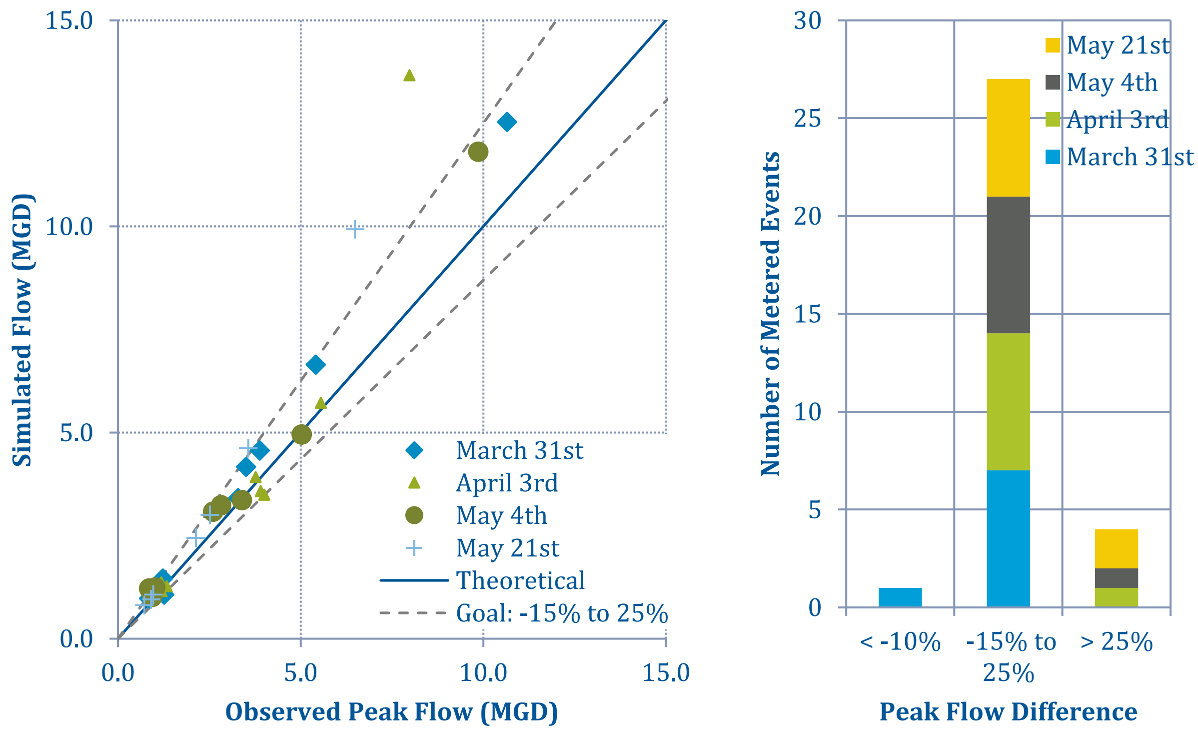


Figure 3-20 Peak Flow Scatter Plot – Wet Weather

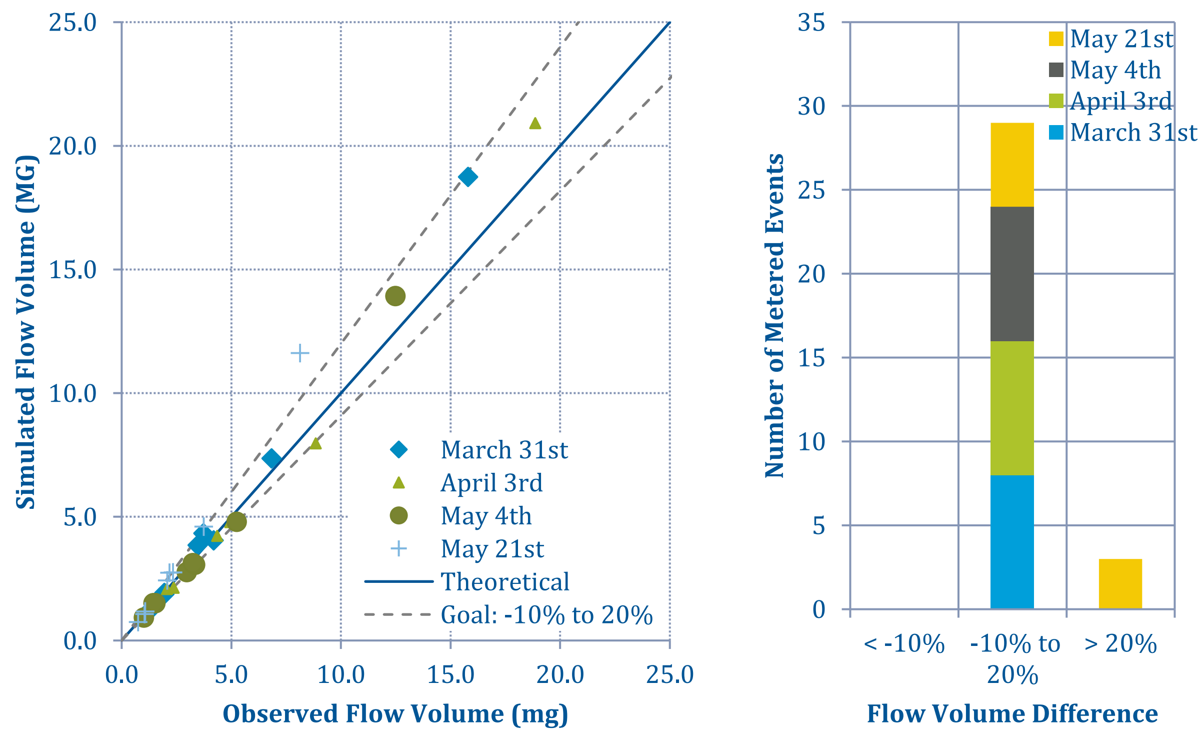


Figure 3-21 Volume Scatter Plot – Wet Weather

The model matches the observed peak depths, peak flows, and volumes within the target range for those flowmeters with valid and acceptable monitored data (Figures 3-18, 3-19, 3-20, and 3-21). Most data points fell near the theoretical 1:1 line indicating the model does not skew results either high or low based on the different calibration storms. Several of the surcharged depths were lower than the observed values (Figure 3-19). However, the peak depths matched for at least one of the events at each meter and the timing of the peak depths also matched. In general, the calibration scatter plots demonstrate the model meets the wet weather calibration guidelines.

Table 3-12 presents a summary of the calibration for each meter. In general, the calibration was successful with the calibration of each meter having a moderate to high confidence level. Overall, the response is balanced. The results balanced the peak flows during the different storms so that the model slightly underpredicted for one storm and slightly over-predicted for another. The greatest discrepancies in the calibration were surcharge storm flow depths, which were mostly caused by operations at the WWTP.

Table 3-12 Calibration Summary

METER	DESCRIPTION	CALIBRATION GOALS	CONFIDENCE / CALIBRATION RESULTS
1	18-inch outfall west of Clear Creek Rd, north of Carolina Village Rd	Reasonably matched flow and depth through each calibration event.	High – This location calibrated well for volume and peak flow for all four events. All flow parameters were within the model goals. The depth during the storms was backed up from the downstream hydraulic conditions at the WWTP resulting in a poor match on the depth.
2	42-inch Interceptor west of Pinehurst Drive	Reasonably matched flow and depth for each calibration event. Because of possible overflows between the upstream meters and FM2, the goal was to match depth at this location.	Moderate – The calibration for FM2 was based on the DWFs for the meter basin plus the total wet weather flows from the three upstream basins. Due to operations at the WWTP, this portion of the sewer surcharges during wet weather events. The calibration indicated that there was no I/I contributed, so the larger peak flows and volumes produced by the model likely result from the model not losing flow between the upstream meters and the FM2 location.
3	24-inch outfall east of Asheville Highway, near Oakhurst Street (upstream of FM2)	Reasonably matched flow and depth through each calibration event.	High – This location calibrated well for depth, volume, and peak flow for all four events. All flow parameters were within the model goals. The depth during the first two storms was either backed up from the downstream WWTP or a possible blockage that subsequently washed out. The calibration for depth was a better match on the two final events.
4	12-inch outfall west of Orleans Avenue, South of Whitmire Circle (upstream of FM3)	Reasonably matched flow and depth through each calibration event.	High – This location calibrated well for volume and peak flow for all four events. All flow and depth parameters were within the model goals, except the peak flow on May 4. This flow discrepancy was likely caused by variability in rainfall in the FM4 basin.

METER	DESCRIPTION	CALIBRATION GOALS	CONFIDENCE / CALIBRATION RESULTS
5	24-inch outfall south of 1st Avenue East, upstream of Jackson Park Force Main discharge (upstream of FM2)	Reasonably matched flow and depth through each calibration event.	High – This location calibrated well for volume and peak flow for all four events. All flow parameters were within the model goals. The depth during the first two storms was backed up from surcharged trunk interceptor likely from operations at the WWTP. The flow volume on the May 21 event was low because of a malfunction in the flowmeter that caused gaps in the data.
6	18-inch outfall crossing West Allen Street (upstream of FM5)	Reasonably matched flow and depth through each calibration event.	High – This location calibrated well for volume and peak flow for all four events. All flow parameters were within the model goals. The model under predicted the depth during the first two storms, but over predicted depths during the two May events balancing the model results.
7	24 inch Interceptor south of New Hope Road, near Powell Street (upstream of FM2)	Reasonably matched flow and depth through each calibration event.	High – This location calibrated well for volume and peak flow for all four events. All flow parameters were within the model goals. The model under predicted the depth during some of the events, but over predicted depths on others.
8	18 inch outfall southwest of Spartanburg Highway, southeast of Shepard Street, near the abandoned Rhodys pump station	Reasonably matched flow and depth through each calibration event.	High – This location calibrated well for depth, volume, and peak flow for all four events. All parameters were within the model goals. The peak flow was slightly out of the range for the March 31 event, but the results were balance by higher flows predicted during the May events.

3.5 CONCLUSIONS

The planning analysis was completed using a skeletonized model inclusive of all pipes with diameters 10 inches and larger. The developed model and plan are useful tools for the collection system which allow for the following:

- Expanded system knowledge.
- Analyze collection system improvements more accurately.
- Assess the impact of new developments and loads on the collection system.
- Develop a more accurate CIP.
- Create a dynamic master plan that can be adjusted as additional knowledge is gained.

3.5.1 Model Limitations

A model is only as accurate as the data used to develop and calibrate it. While the model can adequately simulate monitored conditions in the collection system, there are certain limitations that the COH should be aware of as it continues to update and apply the model.

The InfoSewer platform does not allow for sediment depth to be added to pipes during calibration and capacity assessment. Significant sediment would not only cause the pipe segments to be rougher than a clean pipe, but the actual cross section of the pipe would be reduced resulted in reduced pipe capacity. Cleaning and maintenance of the pipes with noted sediment depths is recommended as part of the COH's ongoing collection system strategy. Sediment was noted in the field at the FM5 and FM6 locations.

The WWTP operation was simulated by the addition of a wetwell and fixed rate pump at the downstream end of the model. The maximum pumped flow from the model was 6.5 MGD. The actual operation of the influent pump station has a significant impact on the depths of flow in the 42 inch interceptor including predicted SSOs in the system. Depths should be monitored in the collection system following any changes at the WWTP. When constraints at the WWTP are mitigated, a level of I/I should be introduced to the FM2 basin during capacity analysis. The significant surcharging monitored during the rain events indicates that the downstream collection system is operating in pressurized conditions which prevents estimating rainfall-dependent I/I for this portion of the system. However, with lower flow depths in the sewers and manholes, some level of I/I would be expected in the area. An estimate for the I/I rate in the FM2 basin for use in the capacity assessment phase should be based on a system average I/I rate.

4.0 Flow Projections

The purpose of this chapter is to present the flow projections that were developed for the base year (2017) and future planning years (2025 through 2040). The following data was used to develop flow projections:

- Historical plant flows.
- Spatially distributed traffic analysis zone (TAZ) polygons from Land of Sky Regional Council that include population and employment projections.
- 2010 and 2040 French Broad River MPO (FBRMPO) TAZ projection data from Land of Sky Regional Council.
- City of Hendersonville (COH) 2017 Water Master Plan.
- Areas of historically failing septic systems provided by Seth Swift, Environmental Health Supervisor with the Henderson County Board of Health.
- Private wastewater treatment plant (WWTP) flows: http://www.epa.gov/enviro/facts/pcs-icis/search.html.
- Industrial and commercial development areas provided by the City partnership.
- Historical precipitation data from the United States Geological Survey (USGS): https://waterdata.usgs.gov/nc/.
- Historical precipitation data from the National Oceanic and Atmospheric Administration (NOAA): https://www.ncdc.noaa.gov/cdo-web/search?datasetid=PRECIP_HLY.
- Henderson County 2020 Comprehensive Plan
- The City of Hendersonville's 2030 Comprehensive Plan
- Stakeholder meeting on April 27, 2018 that included Town of Laurel Park, Henderson Co. Partnership for Economic Development, and Henderson County Schools.

4.1 SANITARY SEWER SERVICE AREA

The COH service area is in Henderson County in western North Carolina. The primary municipalities served are Hendersonville, Flat Rock, and Laurel Park. The WWTP is in the Mud Creek Basin and serves the area that drains to the WWTP by gravity. Figure 4-1 shows the existing service area boundary. The 2000 agreement between the County and COH, and the 2002 addendum defined the future boundaries of the County's Cane Creek and the COH sewer service area. The agreement defined the ultimate service area of the Hendersonville WWTP as the entire Mud Creek basin that extends east of the city as shown on Figure 4-2. The COH service area is bordered to the north by Henderson County's Cane Creek service area.

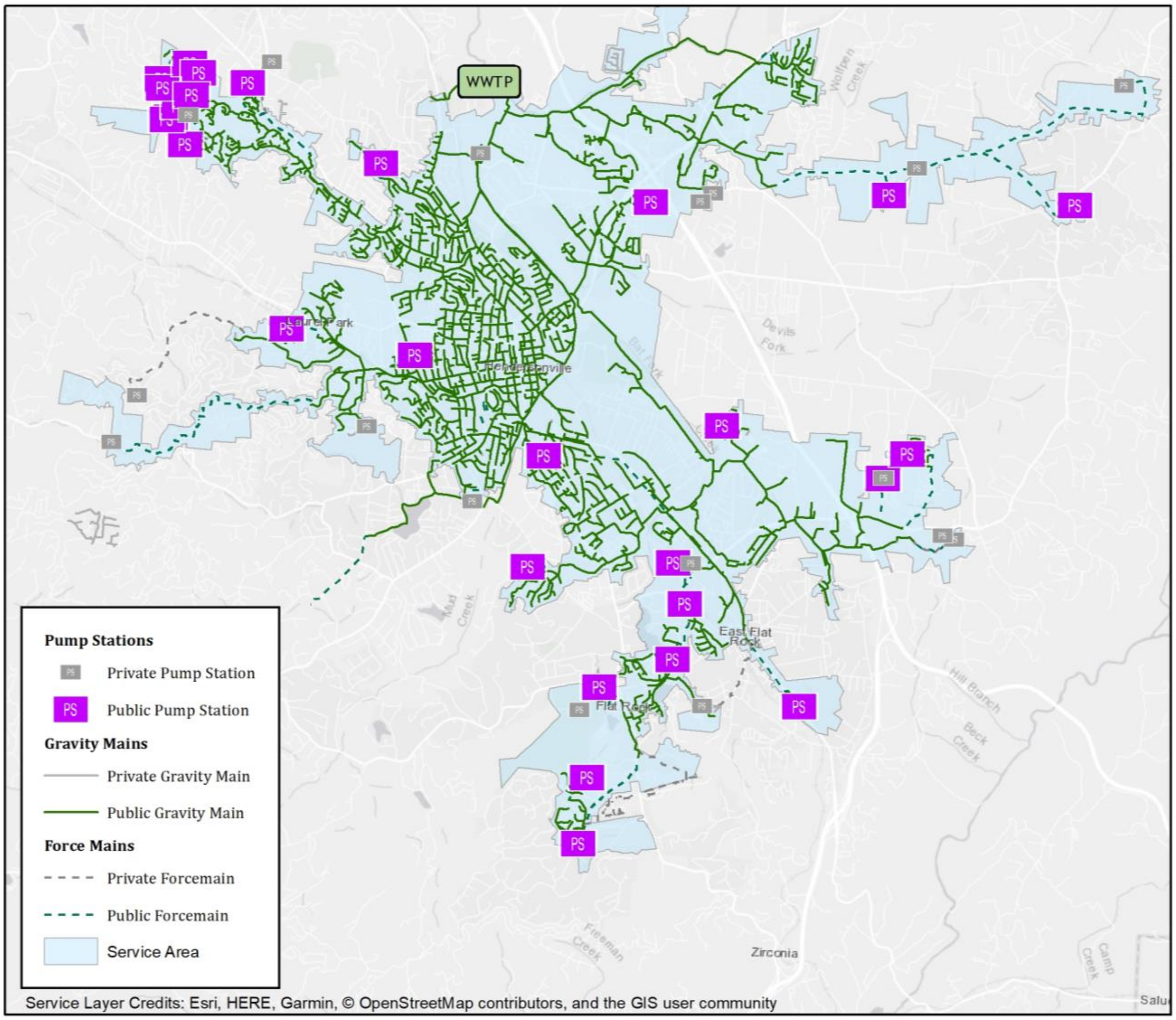


Figure 4-1 Existing Sewer Service Area

Henderson County's comprehensive plan defined three areas in their Growth Management Strategy: The Urban Services Area (USA), the Rural/Urban Transition Area (RTA), and the Rural Agricultural Area (RAA). The County recommended that investment in water and sewer infrastructure should be focused on the USA through at least 2020. In the future, investment is expected to expand from the USA into the RTA in response to development and extending services to existing schools, industries, commercial properties, or residential areas with failing septic systems. The future service area for COH sewer system was discussed at the stakeholders meeting held April 27, 2018 and during subsequent discussions with partnership and Henderson County Environmental Services. The 2040 service area was planned to include the USA outside of the Cane Creek Service area, the Upward Road area, and Chimney Rock Road area. The 2040 service boundary is shown on Figure 4-2. After discussions with the stakeholders, it was decided that the Mud Creek basin would serve as the boundary for the ultimate COH service area for long-term growth beyond 2040.

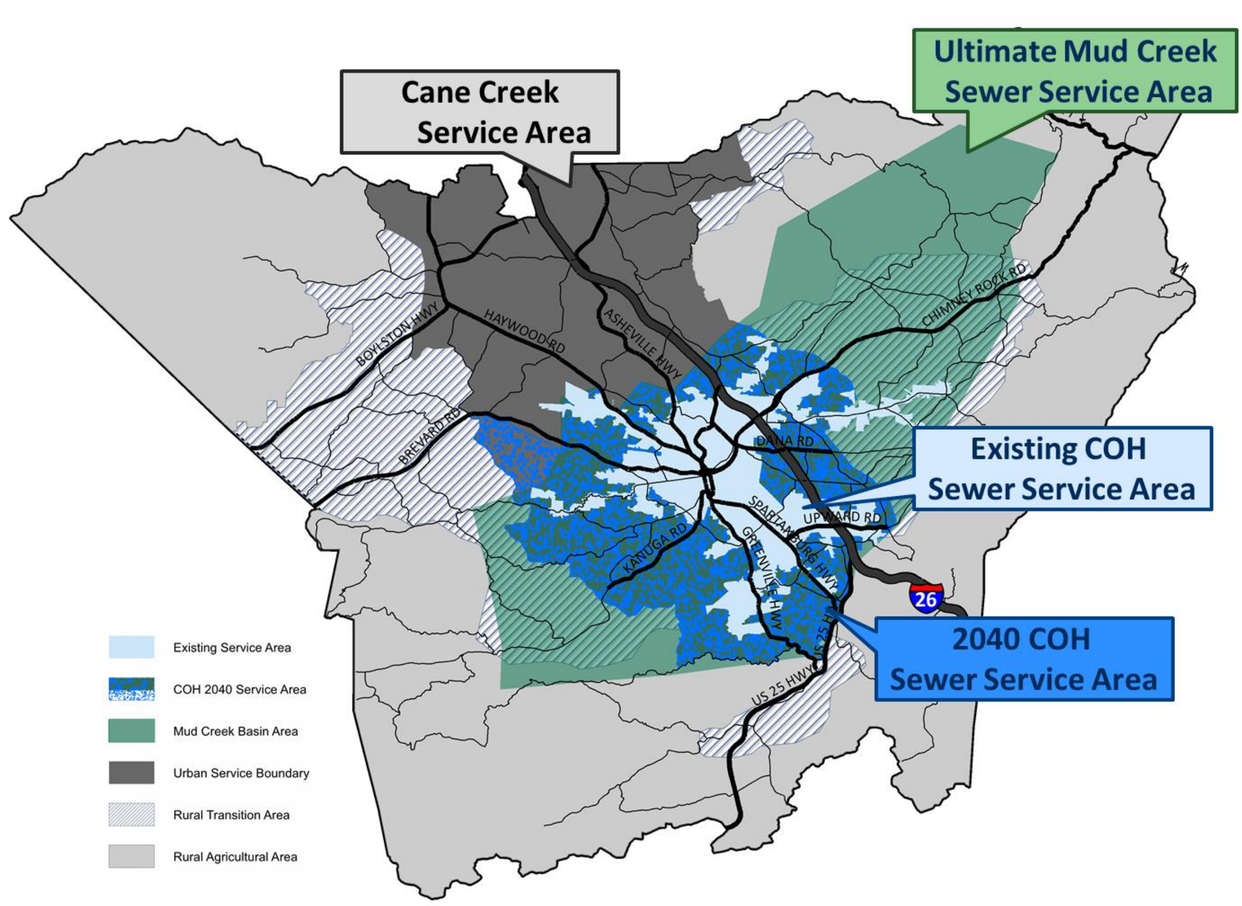


Figure 4-2 Hendersonville Service Area

4.2 REVIEW OF POPULATION AND EMPLOYMENT PROJECTIONS

The COH Water System Master Plan used data from the FBRMPO TAZ based population and employment projections to develop water demands. The same data was used to develop population and employment numbers for use in the flow projections for 2025 and 2040. The FBRMPO TAZ data included population projections for 2010 and 2040. These population and employee projections were linearly interpolated to develop the 2025 planning year. The population and employment density growth is shown below on Figure 4-3 and Figure 4-4, respectively.

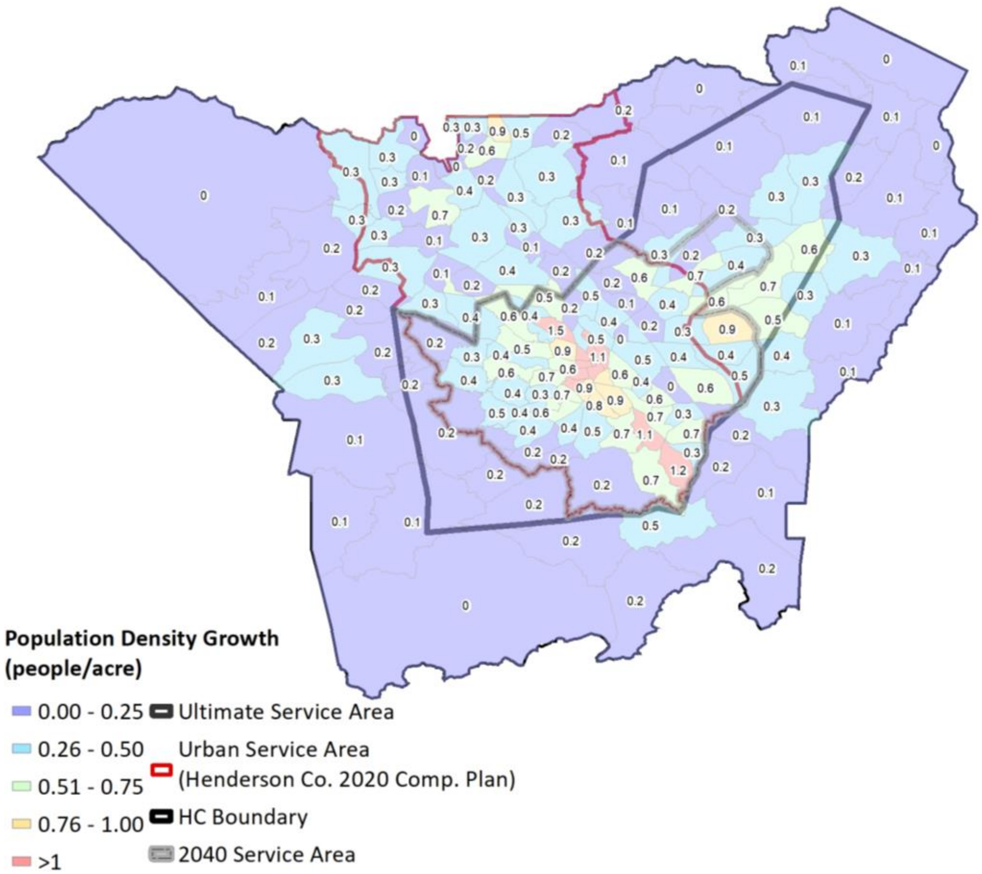


Figure 4-3 Population Density Growth

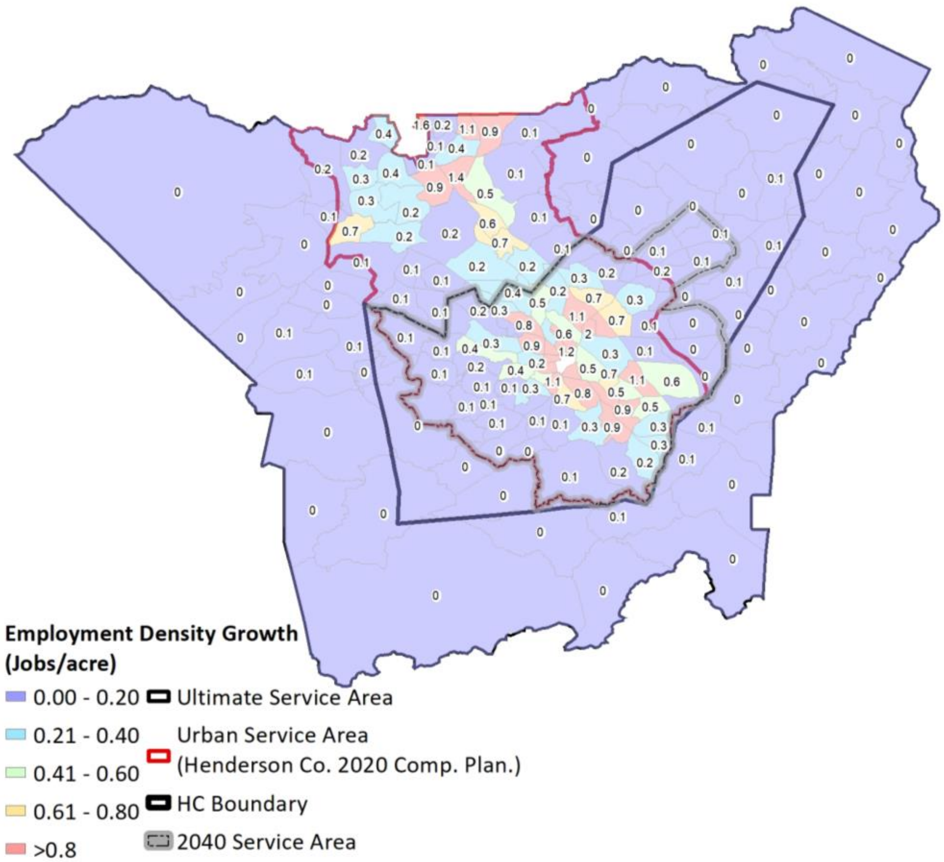


Figure 4-4 Employment Density Growth

The following tables break out the TAZ population, employment and household data by existing town boundaries. In the future, unincorporated areas that develop are expected to be annexed into one of the jurisdictions. Therefore, the total town and city populations will likely be larger than the 2040 population than indicated in the tables for the current jurisdictions. A good portion of the growth shown in the Unincorporated areas would actually become part of one of the jurisdictions, including the City of Hendersonville.

Table 4-1 TAZ Population Data

PLANNING BOUNDARY	2010 POPULATION	2040 POPULATION*
Flat Rock 2018 Village Limits	3,199	5,480
Laurel Park 2018 Town Limits	1,881	2,740
COH 2018 City Limits	11,223	15,481
Unincorporated Areas within service area	25,669	35,907
Chimney Rock Road Area and Upward Road	3,244	5,865
Total in COH Sewer System Service Area	**45,217**	**65,472**
Henderson County	**105,438**	**158,135**

*Limits based on No Annexation

Table 4-2 TAZ Employment Data

PLANNING BOUNDARY	2010 EMPLOYMENT	2040 EMPLOYMENT*
Flat Rock 2018 Village Limits	799	1,594
Laurel Park 2018 Town Limits	255	586
COH 2018 City Limits	11,779	16,792
Unincorporated Areas within service area	11,079	17,776
Chimney Rock Road Area and Upward Road	494	866
Total in COH Sewer System Service Area	**24,406**	**37,615**
Henderson County	**39,988**	**64,830**

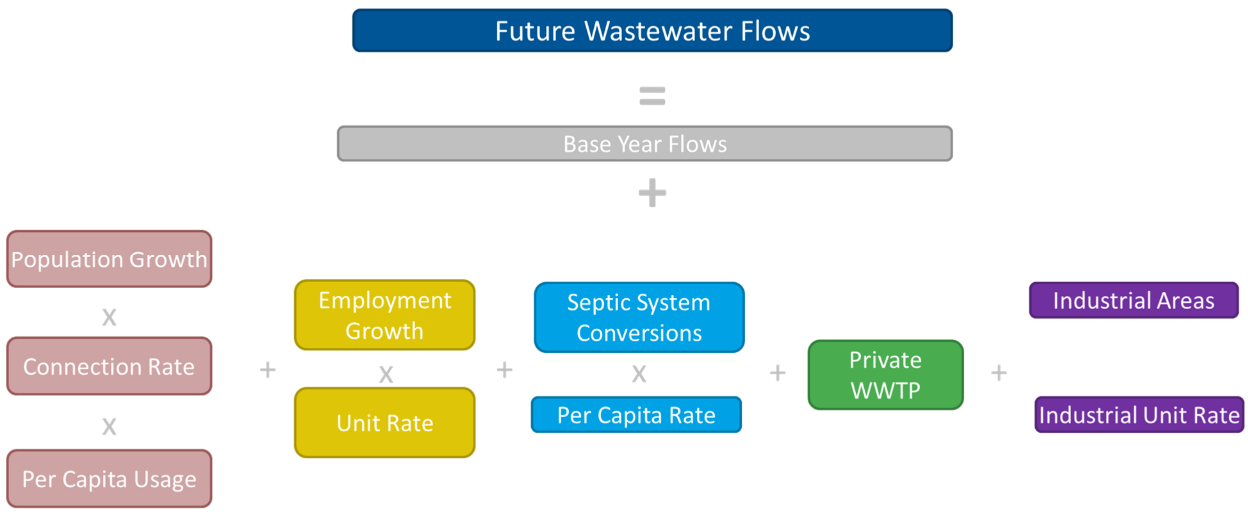
*Limits based on No Annexation

Table 4-3 TAZ Household Data

PLANNING BOUNDARY	2010 HOUSEHOLDS	2040 HOUSEHOLDS	2010 POPULATION PER HOUSEHOLD
Flat Rock 2018 Village Limits	1,538	2,548	2.08
Laurel Park 2018 Town Limits	904	1,284	2.08
COH 2018 City Limits	5,430	7,316	2.07
Unincorporated Areas within service area	11,357	15,890	2.26
Chimney Rock Road Area and Upward Road	1,429	2,588	2.27
Total in COH Sewer System Service Area	**20,658**	**29,626**	**2.19**
Henderson County	**45,448**	**68,776**	**2.32**

4.3 FUTURE FLOW PROJECTIONS METHODOLOGY

The future year flow projections were developed by combining historical wastewater flow rates, data from other COH planning studies, and the feedback received from the stakeholders. The base year flow to the Hendersonville WWTP was determined by analysis of historical flow data recorded at the plant. The potential for increased future flows from population and employment growth was determined using the TAZ data. Impacts to the future flows from the elimination of private WWTPs and the addition of future industrial customers were also considered. Additionally, locations of septic systems that were potential health concerns were identified as areas to convert from septic systems to public sewers in the future. The equation below shows how each portion of the flow contributes to the future wastewater flows.



Each component of the future year flows, as well as the factors and assumptions used in developing this equation, are described in more detail in the following sections.

4.4 BASE YEAR FLOWS

The first step in developing the future flow projections is to develop base year flows. Wastewater flows are highly correlated to rainfall data because of inflow and infiltration (I/I). It is important to evaluate the historical flows and historical rainfall data together to filter outliers and evaluate average annual flows.

4.4.1 Historical Precipitation

Henderson County lacks an abundance of historical rainfall data; therefore, historical gauge data from NOAA (1998 – 2010), a USGS gauge in Asheville (2010 – 2014), and the rainfall data provided by COH (2014 – 2017) were combined to generate historical precipitation data. Total monthly precipitation data was tabulated based on the average of the rainfall recorded at each gauge. Table 4-4 summarizes the precipitation data from 1998 through 2017.

Table 4-4 Estimated Historical Monthly Total Precipitation Data for the City of Hendersonville Service Area

YEAR	JAN (in.)	FEB (in.)	MAR (in.)	APR (in.)	MAY (in.)	JUN (in.)	JUL (in.)	AUG (in.)	SEP (in.)	OCT (in.)	NOV (in.)	DEC (in.)	ANNUAL* (in.)
1998	7.6	4.7	2.5	6.2	1.3	3.3	1.5	1.0	1.2	2.2	1.7	2.4	35.5
1999	4.6	3.1	2.4	2.6	2.5	2.6	3.9	0.9	1.7	1.5	3.5	1.1	30.3
2000	2.6	2.2	4.0	4.7	2.5	2.9	4.4	1.8	2.2	0.0	2.7	0.0	29.9
2001	2.3	2.4	3.9	1.6	2.3	2.5	5.0	2.5	3.7	0.7	1.3	2.0	30.2
2002	3.8	1.0	4.2	1.0	3.3	4.6	1.9	1.1	5.7	3.2	3.2	4.9	38.0
2003	1.4	4.6	4.2	4.3	8.2	3.9	4.7	4.6	3.2	2.0	4.4	2.9	48.3
2004	1.0	3.2	2.3	3.7	3.5	3.8	2.9	4.6	12.9	1.6	4.7	2.3	46.4
2005	2.0	2.8	3.8	3.5	2.3	6.3	6.6	5.5	0.5	1.1	3.6	2.6	40.6
2006	4.2	1.5	1.1	5.1	2.8	4.0	3.0	3.6	3.9	2.4	3.9	3.9	39.3
2007	1.8	1.0	3.5	1.6	0.7	1.5	3.7	1.1	2.7	2.0	1.2	3.0	23.6
2008	1.7	3.4	3.7	2.3	1.6	1.4	2.0	4.9	1.5	1.0	2.3	2.9	28.6
2009	1.9	1.9	4.9	4.3	6.8	5.7	3.5	5.1	9.9	5.2	5.9	4.6	59.7
2010	7.2	3.5	3.5	2.8	3.8	1.1	3.9	3.3	3.8	3.7	6.6	0.9	44.3
2011	2.1	3.1	7.3	5.5	1.2	2.8	3.2	5.0	5.4	2.0	5.7	4.1	47.2
2012	3.7	1.2	2.8	5.6	5.5	1.1	8.1	5.5	4.4	4.3	0.8	4.9	48.0
2013	8.6	2.2	3.0	6.1	6.6	8.3	16.1	5.7	2.4	1.5	3.3	7.6	71.6
2014	2.4	2.5	1.7	5.2	4.3	5.2	6.1	2.2	2.4	5.2	4.6	2.4	44.3
2015	3.3	2.5	2.1	0.9	0.9	4.8	1.7	2.7	5.0	9.8	9.0	8.5	51.2
2016	3.2	6.6	1.1	2.4	2.8	5.6	3.4	4.5	0.5	0.5	1.4	2.2	34.1
2017	3.1	0.8	4.8	8.2	8.2	2.9	7.1	5.9	4.7	9.5	1.0	2.4	58.3
Average	3.4	2.7	3.3	3.9	3.6	3.7	4.6	3.6	3.9	3.0	3.5	3.3	42.5

Monthly data is an average of USGS gauge 3451500, NOAA data, and COH rainfall data.
*Annual is sum of monthly averages.

The average annual precipitation was 42.5 inches from 1998 to 2017. The highest precipitation years were 2009 and 2013. In contrast, the lowest precipitation years were 2007 and 2008 with less than 30 inches of rain. Statistically, the closest year to the 20-year average for rainfall totals was 2002. Figure 4-5 shows a graph of the historical precipitation.

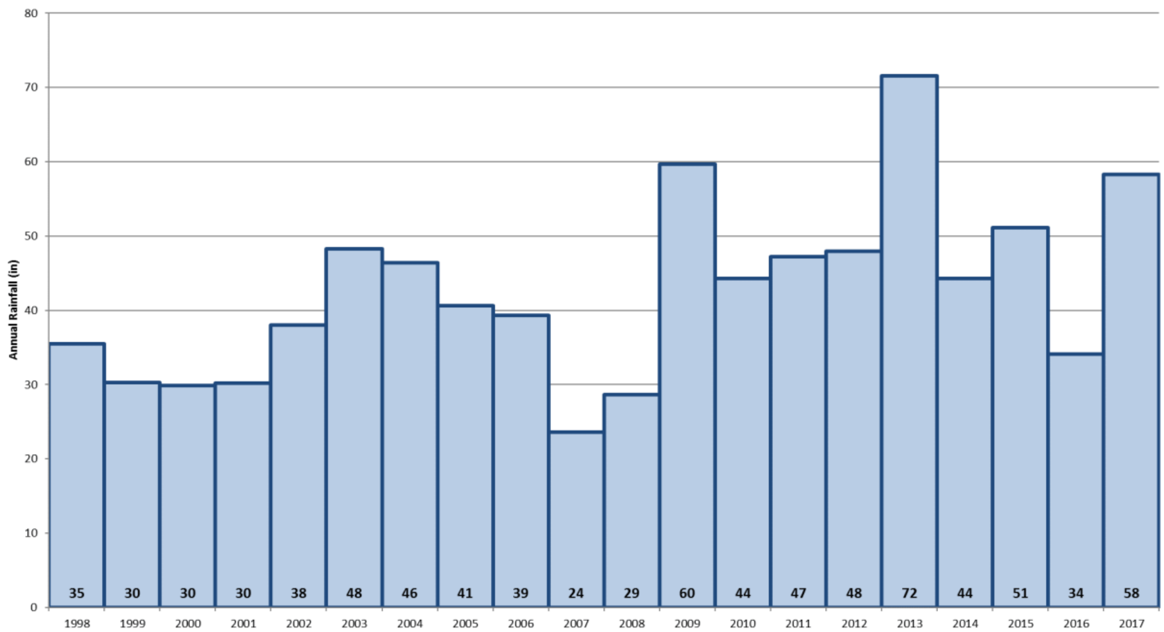


Figure 4-5 City of Hendersonville Historical Precipitation

4.4.2 Base Year Flow and Maximum Month Peaking Factor

The base year flow serves as the foundation for the future flow projections. The projected flows for each planning year will be added on top of the base year flow. The base year flow should reflect reasonably current collection system conditions and rainfall impacts. The base year flow approach incorporates the present I/I rates into the total flow.

WWTP permitted capacity is based on maximum month flow, so the forecasted annual average future flow will need to be peaked in a similar fashion to assess the plant capacity in the future planning years. Maximum Month Peaking Factors (MMPFs) can be derived for the selected base year, either using the historical maximum or a selected value somewhere within the range experienced for the Hendersonville WWTP.

To explore both key factors, 20 years of WWTP effluent flow data were analyzed. The historical average annual flow and the MMPFs are summarized in Table 4-5 and presented on Figure 4-6.

Table 4-5 Historical WWTP Annual Average Flow and Maximum Month Peaking Factors for Hendersonville WWTP

YEAR	ANNUAL AVERAGE FLOW (MGD)	MAXIMUM MONTH FLOW[1] (MGD)	MMPF[2]
1998	2.58	2.70	1.05
1999	2.82	3.32	1.18
2000	2.74	3.35	1.22
2001	2.84	3.19	1.12
2002	2.70	3.88	1.44
2003	3.62	4.47	1.24
2004	3.31	4.92	1.49
2005	2.79	4.29	1.53
2006	2.69	3.18	1.18
2007	2.48	3.94	1.59
2008	2.40	3.16	1.32
2009	2.84	4.37	1.54
2010	2.42	4.60	1.90
2011	2.33	3.21	1.38
2012	2.54	3.11	1.22
2013	3.33	4.42	1.33
2014	2.94	3.86	1.31
2015	3.14	4.18	1.33
2016	3.08	4.24	1.38
2017	2.88	3.31	1.15
Average	2.82	3.79	1.34
5 Year Average[3]	3.07	4.00	1.30

(1)Based on a 30 day rolling average.
(2)MMPF = Maximum Month Peaking Factor.
(3)Based on 2013-2017.

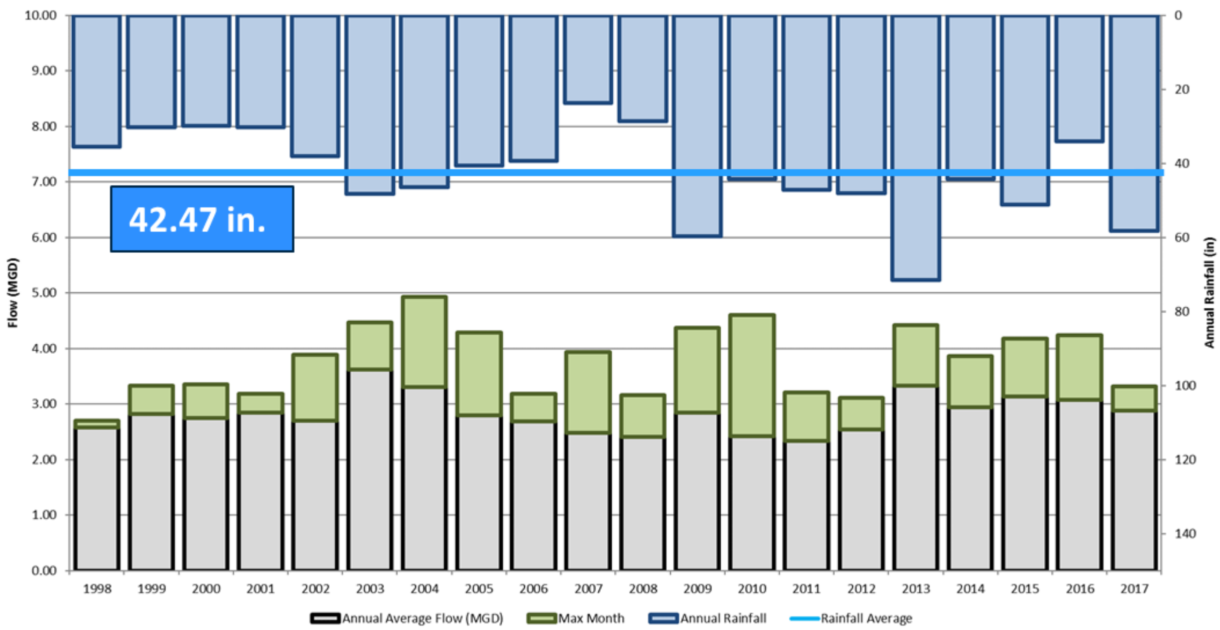


Figure 4-6 Historical Rainfall and Average Annual and Maximum Month Flows

The approximate average flow to the Hendersonville WWTP from 1998 to 2017 was 2.82 MGD, and average over the last 5 years was 3.07 MGD. The 5-year average was used to capture dry, wet, and average flows that occurred recently, and was representative of the base year flow. The 5-year average MMPF of 1.30 for Hendersonville WWTP flows was selected as representative of the sewer conditions that contribute to the peak flows to the plants. The maximum peaking factor for the 20 years of data was 1.90 in 2010. This was considered an outlier because it was much higher than the average peaking factors both over the last 5 years and the 20-year history.

4.5 FUTURE YEAR FLOWS

4.5.1 Flows from New Population

Future year population flows were developed from the population numbers using unit factors that are representative of state guidance and local experience. The incremental population growth, the population growth from the 2017 to the respective planning years, was calculated from the TAZ population data. The per capita wastewater flows can be derived from the per capita water usage because a majority of water demands becomes waste. The recently completed 2017 Water System Master Plan Report found that the average annual residential water use was 84 gallons per capita per day (gpcd). The flow metering performed during Phase 1 of the SSAIA showed recorded higher wastewater flow volumes than the corresponding water usage from the geocoded billing data. Since the wastewater flow monitoring in the existing system did not indicate any consumptive losses, the per capita wastewater flow rate was set to match the values used in 2017 Water System Master Plan Report. For consistency with the water plan, 84 gpcd was used to account for returned wastewater from new customer accounts and for groundwater infiltration driven by the new pipes extended to new customers. Another factor to consider is the rate that new population growth would connect to public sewer. Following discussion with COH staff, it was assumed that 70 percent of new population growth will have a connection to public sewer, while the other 30 percent are expected to connect to septic tanks or private systems within the COH 2040 service area. Table 4-6 shows a summary of population flows.

Table 4-6 Population Summary Table

YEAR	INCREMENTAL POPULATION GROWTH FROM 2017	PERCENT SEWERED (%)	SEWERED POPULATION	PER CAPITA RATE (GPCD)	FLOW (GALLONS)
2025	5,401	70	3,781	84	317,599
2040	15,529	70	10,870	84	913,098

4.5.2 Flow from New Employment

TAZ employment projections were the basis of the future year employment flows. The TAZ projections were used to estimate flows from employment in offices or retail establishments. Flow from new employment in industrial applications will likely be inside the growth areas identified by COH that are analyzed and accounted for later in Subsection 4.5.4. However, there will be some employment growth outside the designated industrial growth areas in nonindustrial capacities. NC DEQ 15A NCAC 02T .0114 (d) estimates employment flows at 25 gpcd, Therefore, the per capita rate for nonindustrial wastewater usage was assumed to be 10 gallons per new employee per day. Table 4-7 shows a summary of the employment analysis.

Table 4-7 Employment Flow Summary

YEAR	INCREMENTAL EMPLOYMENT GROWTH FROM 2017	PER CAPITA RATE (GPCD)	FLOW (GALLONS)
2025	3,522	10	35,223
2040	10,127	10	101,267

4.5.3 Septic Conversion Rates

Septic system areas likely to convert to public sewer within the existing service area were identified by the Environmental Health Supervisor with the Henderson County Board of Health. There were 2,281 households identified within these septic areas, or 21 percent of the existing un-sewered population. By 2025, 24 percent of the households within the septic areas were assumed to convert to public sewer. This is an annual conversion rate of 60 homes per year. This rate is expected to continue to 2040, which results in 1,360 total septic conversions from 2017 to 2040. Table 4-8 shows a summary of septic conversions, and Figure 4-7 shows the locations of these areas.

Table 4-8 Septic Conversion Summary

YEAR	INCREMENTAL SEPTIC CONVERSIONS (HOUSE HOLDS) FROM 2017	INCREMENTAL SEPTIC CONVERSIONS (PERSONS) FROM 2017	PER CAPITA RATE (GPCD)	FLOW (GALLONS)
2025	560	1,069	84	89,797
2040	1360	3,029	84	254,418

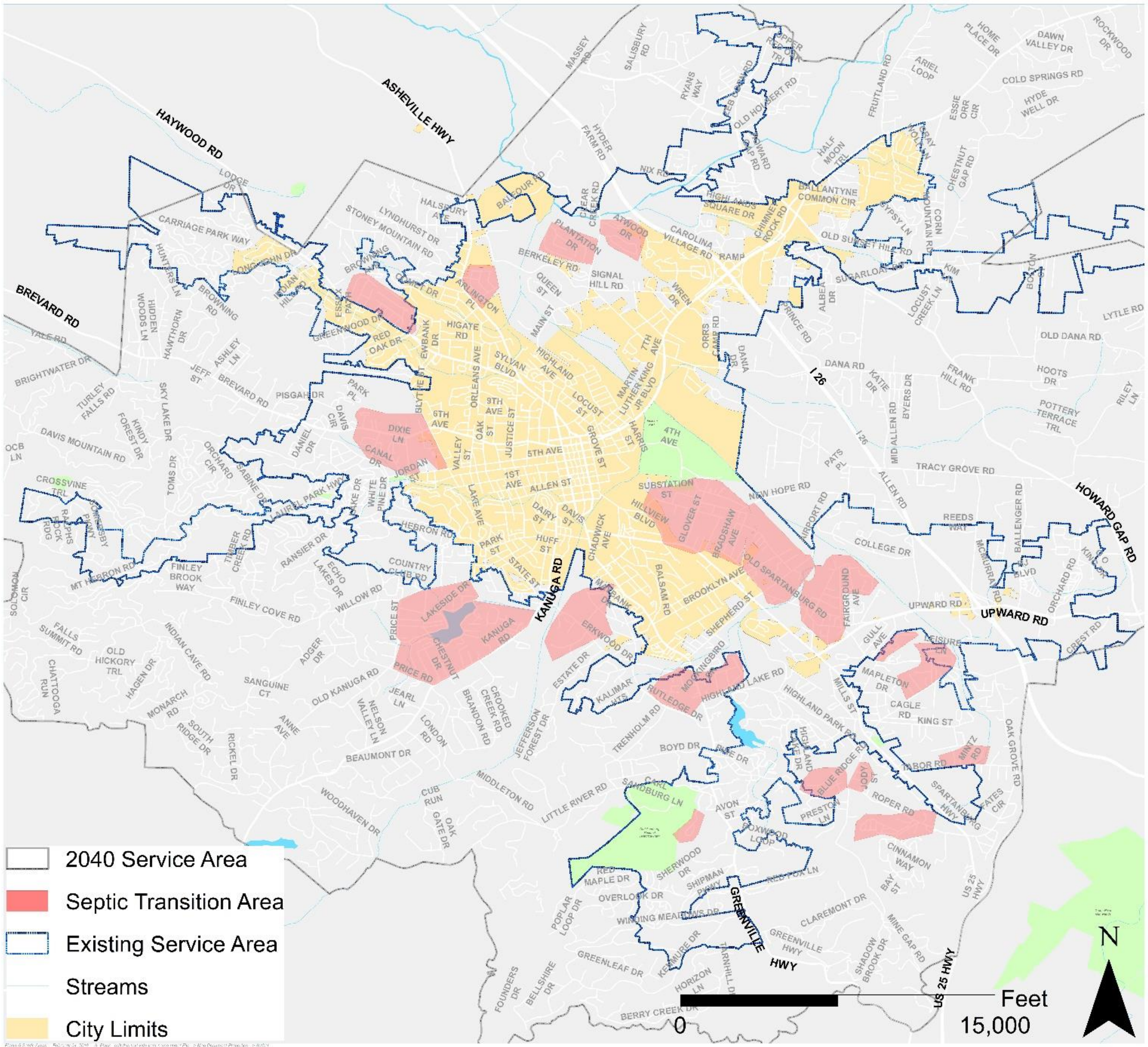


Figure 4-7 Areas of Septic Transition

4.5.4 Industrial Flows

The Henderson County Partnership for Economic Development provided a GIS shapefile with areas for potential industrial and commercial development. There were 2,547 total acres identified for potential development. The area within the 2040 COH service area was 1,481 acres. The projected industrial/commercial flow was estimated using 880 gallons per acre as specified by NC DEQ 15A NCAC 02T .0114 (d) for non-residential uses. Table 4-9 shows the summary of industrial flow data.

Table 4-9 Industrial Flow Summary

YEAR	INDUSTRIAL AREA (ACRES)	PER CAPITA RATE (GALLONS/ACRE)	FLOW (GALLONS)
2025	740	880	651,477
2040	1,481	880	1,302,953

4.5.5 Private Wastewater Treatment Plants

The North Carolina Department of Environment Quality (NCDEQ) Division of Water Quality maintains a list of active National Pollutant Discharge Elimination System (NPDES) permits in the state. There are 28 private WWTPs in Henderson County, including 10 private facilities within the future service area. Table 4-10 lists the Henderson County facilities along with their permitted flow and location. The locations of the private WWTPs within the service area are shown in orange on Figure 4-8, and are listed in bold in Table 4-10.

Table 4-10 Private Wastewater Treatment Plants in Henderson County

NAME	PERMIT FLOW (GPD)	RECEIVING STREAM	LOCATION
Camp Highlander	7,400	South Fork Mills River	Henderson County
Etowah Sewer Company WWTP	125,000	French Broad River	Henderson County
Country Acres MHP WWTP	6,000	McDowell Creek	Henderson County
Mountain Valley WWTP	20,000	French Broad River	Henderson County
Riverwind Mobile Home Park	72,000	French Broad River	Henderson County
High Vista Falls WWTP	45,000	Line Creek	Henderson County
Rosewood Mobile Home Park	20,000	Line Creek	Henderson County
Cummings Cove WWTP	80,000	French Broad River	Henderson County
Blacksmith Run WWTP	89,000	Lewis Creek	Henderson County
Blue Star Camps WWTP	60,000	Mud Creek	Henderson County
Bear Wallow Valley MHP WWTP	10,000	Clear Creek	Henderson County
Kanuga Conferences WWTP	35,000	Little Mud Creek	Henderson County
Fletcher Academy WWTP[2]	100,000	Byers Creek	Henderson County
Benson Apartments	8,000	Mud Creek	Henderson County
Henderson's Assisted Living WWTP	7,000	Featherstone Creek	Henderson County
Mountain View Assisted Living	5,000	Featherstone Creek	Henderson County
Brookside Village Condos WWTP	5,000	Featherstone Creek	Henderson County

NAME	PERMIT FLOW (GPD)	RECEIVING STREAM	LOCATION
Six Oaks Complex	20,000	Green River	Henderson County
Greystone Subdivision[1]	**21,700**	**Clear Creek**	**COH Service Area**
Bon Worth WWTP	**6,000**	**Allen Branch**	**COH Service Area**
Pine Park Retirement Inn	**35,000**	**Clear Creek**	**COH Service Area**
Magnolia Place WWTP	**22,000**	**Clear Creek**	**COH Service Area**
Dana Hill WWTP	**30,000**	**Devils Fork**	**COH Service Area**
Camp Judaea WWTP[1]	**30,000**	**Henderson Creek**	**COH Service Area**
Hunter's Glen WWTP	**35,000**	**Shaw Creek**	**COH Service Area**
Champion Hills WWTP	**70,000**	**South Fork Big Willow Creek**	**COH Service Area**
Edneyville Elementary [1]	**5,000**	**Henderson Creek**	**COH Service Area**
Justice Academy [1]	**5,000**	**Henderson Creek**	**COH Service Area**
Total County Permitted Flow		**974,100 gpd**	
Total Permitted Flow In COH Service Area		**259,700 gpd**	

[1]Assumed to connect to public sewer by 2025 for planning purposes.
[2]Currently Being Connected to Cane Creek

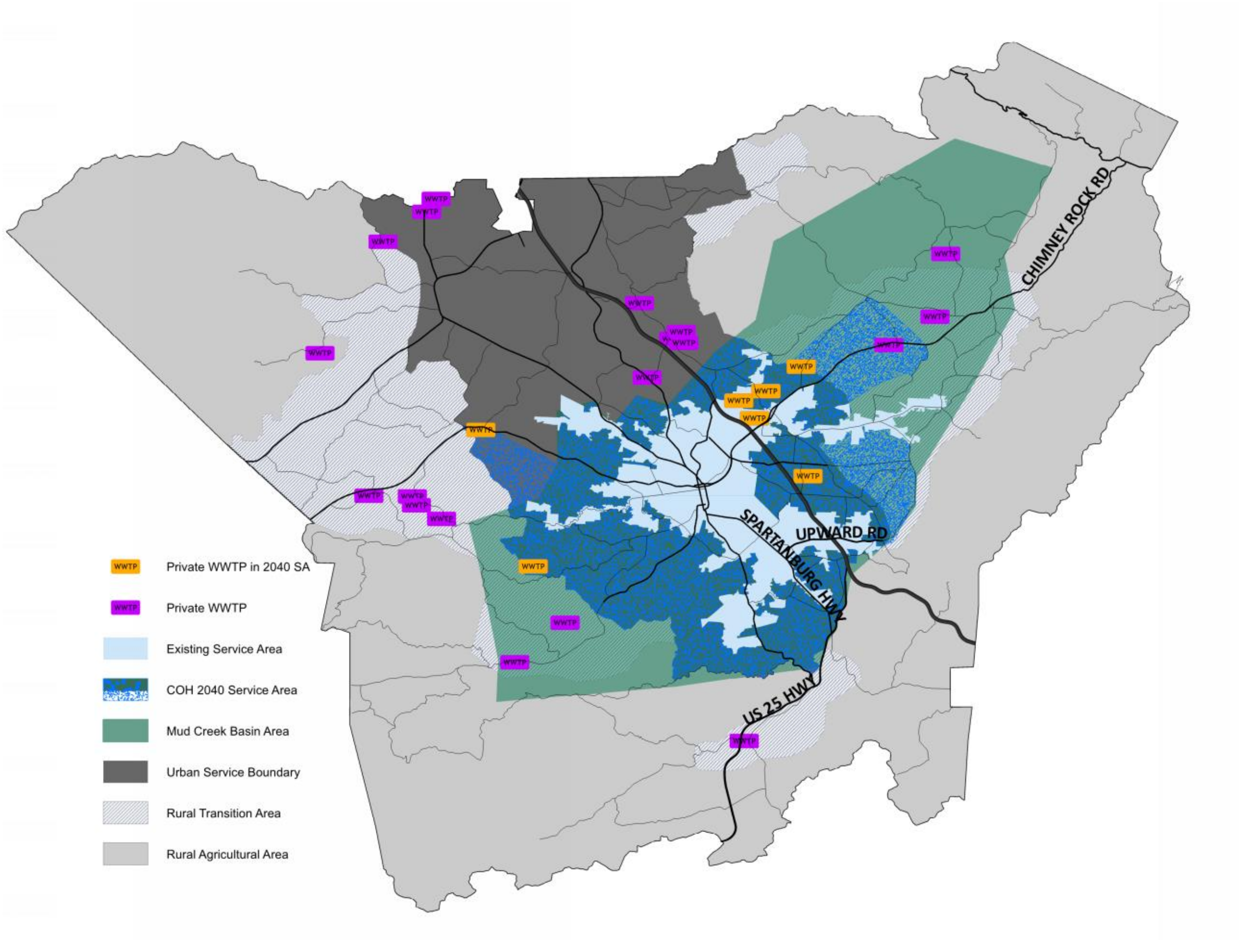


Figure 4-8 Private Wastewater Treatment Plants in Henderson County

4.5.6 Future Infiltration/Inflow

The amount of maintenance a utility will perform on a sewer collection system is a key assumption in evaluating the I/I component of the future flows. Without adequate maintenance and rehabilitation activities, the COH collection system will deteriorate in the future with the potential for the rate of I/I to increase over time, while ongoing, iterative assessment and rehabilitation of the collection system will keep I/I within reasonable limits. COH has ongoing condition assessment goals. Paired with a robust rehabilitation and replacement program, newly installed lines, better build material and construction practices, and aggressive inspections, COH can expect to maintain their current rate of I/I in the future. The flow projections and the future year models will assume that the groundwater infiltration rates and the rainfall dependent infiltration and inflow rates will stay constant through the planning period. The infiltration rates were discussed in the Section 3.

4.6 AVERAGE ANNUAL FLOW PROJECTIONS

The flow projections are developed from the total of all the sources included in Section 4.5. Table 4-11 shows each flow source and the respective flow increase from the 2017 annual average dry weather base year flows. COH's wastewater flows are projected to increase 1.2 MGD by 2025 and 2.8 MGD by 2040.

Table 4-11 Flow Projections: Incremental Increase from 2017

FLOW SOURCE	2025 ADDITIONAL FLOW (GALLONS)	2040 ADDITIONAL FLOW (GALLONS)
Population Flow	317,599	913,098
Employment Flow	35,223	101,267
Septic Conversions Flow	89,797	254,418
Industrial Flow	651,477	1,302,953
Private WWTP Flow	61,700	259,700
Total	**1,155,796**	**2,831,436**

4.7 HENDERONSVILLE WWTP FLOWS

The 5-year annual average flow to the Hendersonville WWTP served as the base flow. This was 3.07 million gallons per day (MGD). The incremental flow projections for the 2040 service area were added to the base flow to determine the future average annual flows to the WWTP. The average annual flow projections are listed in Table 4-12. The projected maximum month flows to the Hendersonville WWTP, which are based on the 5-year maximum month peaking factor of 1.30, are presented in Table 4-13. A graphic representation of average flow projections for the WWTP in relationship to the plant's permitted capacity at 80 and 100 percent is shown on Figure 4-9. The Hendersonville WWTP has a 4.8 MGD discharge permit that allows for system upgrades and discharges up to 6.0 MGD. The maximum month projections are shown against the permitted 6.0 MGD on Figure 4-9. The maximum month flow will surpass the plant capacity (4.8 MGD) in 2021 and the discharge permit capacity in 2028. The average flow surpasses the plant capacity (4.8 MGD) in 2030 and the discharge capacity in 2040.

Timing of plant expansions is dictated by the permit capacity and 15A NCAC 02T.0118, often referred to as the 80/90 Rule. The 80/90 rule states that prior to exceeding 80 percent of the wastewater treatment system's permitted hydraulic capacity based on average flow of the last calendar year, an evaluation on meeting future wastewater needs must be submitted to the State. Additionally, at 90 percent plant capacity, final plans and specifications for expansion must be submitted and approved. Based on the 80/90 Rule, COH should be ready to submit an evaluation of their future treatment needs and outline plans going forward by the time the average annual flow exceeds 80% of the permitted treatment capacity (3.84 MGD) in 2022.

However, it can be seen from Figure 4-9 that there is a possibility that the max month flows will exceed the plant capacity by 2021. This is sooner than the 80/90 rule. To reduce the risk of violating the permit during a single month, an expansion of the WWTP is recommended to occur by 2021. The max month flows are projected to exceed the 6.0 MGD discharge capacity by 2028

Table 4-12 Average Annual Flow Projections (MGD)

	2017	2025	2040
COH Service Area	3.07	4.23	5.90

Table 4-13 Maximum Month Total Flow Projections (MGD)

COH SERVICE AREA	2017	2025	2040
5 Year Average Maximum Month PF 1.30	4.00	5.50	7.68

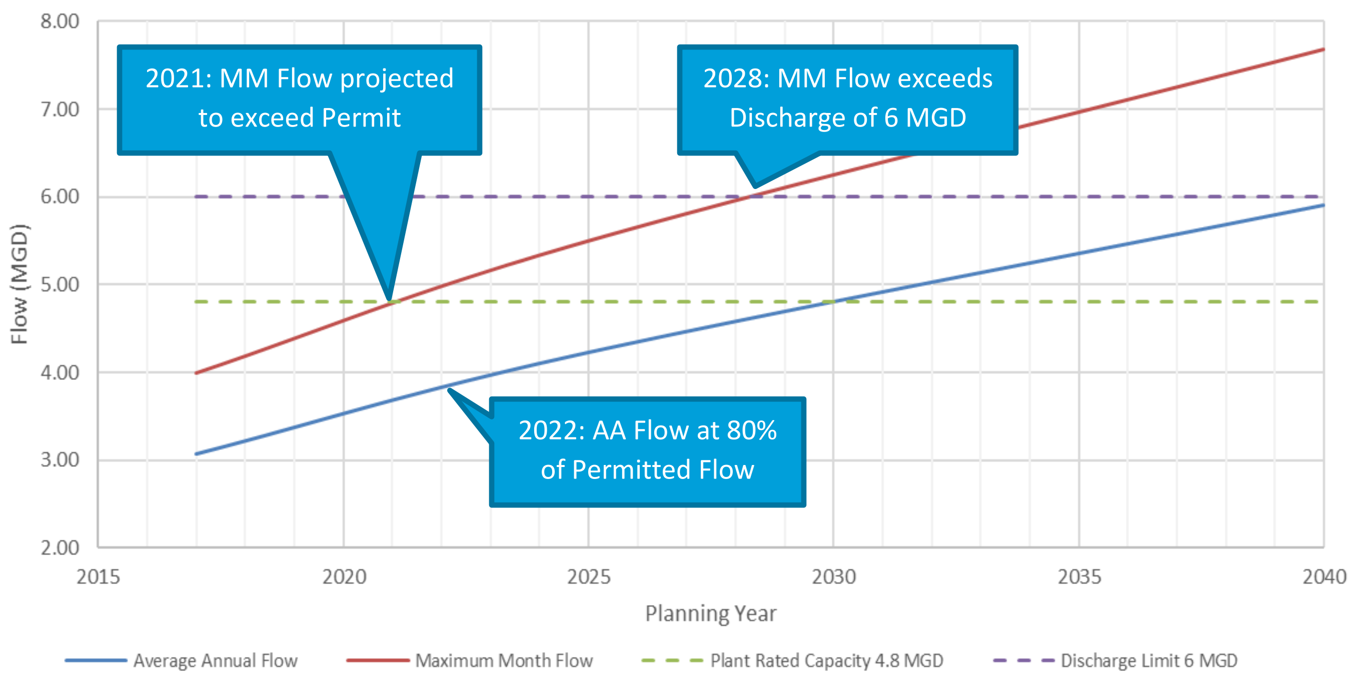


Figure 4-9 Hendersonville WWTP Flow Projections

5.0 Capacity Assessment

After assessing the condition of COH's system, COH's calibrated hydraulic model was used to evaluate the available capacity in its collection system. Section 3 documented the calibration process for the model. The calibrated model was developed for a "snapshot" in time corresponding to the conditions observed during the flow monitoring period. This process validated that the model could be used to accurately predict existing flow conditions and is the basis for future modeling evaluations. The objective of this section is to evaluate the existing and future capacity of the City's collection system and to provide collection system improvements that safely mitigate the capacity constraints. The capacity assessment consisted of updating the model base and future year flows, developing design and trigger criteria, and performing the analysis for each planning year.

5.1 BASE YEAR AND FUTURE YEAR FLOWS

The model was updated to match the base year system flow and the flow projections. The base year and future dry weather flow projections for the City's wastewater collection system were developed as part of Section 4. Table 5-1 contains the average annual flows summarized for each planning year from the flow projections for the Mud Creek Basin, which flows to the City's WWTP.

Table 5-1 Average Annual Flow Projections (MGD)

AREA	YEAR		
	[1]BASE YEAR	2025	2040
COH Service Area (Hendersonville WWTP)	3.07	4.23	5.90

[1]Base year flows were the 5-year annual average flow to the COH WWTP. See Table 4-5.

The projected loadings included population growth, employment growth, redevelopment, industrial development, septic conversions, and private WWTP connections as discussed in Section 4. The future year model networks were created for each planning year by allocating the projected loadings to the model manholes. Loadings from system expansion were assumed to reach the existing modeled system through future gravity sewers.

In order to evaluate the collection system under peak flow conditions, wet weather model scenarios were developed. The wet weather flows for the hydraulic model are dependent on the contributing catchment areas, and therefore any additional area needed to be allocated to the model nodes similarly to the loadings. The contributing area in the system increases as the system expands to new customers. A ratio of the average contributing area per unit of dry weather flow in the calibrated model was used as a factor to determine future contributing areas as shown below:

$$\frac{\text{Base Contributing Area}}{\text{Base Dry Weather Flow}} * \text{Increase Dry Weather loadings} = \text{Increase in Contributing Area}$$

The contributing area factor (base area/base DWF) was 1,983 acres per MGD.

The ground water infiltration (GWI) also needed to be adjusted. The base year flow, as documented by the flow projections, was determined to be 3.07 MGD. This represents an annual average flow based on the past 5 years of historical records provided by the city. The calibrated model had 3.35 MGD of dry weather loadings based on the March 2017 flow metering. The spring typically has greater dry weather flows because of higher groundwater elevations, which is attributed to lower evapotranspiration rates. The GWI component of the dry weather flows was decreased to create the base year model. Decreasing the GWI component was appropriate since the groundwater conditions observed during the monitoring period were higher than the average condition during the previous five years.

5.2 SIZING AND TRIGGER CRITERIA

Sizing and trigger criteria are used to determine whether an improvement is required. The criteria are separated into two groups – trigger and sizing criteria. Trigger criteria is a set of conditions that when exceeded will initiate an improvement. The sizing criteria are the conditions that the improvements will be designed to convey without exceeding. The criteria are separated to prioritize the capacity investment for the City. A design storm analysis was performed to evaluate the financial risks of higher and lower probability rainfall events and to select the appropriate criteria that efficiently mitigates risk.

5.2.1 Design Storms

For each of the planning years, the hydraulic system was analyzed under peak flows resulting from the selected design storm. These events will test the system's ability to convey a high flow event, and the rainfall frequency indicates how frequently this peak flow will occur. A 2-year storm was used to trigger an improvement and the 10-year storm was used to assess any risk in the system of overflows after improvements were completed. The depths for both these events were developed from *NOAA's National Weather Service Hydrometeorological Design Studies Center Precipitation Frequency Data Server* (https://hdsc.nws.noaa.gov/hdsc/pfds/index.html) for the City and are shown in Table 5-2. Each rainfall event was given a distribution consistent with a National Resources Conservation Service (NRCS) Type II distribution (formally known as an SCS Type II distribution).

Table 5-2 Design Storm Depths

STORM RETURN PERIOD	DEPTH (in.)	APPLICATION
2-year storm	4.02	Risk assessment. Used to determine when a project is needed.
5-year storm	4.93	Risk assessment.
10-year storm	5.66	Risk assessment. Used to size improvements to risk of SSOs, to determine phasing, and verify the sizing of future projects.
25-year storm	6.67	Risk assessment.

5.2.2 Design Storm Selection

To evaluate the appropriate design storm the baseline hydraulic model was analyzed under four different design storms: 2-year, 5-year, 10-year, and 25-year, as well as the calibration events. The design storms were based on a synthetic rainfall event using an NRCS (formally known as the Soil Conservation Service) Type II rainfall distribution. The results of the storm event on the base year model were analyzed for capacity. Where a pipe was surcharged (the flow depth exceeds the pipe diameter, d/D>1) and lacked capacity (the flow in the pipe is larger than pipe capacity, q/Q>1), a pipe replacement was sized using manning's equation and a preliminary cost was estimated. The annual exceedance of each design storm was plotted against the preliminary cost of improvement.

The resulting curve shows the economic risk based on each return period. The return period is the inverse of the expected number of occurrences in 1 year; therefore, as the return period increases, the probability of the event decreases.

To select a design storm, the knee of the curve, or the point on the curve where the cost begins to increase dramatically relative to the level of risk, was identified. Figure 5-1 shows the cost of improvements in the base year model by design storm event. The May 21, 2017, calibration event is also shown for reference. It should be noted that the cost shown on Figure 5-1 represents the relative cost of improvements based on the different design storms, and does not represent the cost to complete base year system improvements based on the capacity assessment. The design storm analysis was preliminary evaluation using Manning's equation, which registered every pipe that had low slope, flat slope and adverse slope to be replaced regardless of the momentum in the pipe. The knee of the curve is between the 2-year and 5-year SCS storm. Beyond that point, the cost of reducing the risk further is cost prohibitive relative to the decrease in risk. In North Carolina, Black & Veatch has observed that it is typical for utilities to use between 1-year and 10-year design storm for the sizing and trigger criteria. In addition, most utilities use a staggered approach to address high priority needs first. Often, this resembles a program to address capacity failures in higher frequency events first, but to size replacement pipes for a large event. This approach allows for implementation of a higher level of service over time. The City selected the 2-year storm as the trigger criteria and the 10-year storm as the sizing criteria based on the knee of curve analysis and the benchmarking of other utilities in North Carolina.

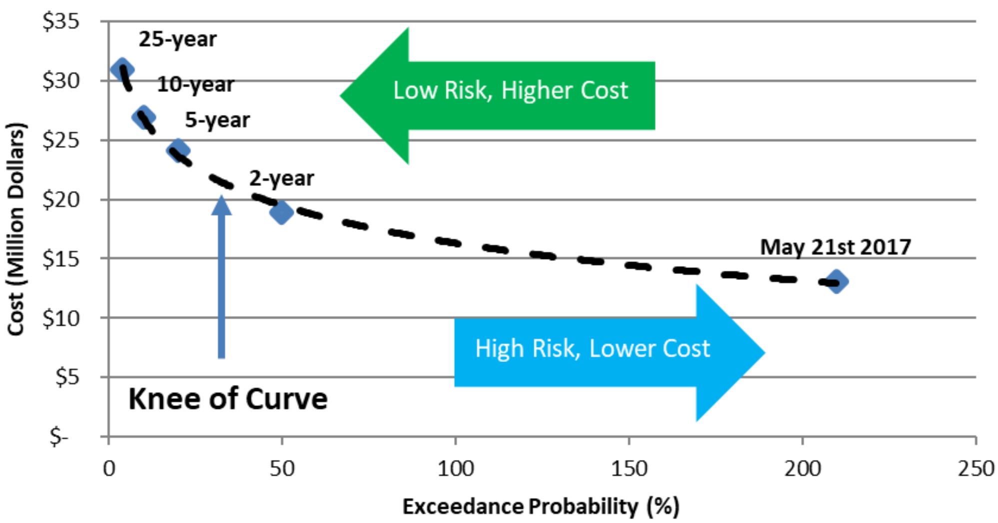


Figure 5-1 Design Storm Cost Curve

5.3 ASSESSMENT AND IMPROVEMENT METHODOLOGY

The existing collection system was modeled with the projected 2040 dry weather flows under the 2-year and 10-year design storm to determine the locations where capacity constraints occurred. In the future, I/I in the system is expected to remain constant. This assumes that the City's maintenance program is able to keep up with the rate of deterioration in the collection system.

The improvements considered consisted of at least one of the following:

- Bolted Manhole Lids (temporary only).

- Relief Sewers.
- Replacement Sewers.
- Pump Station Pump Replacement.
- Force Main Replacement/Parallel.
- New Pump Stations/Force Main.
- Equalization Storage.
- Flow Redirections.

Capacity improvements for the pump stations were implemented if the firm capacity was exceeded. The improvements consisted of an actual pump replacement and/or force main replacement depending on the circumstances at each station. If excessive velocities were observed in the force main (greater than 8 ft/s), a force main replacement was considered. If the force main velocities were not excessive, then the pump station needed improvements to install greater capacity pumps.

After all necessary improvements were determined and sized, the phasing schedule for the improvement was established. The loading conditions for each planning year were used starting with the base year (2017). Each planning year was evaluated with the 2-year and 10-year storms. Critical improvements were only recommended for the base planning year if they were necessary to relieve surcharge within 2 feet of the manhole rim or model predicted overflows during the 2-year storm. The 2-year and 10-year storm responses were also assessed for the future planning years. Projects that alleviated SSOs during a 2-year storm became the highest priority. This process was implemented for all the planning years. The improvements recommended for the previous planning year were modeled and tested under the peak flow conditions for the next planning year. Until the ultimate planning year (2040) was reached. All improvements were sized to convey the 2040 peak flows during a 10-year storm without surcharging within 2 feet of any manhole rim. In some cases, surcharging in the gravity sewer is still observed. All improvements assumed adverse slopes were replaced with average slopes in the pipe segments.

Additionally, there are several known Department of Transportation (DOT) projects that will occur in the next five years. Any project that intersected with the DOT projects was prioritized earlier to save costs.

5.4 ANALYSIS

The base year and future year models resulted in the need for several improvements in the Mud Creek basin. The assessment of the 2040 planning year flows under a 2-year storm event indicated capacity constraints along the main Mud Creek interceptor, Bat Fork outfall, Wash Creek outfall, Brittain Creek outfall, Shepherds Creek outfall, Clear Creek outfall, Devils Fork outfall, and King Creek outfall. The existing system performance under the 2040 planning year loads with a 2-year storm is shown on Figure 5-2. The map shows many SSOs near the capacity constraints. Once improvements are implemented in these areas, the SSOs are mitigated and the system has sufficient capacity. A map of all the capacity projects is shown on Figure 5-3.

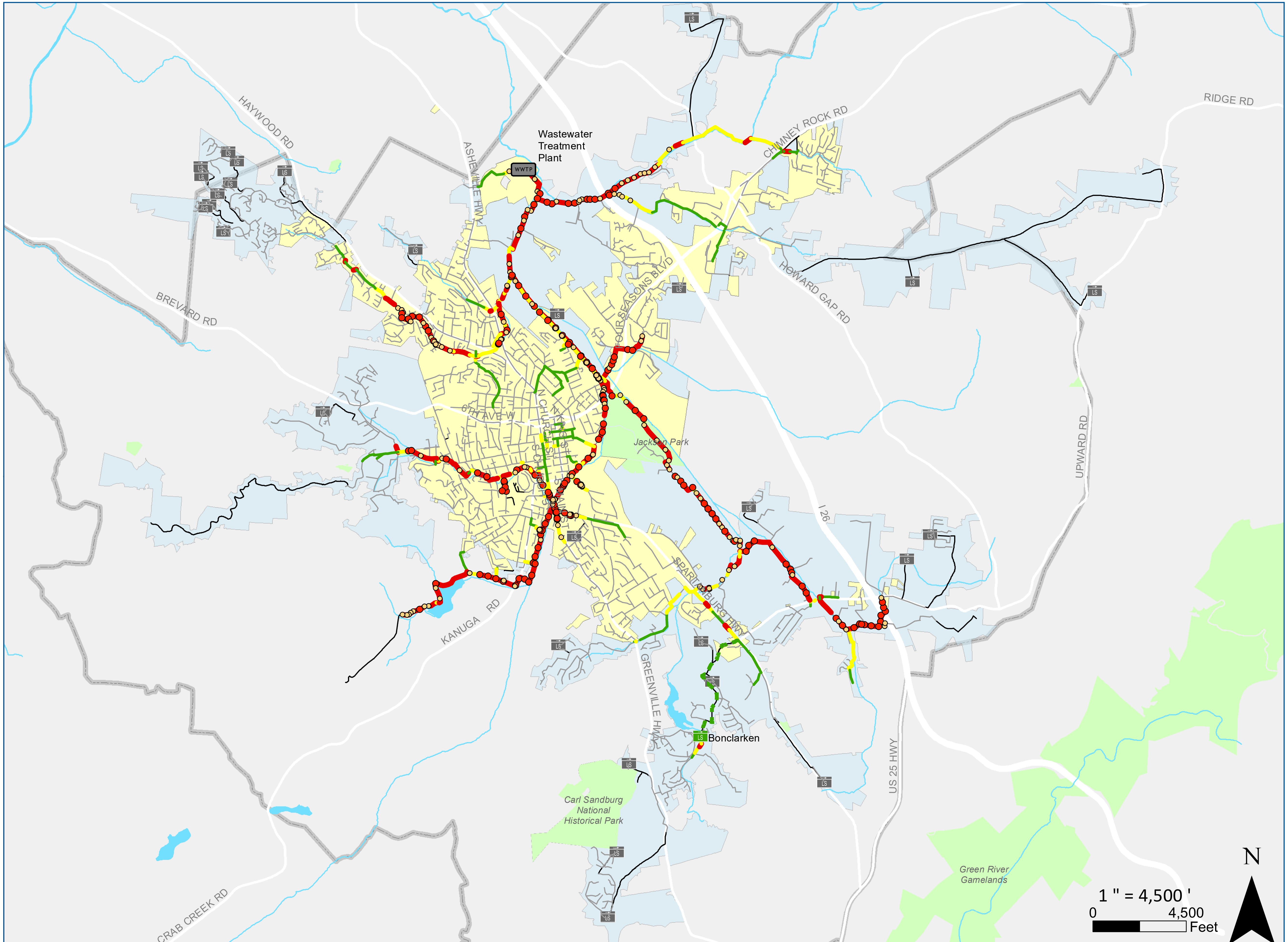
The assessment figure categorizes the predicted manhole surcharge at different levels. Manholes that are not surcharged or that have more than 8 feet of freeboard between the rim and the peak water surface elevation are not displayed for clarity. The 8 foot freeboard criterion was established to estimate whether basement flooding for nearby customers were possible. Manholes that have less than 8 feet of freeboard but more than 2 feet are noted as green nodes. Manholes that have less than 2 feet of freeboard but are not flooding are noted as yellow nodes. Finally, model predicted overflow/flooding manholes are shown as red nodes.

Pump stations are evaluated based on the firm capacity and total station capacity. The Bonclarken pump station was the only pump station evaluated in the skeletonized model (shown in green) and it did not exceed its firm capacity.

The gravity sewers are also categorized if they are surcharged (yellow) or are not surcharged (green). The sewers where capacity restrictions exist (i.e., where the peak modeled flow is greater than the pipe flowing full capacity) are coded as red pipes in the assessment figure. Whereas the manholes show the results of the system assessment, the color coding of the pipes provides insight into the reason for the surcharging. The red colored gravity sewers are often the hydraulic bottleneck that causes the upstream surcharging conditions.

The peak force main velocity is also shown in the assessment figure. This information also provides an indication for the reason why the upstream pump station's capacity is exceeded. If the capacity were exceeded with excessive force main velocities, the indication is that a force main improvement is likely required. If the force main velocity were in acceptable ranges with simultaneous wetwell flooding, the improvement would focus on increasing the capacity of the pumps. It should be noted that the force main sizing would have to be reviewed if there were a pump station capacity increase to verify whether the velocity is acceptable. The only force main evaluated in the skeleton model was the Bonclarken force main, and the velocity in the force main was within the criteria.

City of Hendersonville
SSAIA
Mud Creek Basin
Existing System Performance
2040 Flows
Figure 5-2



LEGEND

Modeled Forcemain
- Velocity: > 7 ft/s
- Velocity: 2-7 ft/s
- Velocity: < 2 ft/s

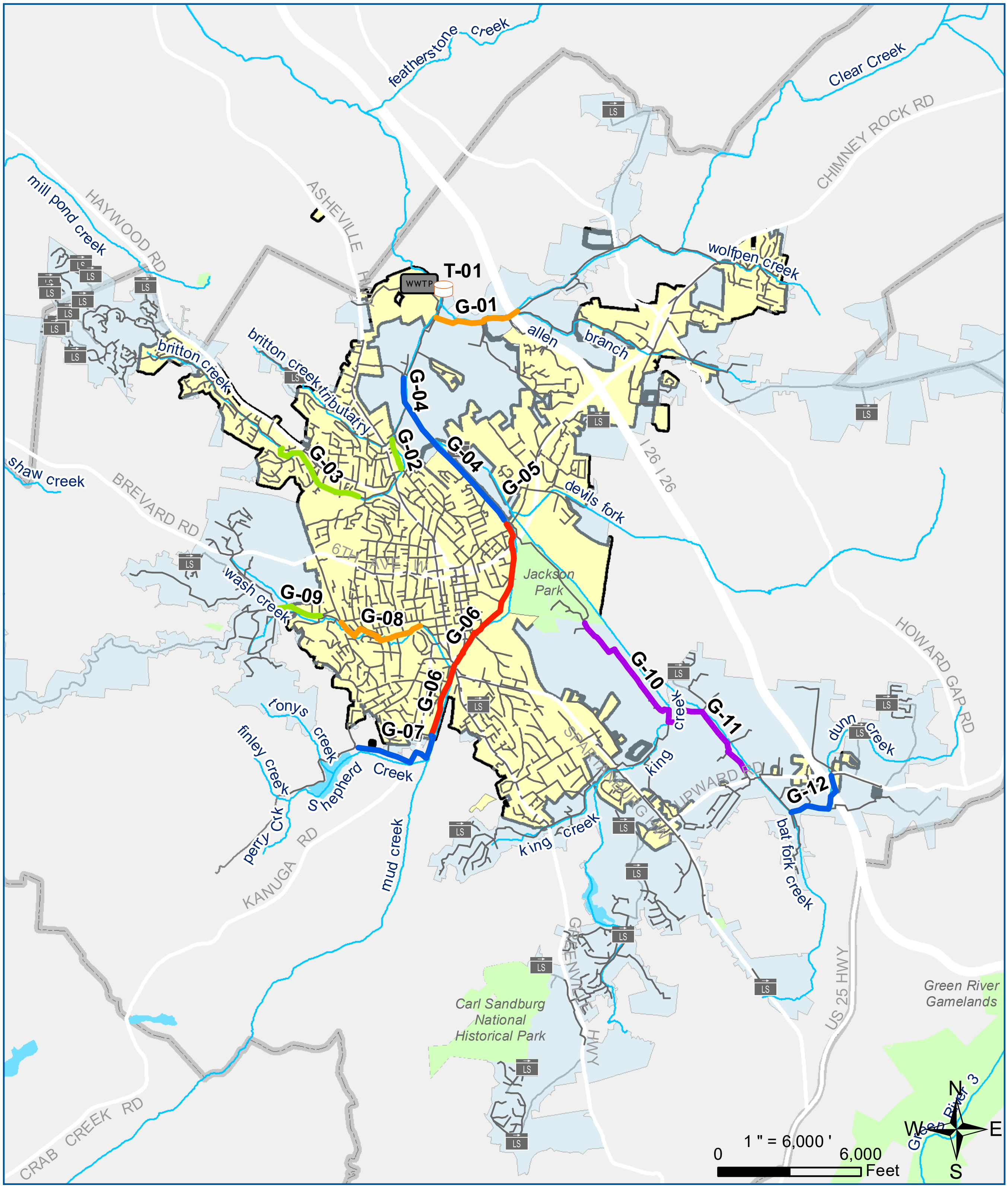
Modeled Manholes
- Overflowing
- Model Simulated Surcharge (Within 2 Feet of the Rim)

Modeled Pipes
- Capacity Limited Sewer
- Surcharged Sewer (Due to downstream restriction)
- Unsurcharged Gravity Sewer
- Existing Service Boundary
- 2040 Service Area
- City Limits
- COH Collection System

Hendersonville Water Sewer

BLACK & VEATCH

Template for Figures 5-1 - 5-6 March 18, 2019



City of Hendersonville
SSAIA

Mud Creek Basin
Capacity Projects
Figure 5-3

- Critical Project (2018)
- 2021 - 2027
- 2028 - 2032
- 2033 - 2037
- 2038 - 2042
- EQ Tank
- COH Collection System
- City Limits
- Existing Service Boundary
- 2040 Service Area

Hendersonville Water Sewer

BLACK & VEATCH

The capacity improvements were designed to accommodate the 10-year storm event in planning year 2040. The required pipe sizing for each improvement is shown in Table 5-3.

Table 5-3 Mud Creek Basin Capacity Projects

	PROJECT	PROJECT TYPE	EXISTING DIAMETER (IN.)	MINIMUM IMPROVEMENT DIAMETER (IN.) [1]	LENGTH (FT)
2018 (Critical)	G-06	Replacement	12	21	3,150
			24	36	6,820
	T-01	WWTP EQ	Size to be determined		
2025	G-07	Replacement	12	18	4,320
2040	G-01	Replacement	18	30	3,630
	G-02	Replacement	18	24	1,700
	G-03	Replacement	12	15	4,480
	G-04	Replacement	27	36	6,310
			42	54	1,180
	G-05	Replacement	15	21	3,070
	G-8	Replacement	12	21	4,150
	G-9	Replacement	12	15	1,950
	G-10	Replacement	18	30	5,970
	G-11	Replacement	12	24	4,810
	G-12	Replacement	12	18	1,640
			12	15	1,530

[1]Minimum improvements diameter was based off the capacity for the pipe and existing. Design engineer should select final pipe size based on installed slope and pipe material availability. Final pipe size should be able to convey the modeled peak flow at the design slope.

The future project specific timing, benefits, and detailed information for each planning year are outlined in the following sections.

5.5 SYSTEM FLOWS

Peaking factors are a commonly applied design parameter for wastewater collection and treatment system design. The factors relate the peak flow that the system must convey. The hydraulic model does not directly use peaking factors; however, a peaking factor from the model can be calculated for comparison purposes. Flow enters the modeled collection system at assigned load points. These load points can either flow from dry weather sources or wet weather I/I sources. The WWTP inflow hydrographs were analyzed to determine a peaking factor for the peak flow conditions as summarized below in Table 5-4. The peak flows during the base year for the 2-year and 10-year storms were 17.4 MGD and 22.8 MGD, respectively. These peak flows resulted in the treatment plant peaking factors of 5.7 and 7.4, respectively, when compared to an average dry weather flow of 3.07 MGD. NCDEQ often requires a peaking factor of 3.0 for system design purposes. Since the peaking factors from the design storms are greater, designing system improvements will be more conservative than the use of a 3.0 peaking factor.

Table 5-4 Base Year Design Storm WWTP Flow and Peaking Factors

DESIGN STORM	PEAK WWTP FLOW (MGD)	PEAKING FACTOR
2	17.4	5.7
10	22.8	7.4

Figure 5-5 shows the peak flows predicted for the 2-year storm in the collection system for the following conditions:

- No improvements under 2017 base year loads
- Critical improvements under 2025 loads
- All improvements under the 2040 loads

Peak flows in the downstream portion of the system (as measured just upstream of the WWTP) are approximately 17.4 MGD in the base year existing system model. The peak flows, shown in the tables and on the schematic, are the instantaneous peaks at that location. Due to differing time of travel and attenuation in the system, the upstream basins do not add up exactly to the peak flows observed at downstream locations. Additionally, in the base year model there was an actual reduction in the peak flows due to predicted SSOs in a 2 year event. The increase in the dry and wet weather flows, the reduction in flow lost in SSOs, and the reduction in attenuation in the pipes increase the peak wet weather flow to 23.1 MGD and 30.2 MGD by 2025 and 2040, respectively. Figure 5-4 shows a basin map for reference.

Table 5-5 2-Year Peak Flows (MGD) in the Collection System

LOCATION	2017 EXISTING	2025 WITH CRITICAL PROJECTS	2040 ALL PROJECTS
WWTP Influent Flow	17.4	22.5	28.2

Table 5-5 shows the 2-year peak flows going to the WWTP per basin. The permitted WWTP capacity is 4.8 MGD. The WWTP can expect peak flows in excess of the hydraulic capacity of the plant during the base year. This is mitigated by project T-01 equalization storage for the WWTP; however, the WWTP will need to be upgraded between 2025 and 2030 because of the increase in dry weather flows as described in Section 4.

Table 5-6 Basin 2-Year Peak Flows (MGD)

BASIN	PLANNING YEAR		
	2017	2025	2040
Meter Basin 1	0.8	2.9	5.3
Meter Basin 2	16.7	20.9	25.8
Meter Basin 3	5.9	6.1	6.7
Meter Basin 4	2.4	2.5	2.8
Meter Basin 5	9.0	9.7	11.4
Meter Basin 6	4.6	4.9	5.7
Meter Basin 7	2.5	3.8	6.2
Meter Basin 8	0.5	0.8	1.3
WWTP	17.4	22.5	28.2

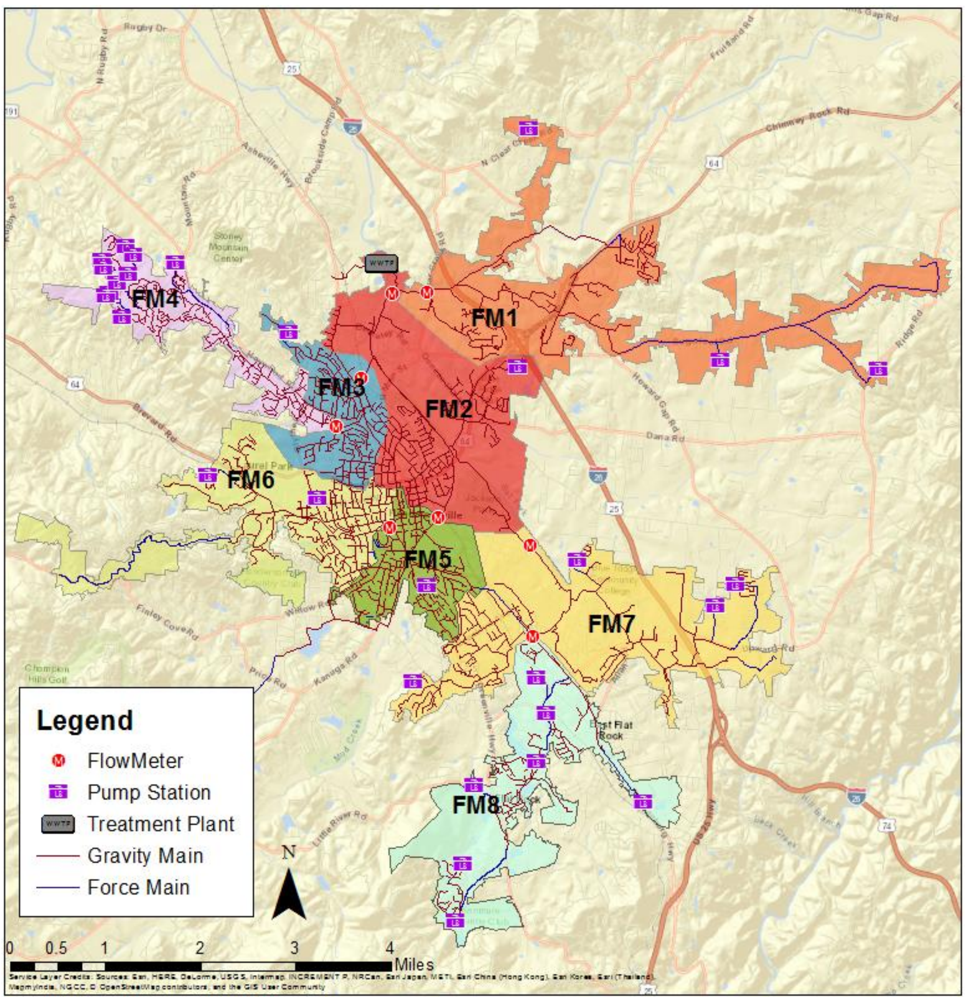
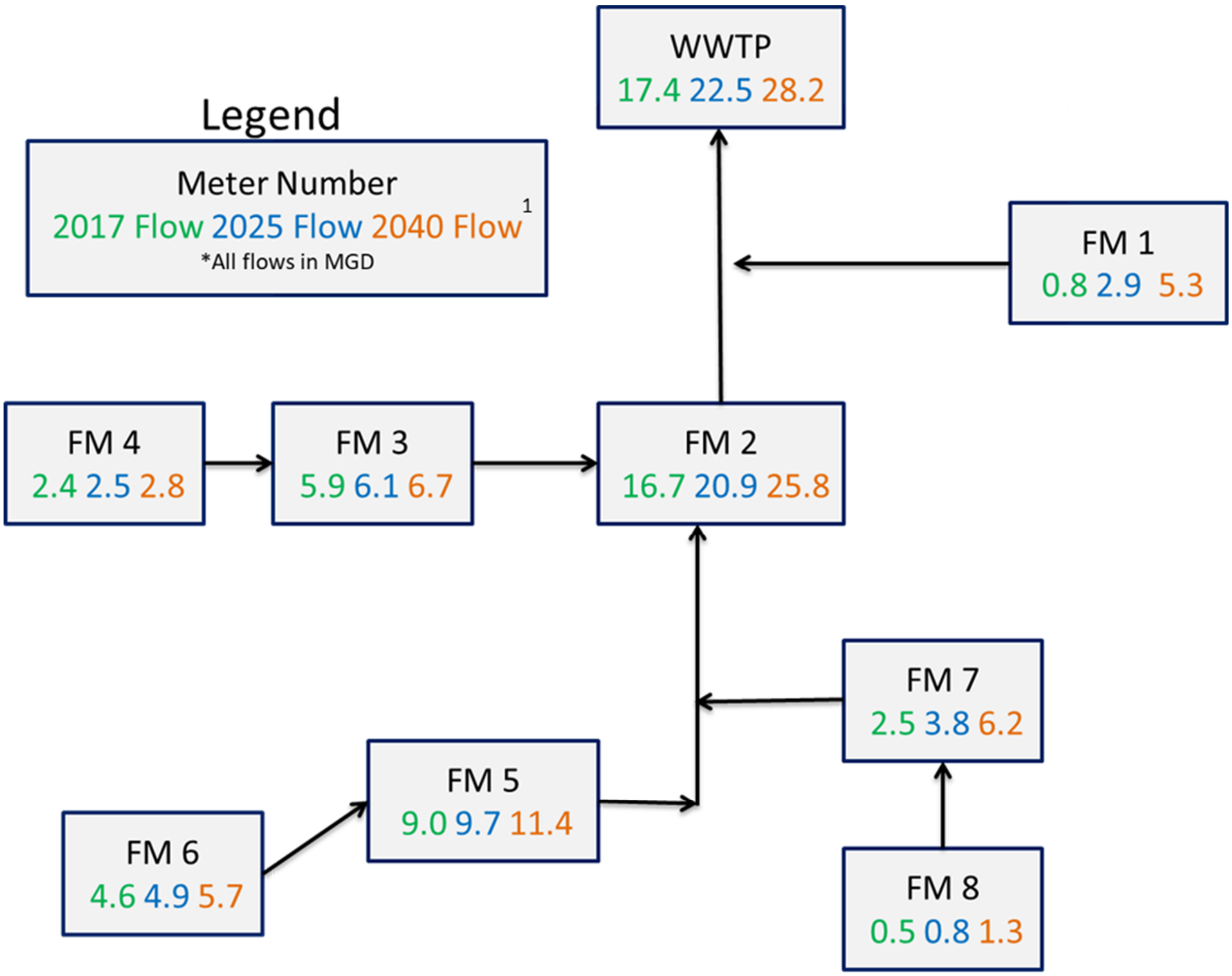


Figure 5-4 COH System Basin Map



1. Peak instantaneous flow. Due to time of travel, surcharge and SSO conditions, the peak flow should not be equal the sum of the upstream basins

Figure 5-5 System Peak Wet Weather Flows (2-Year Storm)

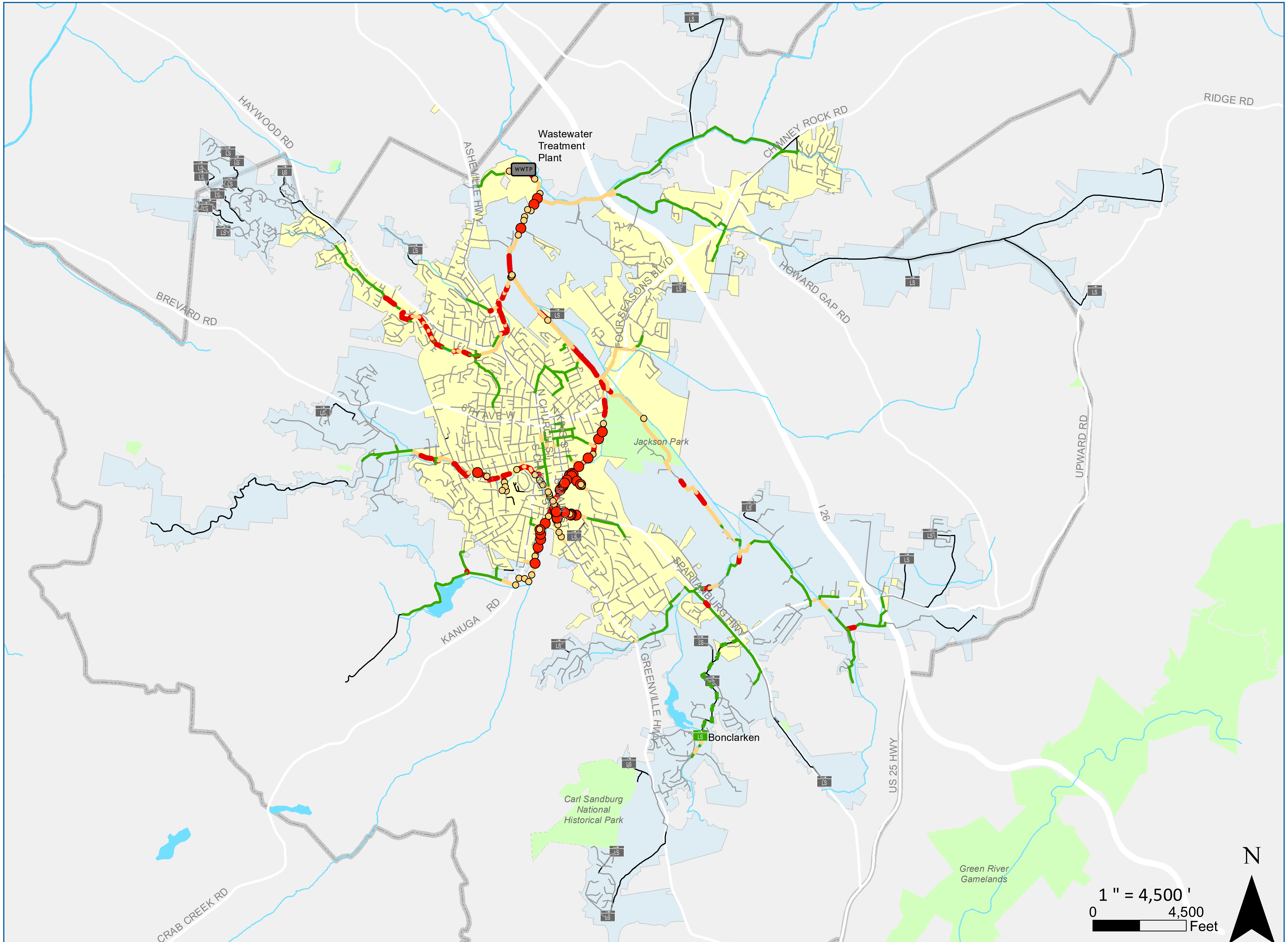
5.6 CRITICAL IMPROVEMENTS (YEAR 2019 IMPROVEMENTS)

As referenced above, improvements were identified for the base year if they were necessary to relieve model predicted overflows during the 2-year storm, and as such are identified as critical. Under the 2-year storm, one major pipeline project was designated as critical. Additionally, improvements at the WWTP are necessary to reduce backup in the collection system. An equalization (EQ) tank and an evaluation of the WWTP hydraulic capacity and process capacity is recommended. The 2-year storm response in the base year is shown on Figure 5-6. The model predicts several overflow locations along the main Mud Creek interceptor caused by backup from the WWTP. There are also several predicted SSOs along Mud Creek extending toward downtown that are driven by insufficient capacity in the gravity outfall.

5.6.1 Summary of Critical Base Year (2019) Improvements

G-06: Replacement Sewer along Mud Creek near Railroad. This improvement will relieve surcharging and potential overflows on the Mud Creek outfall. The model predicts SSOs during a 2-year storm starting in the base year. The replacement will include approximately 9,960 feet of gravity sewer. The sewer will follow the alignment of the existing sewer along Mud Creek; however, the alignment should be evaluated to avoid the railroad crossing near South Kind Street, if possible. The project will include 3,150 feet of 21 inch and 6,820 feet of 36 inch. The project has several stream crossings, three potential rail road crossings, and six road crossings: White Street, S. Main Street, S. Grove Street, 4th Avenue, and Four Seasons Boulevard. The project should be evaluated to reroute the alignment through the existing 36 inch jack and bore previously performed for the Jackson Park sewer line. This project is a critical project and should be started immediately. This project is planned for 2021 to coordinate with the NCDOT White and Main street project.

T-01: Equalization Basins and WWTP Capacity Study. During wet weather events, only one of the large pumps (6.5 MGD) in the Influent Pump Station is kept running to maintain the treatment process performance at the Mud Creek WWTP. When limiting the plant to 6.5 MGD, the City would need a 5 MG EQ tank to store a 2-year storm event with existing system loadings without creating surcharge back into the collection system. Alternatives to only adding equalization could be improvements at the primary clarifiers, installing variable frequency drives at the influent pump station to send more flow through the plant, or some combination to increase treatment and EQ capacity. A preliminary evaluation of the plant hydraulics and process is currently being conducted and will be added as an addendum.



City of Hendersonville
SSAIA

Mud Creek Basin
No Improvements
Base Year Flows
Figure 5-6

LEGEND

Modeled Forcemain

- Velocity: > 7 ft/s
- Velocity: 2-7 ft/s
- Velocity: < 2 ft/s

Modeled Manholes

- Overflowing
- Model Simulated Surcharge (Within 2 Feet of the Rim)

Modeled Pipes

- Capacity Limited Sewer
- Surcharged Sewer (Due to downstream restriction)
- Unsurcharged Gravity Sewer
- Existing Service Boundary
- 2040 Service Area
- City Limits
- COH Collection System

Hendersonville Water Sewer

BLACK & VEATCH

Template for Figures 5-1 - 5-6 February 28, 2019

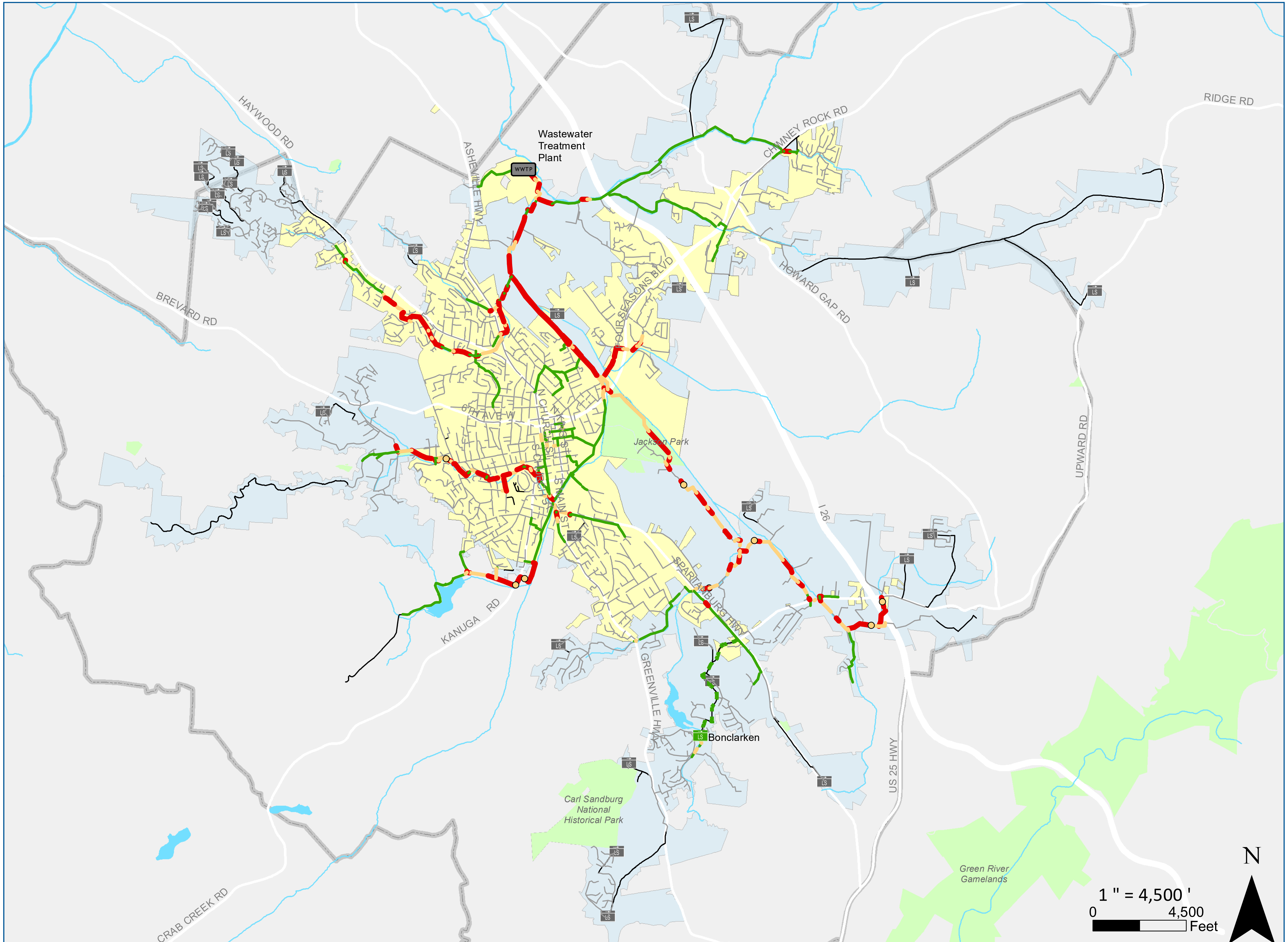
5.7 2025 PLANNING YEAR (YEAR 2022-2025 IMPROVEMENTS)

Figure 5-7 shows the impacts of a 2-year storm on the 2025 flows, with the critical improvements in place. The 2025 assessment figure shows that the next segment upstream of project G-06 will also need to be replaced. The risk of an SSO is also higher since the sewer is not buried very deep in this section. The project is described below.

5.7.1 Summary of Year 2026 Improvements

G-01: Clear Creek Sewer Replacement near Future Greenway. This improvement will relieve surcharging and potential overflows along Clear Creek. This improvement is driven by future flows. The model predicts SSOs during a 2-year storm starting in 2040. Approximately 6,730 feet of 30-inch gravity sewer will replace the existing 18-inch and 24-inch that parallels Clear Creek. The sewer will follow the existing alignment along Clear Creek and will cross Clear Creek once and Allen Branch once. This project will require 3 road crossings: Clear Creek Road, I-26, Nix Rd. This project should be coordinated with the NCDOT I-26 construction set to occur in 2026.

G-08: Wash Creek Replacement Sewer. This improvement will relieve surcharging and potential overflows along Wash Creek. This improvement is driven by wet weather flows. The model predicts SSOs during a 2-year storm starting in 2040 and during a 10-year storm as early as the base year. Approximately 4,150 feet of 21 inch gravity sewer will replace the existing Wash Creek outfall. The sewer will follow the exiting alignment along Wash Creek and includes three road crossings: Kanuga Road, W. Barwell Street, and S. Washington Street.



City of Hendersonville
SSAIA

Mud Creek Basin
Critical Improvements
2025 Year Flows
Figure 5-7

LEGEND

Modeled Forcemain
- Velocity: > 7 ft/s
- Velocity: 2-7 ft/s
- Velocity: < 2 ft/s

Modeled Manholes
- Overflowing
- Model Simulated Surcharge (Within 2 Feet of the Rim)

Modeled Pipes
- Capacity Limited Sewer
- Surcharged Sewer (Due to downstream restriction)
- Unsurcharged Gravity Sewer
- Existing Service Boundary
- 2040 Service Area
- City Limits
- COH Collection System

Hendersonville Water Sewer

BLACK & VEATCH

5.8 2040 PLANNING YEAR (2025 - 2040 IMPROVEMENTS)

The 2-year storm with 2040 flows will cause more surcharge in the existing Mud Creek basin. The deficiencies in the 2040 model under a 2-year storm are shown on Figure 5-8. The model results with the improvements are shown in Figure 5-9. The projects listed are driven by the projected growth in the system and the accompanying wet weather I/I associated with system expansion. Development in the system should be tracked to determine if any of the recommended improvements should be completed earlier or whether the improvement can be postponed until development moves forward.

The collection system performance with all of the improvements was analyzed under the projected 2040 loadings with the 2-year storm. There are no predicted overflows in the modeled system.

5.8.1 Summary of Year 2040 Improvements

G-02: Brittain Creek Sewer Replacement near Patton Park. This improvement will relieve surcharging and potential overflows along Brittain Creek. This improvement is driven by wet weather flows. The model predicts SSOs during a 2-year storm starting in 2040 and during a 10-year storm as early as the base year. Approximately 1,700 feet of 24 inch gravity sewer will replace the existing Brittain Creek outfall. The new sewer has one road crossing: E Clairmont Drive. One portion of the G-02 project will be completed in 2019.

G-03: Brittain Creek Sewer Replacement near Haywood Road. This improvement will relieve surcharging and potential overflows along Brittain Creek. This improvement is driven by wet weather flows. The model predicts SSOs during a 2-year storm starting in 2040 and during a 10-year storm as early as the base year. Approximately 4,480 feet of gravity sewer will replace the existing Brittain Creek outfall. The gravity sewer will follow the existing alignment along Brittain Creek. This project will include 4,480 feet of 15 inch sewer and five road crossings: Maplewood Court, Blythe Street, Hampton Court, Haywood Townes Drive, and White Oaks Drive. This project would be needed before 2040. The perpendicular crossing should be coordinated with the NCDOT Blythe Street project expected to occur in 2023.

G-04: Mud Creek Parallel Replacement. This improvement will relieve surcharging and potential overflows along Mud Creek. This improvement is driven by future development and wet weather flows. The model predicts SSOs during a 2-year storm starting in 2040 and during a 10-year storm as early as 2025. Approximately 7,490 feet of gravity sewer will replace the existing 27 inch Mud Creek outfall. The sewer will follow the alignment of the existing sewer along Mud Creek, but should be placed deeper to accommodate the lower elevations of the upstream sewers. The project will include 6,310 feet of 36 inch and 1,180 feet of 54 inch gravity sewer.

G-05: Devils Fork Sewer Replacement near MLK Jr. Boulevard. This improvement will relieve surcharging and potential overflows along the Devils Fork outfall. This improvement is driven by future development. The model predicts SSOs during a 2-year storm starting in 2040. Approximately 3,100 feet of 21 inch gravity sewer will replace the existing Devils Fork outfall. The alignment will follow the existing sewer along 7th Avenue. This project has one stream crossing and requires three road crossings: Dana Road, Tracy Grove Road, and 7th Avenue.

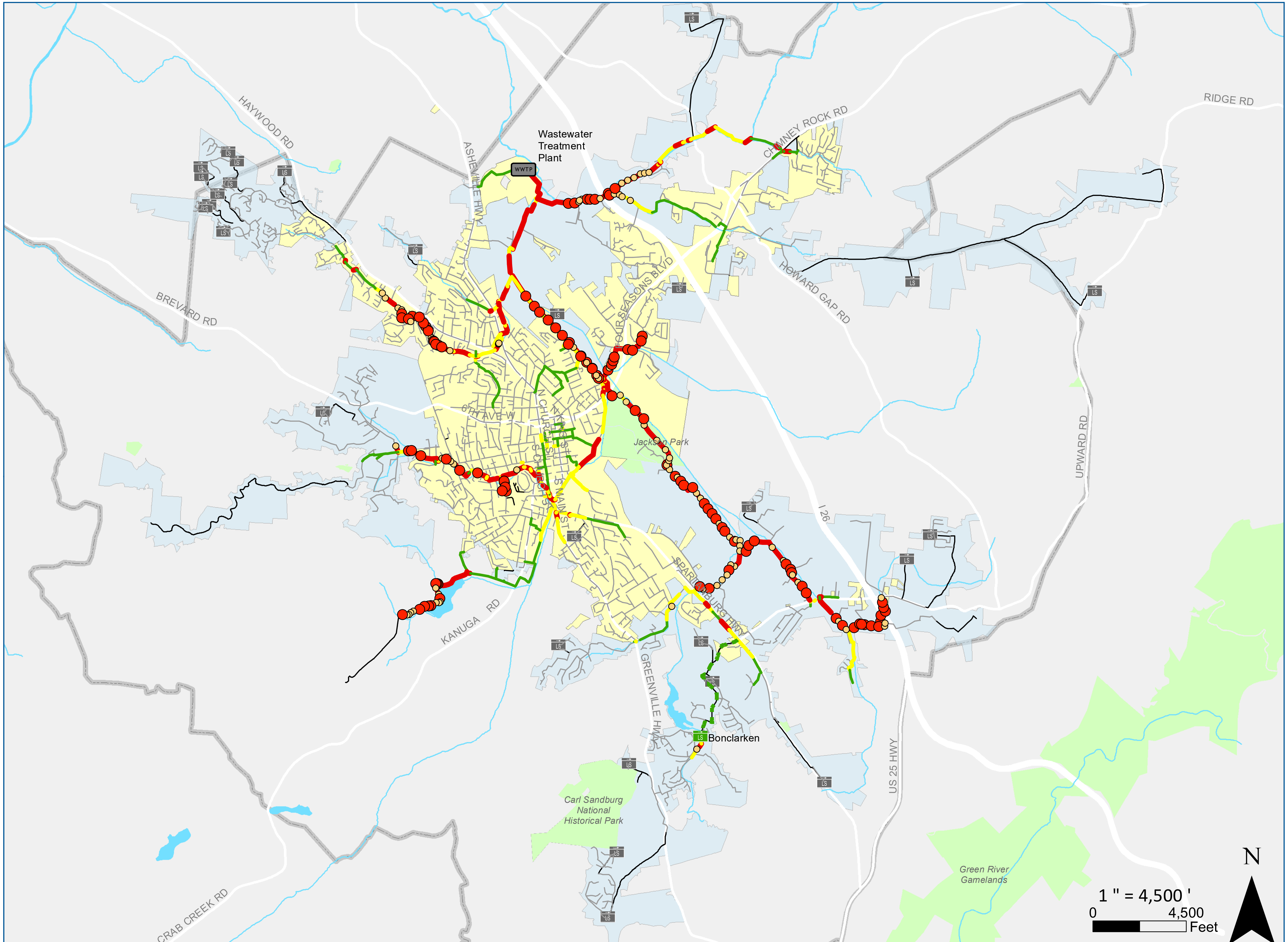
G-07: Shephard Creek Replacement Sewer near Kanuga Road. This improvement will relieve surcharging and potential overflows on the Shepard Creek outfall upstream of project G-06. The model predicts SSOs during a 2-year storm starting in 2025. Approximately 4,320 feet of gravity sewer will need to replace the existing Shephard Creek outfall. The sewer will follow the alignment of the existing sewer along Shepherd Creek. The project will include 4,320 feet of 18 inch gravity sewer. This project will include two road crossings: Erkwood Drive, and Kanuga Road. This project is planned for 2022 to coordinate with the NCDOT Kanuga Road project.

G-9: Wash Creek Replacement Sewer near Wash Creek Drive. This improvement will relieve surcharging and potential overflows along Wash Creek. This improvement is driven by future development and wet weather flows. The model predicts SSOs during a 2-year storm starting in 2040 and during a 10-year storm as early as 2025. Approximately 1,950 feet of 15 inch gravity sewer will replace the existing Wash Creek outfall. The sewer will follow the existing alignment along Wash Creek. The sewer crosses Wash Creek once and includes one road crossing on Wash Creek Drive.

G-10: King Creek Replacement Sewer near Airport. This improvement will relieve surcharging and potential overflows along the King Creek outfall. This improvement is driven by future development and wet weather flows. The model predicts SSOs during a 2-year storm starting in 2040 and during a 10-year storm as early as 2025. Approximately 5,970 feet of gravity sewer will replace the existing Bat Fork outfall. The sewer will follow the existing alignment along Bat Fork; the new sewer will cross Kings Creek one time. The project will include 5,970 feet of 30 inch gravity sewer and has three road crossings: Airport Road, Grandeur Lane, and New Hope Road.

G-11: Bat Fork Replacement Sewer near Blue Ridge Community College. This improvement will relieve surcharging and potential overflows along the Bat Fork outfall. This improvement is driven by future development. The model predicts SSOs during a 2-year storm starting in 2040. Approximately 4,810 feet of 24 inch gravity sewer will replace the existing Bat Fork outfall.

G-12: Dunn Creek Replacement Sewer near I-26. This improvement will relieve surcharging and potential overflows along Dunn Creek outfall. This improvement is driven by future development and wet weather flows. The model predicts SSOs during a 2-year storm starting in 2040 and during a 10-year storm as early as 2025. Approximately 3,170 feet of gravity sewer will replace the existing Dunn Creek outfall. The project will include 1,530 feet of 15 inch and 1,640 feet of 18 inch gravity sewer. The new sewer will cross Dunn Creek once and has two road crossings: I-26 and Commercial Boulevard.



City of Hendersonville
SSAIA

Mud Creek Basin
2025 Improvements
2040 Year Flows
Figure 5-8

LEGEND

Modeled Forcemain

- Velocity: > 7 ft/s
- Velocity: 2-7 ft/s
- Velocity: < 2 ft/s

Modeled Manholes

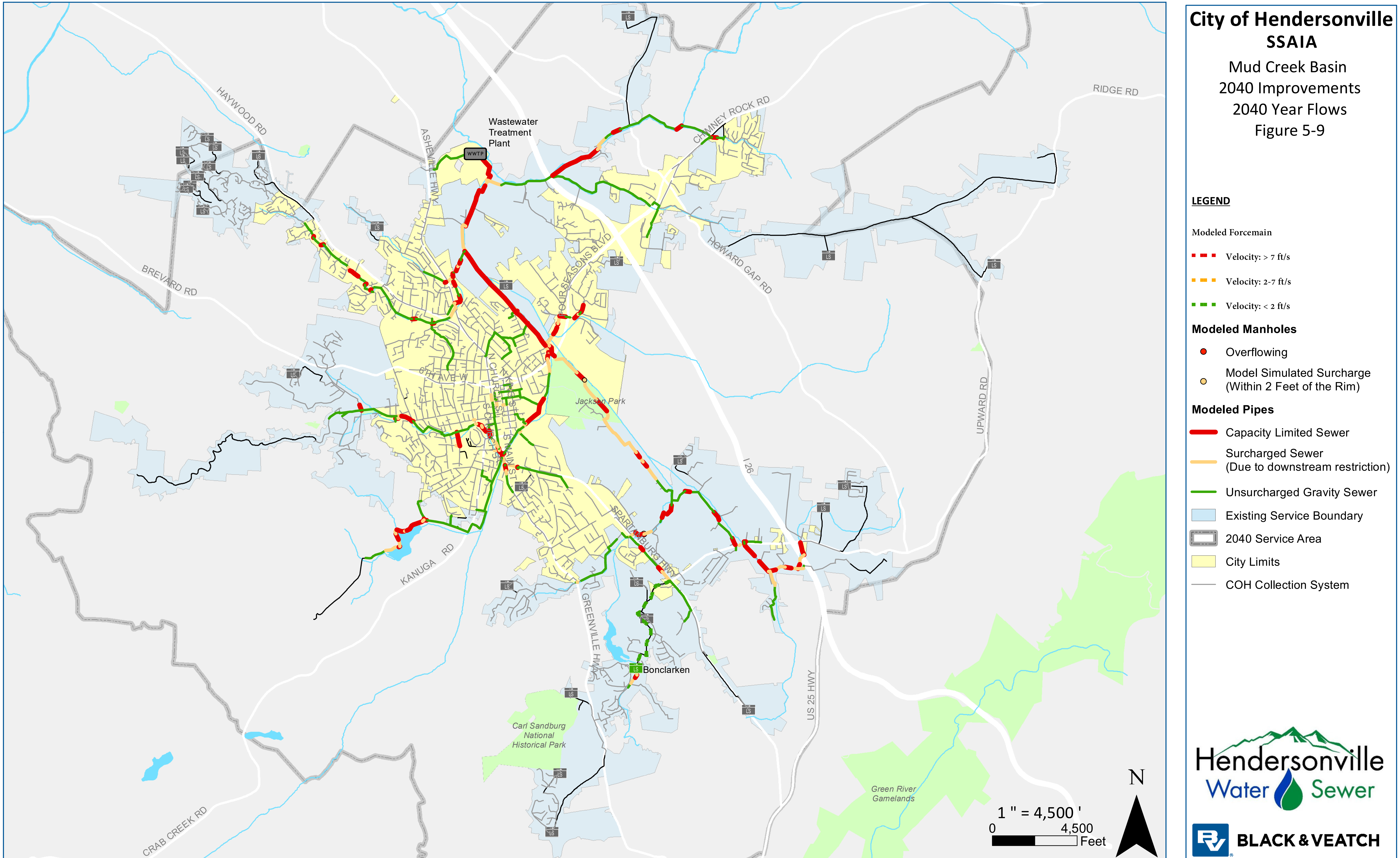
- Overflowing
- Model Simulated Surcharge (Within 2 Feet of the Rim)

Modeled Pipes

- Capacity Limited Sewer
- Surcharged Sewer (Due to downstream restriction)
- Unsurcharged Gravity Sewer
- Existing Service Boundary
- 2040 Service Area
- City Limits
- COH Collection System

Hendersonville Water Sewer

BLACK & VEATCH



Template for Figures 5-1 - 5-6 March 18, 2019

5.9 PRELIMINARY OPINIONS OF PROBABLE COST

Planning level's unit costs were developed by analyzing recent bids for construction projects, as well as Black & Veatch experience in North Carolina. The unit costs vary based on pipe diameter and material. Additional costs items were included for manhole construction, clearing and grubbing, seeding, road and railroad crossings, and stream crossings. Table 5-7 defines the installation costs for gravity sewer. Other cost assumptions are shown in Table 5-8.

Table 5-7 Unit Costs for Construction of Gravity Sewer ($/ft)

DIAMETER	COST ($/LF)	MANHOLE COST ($/EA)
8/10	$134	$4,000
12	$144	$4,000
15	$174	$6,500
16	$184	$6,500
18	$209	$6,500
21	$249	$6,500
24	$289	$6,500
27	$324	$6,500
30	$358	$8,000
36	$443	$8,000
42	$548	$8,000
48	$668	$8,000
54	$760	$10,000

Table 5-8 Additional Cost Assumptions

COST ITEM	COST	OTHER ASSUMPTIONS
Pavement Removal and Replacement	$65/SYD	
Secondary Road Crossing	$800/LF	Jack and bore
Major Roadway Crossing	$1,500/LF	Micro tunneling
Railroad Crossing	$2,000/LF	Micro tunneling
Stream Crossing	$20,000/ea	
Erosion Control	$3/LF	Synthetic
Restoration	$2.50/LF	Assumes a 30 foot width

The project-specific costs are outlined in the construction costs table and are included in Appendix I. Final costs, including an additional 20 percent for planning and engineering design and 20 percent project contingency are included in the Final CIP table. The costs from the CIP table are summarized in Table 5-9. The cost opinion of plant improvements (T-01) will be provided following preliminary evaluation of the plant hydraulics and process.

Table 5-9 Cost of Hendersonville Gravity Projects

PROJECT ID	ENGINEERING (20%)	CONSTRUCTION COST	CONSTRUCTION CONTINGENCY (20%)	TOTAL PROJECT COST (2018 $)
G-01	$625,300	$2,537,300	$507,000	**$3,669,600**
G-02	$174,700	$693,600	$139,000	**$1,007,300**
G-03	$434,200	$1,724,000	$345,000	**$2,503,200**
G-04	$1,279,700	$5,193,000	$1,039,000	**$7,511,700**
G-05	$409,800	$1,650,900	$330,000	**$2,390,700**
G-06	$1,574,900	$6,375,600	$1,275,000	**$9,225,500**
G-07	$395,400	$1,567,900	$314,000	**$2,277,300**
G-08	$559,700	$2,255,600	$451,000	**$3,266,300**
G-09	$218,800	$873,400	$175,000	**$1,267,200**
G-10	$838,900	$3,384,800	$677,000	**$4,900,700**
G-11	$544,600	$2,178,200	$436,000	**$3,158,800**
G-12	$355,300	$1,421,100	$284,000	**$2,060,400**
Total Cost of Capacity Driven Projects				**$43,238,700**

5.10 EXTENSIONS AND PUMP STATION ABANDONMENT

In addition to the capacity projects, future gravity line extensions were required to collect the future flows in the 2040 service area. The alignments were projected to follow major existing creeks and natural drainage patterns to connect to the existing collection system. Six gravity extensions were identified and are shown with the abandoned pump stations on Figure 5-10. The gravity extension projects were sized using the 2040 2-year wet weather flows at the most downstream segment of the extension where it would connect to existing collection system. This overestimated the size of the line at the upstream end and should be re-evaluated as developments are built and the gravity line is designed. The gravity extensions' project timing were estimated using the project timing of the nearest downstream capacity project. The timing and costs are shown below in Table 5-10. As a result of the gravity extensions, two pump stations can be abandoned. These pump stations were the General Electric and the Lakewood RV pump stations. Additionally, there were several gravity sewers that could be installed that would enable pump stations to be abandoned. These are listed below in Table 5-10. To assess timing of future sewer line extensions, it is recommended that COH adopt an extension policy to establish guidelines for when an extension project is constructed. Extension Projects can also be implemented in phases to better serve development as it occurs.

EX-01: New Sewer Line Extension along Clear Creek. This extension will serve customers in the Edneyville area. This extension is driven by future development and the timing will be driven by agreements to serve customers in the Edneyville area. Approximately 14,500 feet of 24-inch gravity sewer will extend sanitary sewer service along Clear Creek. This project can be completed in phases to serve future customers. This extension was included in the CIP to start in 2024. This line would serve to decommission the private facilities at Camp Judaea, Justice Academy, and Edneyville Elementary. The timing could be pushed back based on demand from those facilities and developers in the area.

EX-02: New Sewer Line Extension along Devils Fork. This extension will serve customers in the east of downtown Hendersonville. This extension is driven by future development and the timing will be driven by agreements to serve customers east of Hendersonville and I-26. Approximately 14,500 feet of 18-inch gravity sewer will extend sanitary sewer service along Devils Fork. This project can be completed in phases to serve future customers. This extension was included in the CIP to start in 2029. The Start Year was set based on the completion of the downstream project, G-05. The timing could be pushed back based the timing of project G-05 and development in the area.

EX-03: New Sewer Line Extension along Mud Creek. This extension will serve customers on the south side of Hendersonville, near Flat Rock. This extension is driven by future development and the timing will be driven by agreements to serve customers in the area. Approximately 12,000 feet of 8-inch gravity sewer will extend sanitary sewer service along Mud Creek. This project can be completed in phases to serve future customers. This extension was included in the CIP to start in 2038. The Start Year was set based on the completion of the downstream project, G-07. The timing could be pushed back based the timing of project G-07 and development in the area.

EX-04: New Sewer Line Extension along Finley Creek. This extension will serve customers in the Laurel Park area. This extension is driven by future development and the timing will be driven by agreements to serve customers in the area. Approximately 10,000 feet of 10-inch gravity sewer will extend sanitary sewer service along Finley Creek. This project can be completed in phases to serve future customers. This extension was included in the CIP to start in 2036. The Start Year was set based on the completion of the downstream project, G-07. The timing could be pushed back based the timing of project G-07 and development in the area.

EX-05: New Sewer Line Extension along Dunn Creek. This extension will serve customers in the Upward Road area. This extension is driven by future development and the timing will be driven by agreements to serve future industrial, commercial and residential customers in the area. Approximately 8,000 feet of 18-inch gravity sewer will extend sanitary sewer service along Dunn Creek. This extension would allow for the abandonment of the Lakewood RV Pump Station. This project can be completed in phases to serve future customers. This extension was included in the CIP to start in 2033. The Start Year was set based on the completion of the downstream project, G-12. The timing could be pushed back based the timing of project G-12 and development in the area.

EX-06: New Sewer Line Extension along Bat Fork Creek. This extension will serve customers in the East Flat Rock area. This extension is driven by future development and the timing will be driven by agreements to serve future customers in the East Flat Rock area. Approximately 6,500 feet of 8-inch gravity sewer will extend sanitary sewer service along Bat Fork Creek. This extension would allow for the abandonment of the GE Pump Station. This project can be completed in phases to serve future customers. This extension was included in the CIP to start in 2037.

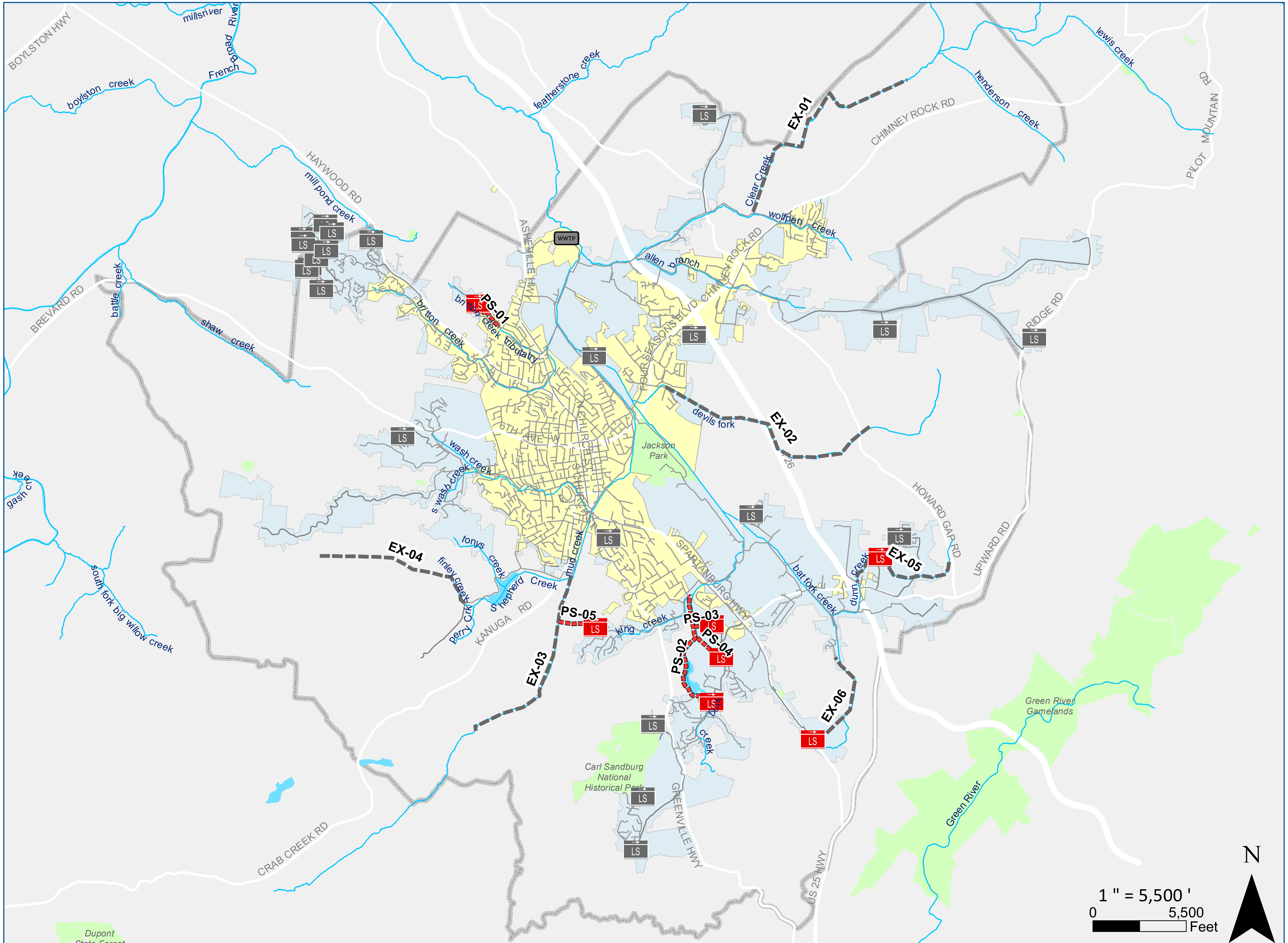
PS-01: Browning Avenue Pump Station Abandonment. Browning Ave. Pump Station can be replaced by 1,800 feet of 8-inch gravity sewer along the Britton Creek tributary. There is no time sensitive driver for this project. This project can be scheduled when COH finds the project necessary.

PS-02: Bonclarken Pump Station Abandonment. Bonclarken Pump Station can be replaced by 8,000 feet of 10-inch sewer along King Creek. There is no time sensitive driver for this project. This project can be scheduled when COH finds the project necessary.

PS-03: Highland Lake Golf Villas Pump Station Abandonment. Highland Lake Golf Villas Pump Station can be abandoned with 1,200 feet of 8-inch gravity sewer. The gravity sewer needs to connect to project PS-02. There is no time sensitive driver for this project. This project can be scheduled when COH finds the project necessary.

PS-04: Highland Lake Pump Station Abandonment. Highland Lake Pump Station can be abandoned with 2,300 feet of 8-inch gravity sewer. The gravity sewer needs to connect to the project PS-02. There is no time sensitive driver for this project. This project can be scheduled when COH finds the project necessary.

PS-05: Donroy Pump Station Abandonment. The Donroy Pump Station can be abandoned with 2,200 feet of 8-inch gravity sewer. The gravity sewer will connect into gravity extension EX-03. There is no time sensitive driver for this project. This project can be scheduled when COH finds the project necessary.



City of Hendersonville
SSAIA

Recommended
Gravity Extensions and
Pump Station Abandonments
Figure 5-10

- Pump Station
- Treatment Plant
- Abandoned Pump Station
- Pump Station Abandonment
- Gravity Extensions
- COH Collection System
- City Limits
- Existing Service Boundary
- 2040 Service Area

Hendersonville Water Sewer

BLACK & VEATCH

Table 5-10 Gravity Extensions and Pump Station Abandonment Project Details

PROJECT NAME	PROJECT START YEAR	DIAMETER (IN)	LENGTH (FT)	PROJECT COST
EX-01	2024	24	14,500	**$8,555,900**
EX-02	2029	18	14,500	**$6,409,300**
EX-03	2038	8	12,000	**$3,447,700**
EX-04	2036	10	10,000	**$2,877,400**
EX-05	2033	18	8,000	**$3,244,700**
EX-06	2037	8	6,500	**$1,870,600**
PS-01	2031	8	1,800	**$517,700**
PS-02	2034	10	8,000	**$2,339,000**
PS-03	2037	8	1,200	**$381,400**
PS-04	2037	8	2,300	**$862,400**
PS-05	2037	8	2,200	**$673,900**
Total Cost				**$31,180,000**

6.0 Project Prioritization

All projects identified in Section 5 were based only on the capacity assessment of the hydraulic model. To provide risk-based pipe prioritization more analysis was required. Black & Veatch used the system information from GIS, defects identified from condition assessment (Section 2) along with the results from the hydraulic modeling (Section 5) to develop a risk-based pipe prioritization to develop COH's CIP. A set of the LOF and COF criteria was selected to quantify the relative importance of each pipe segment, which is referred to as the risk. The risk is based on the acceptable levels of service and impacts to the social, economics, health, and safety factors.

Black & Veatch worked with COH staff on score criteria during the August 29th, 2018 Risk Prioritization conference call. Feedback from COH staff was incorporated into the final scoring and weighting. A scoring range of 1 to 5 was used for the COF criteria and 1 to 5 for the LOF criteria. An importance weighting was applied to each of the factors to determine the overall risk score for each individual pipe. These factors are described in detail in the following sections.

6.1 LIKELIHOOD OF FAILURE

LOF factors account for the physical characteristics as well as the level of service performance of the pipe segments. The four parameters that are used to define these factors are as follows:

- Pipe material.
- Pipe age.
- Pipe capacity.
- Basin I/I rate.

These factors are used to account for physical properties that are associated with pipe failure. Attributes were ranked on a scale of 1 to 5, where 1 represents the lowest LOF and 5 represents the highest LOF. A score of 5 was reserved for pipes with structural defects. Pipe material was weighted as 20 percent, pipe age was 20 percent, and pipe capacity or basin I/I was 60 percent. In addition to these factors, the smoke and acoustic testing and proximity to streambanks were incorporated into the LOF. The selected LOF criteria and associated scoring are described below.

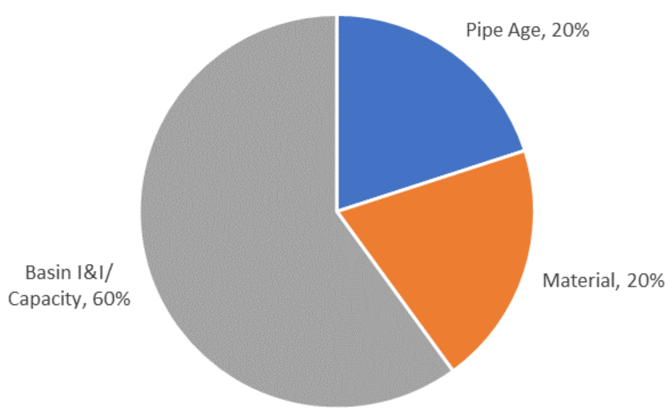


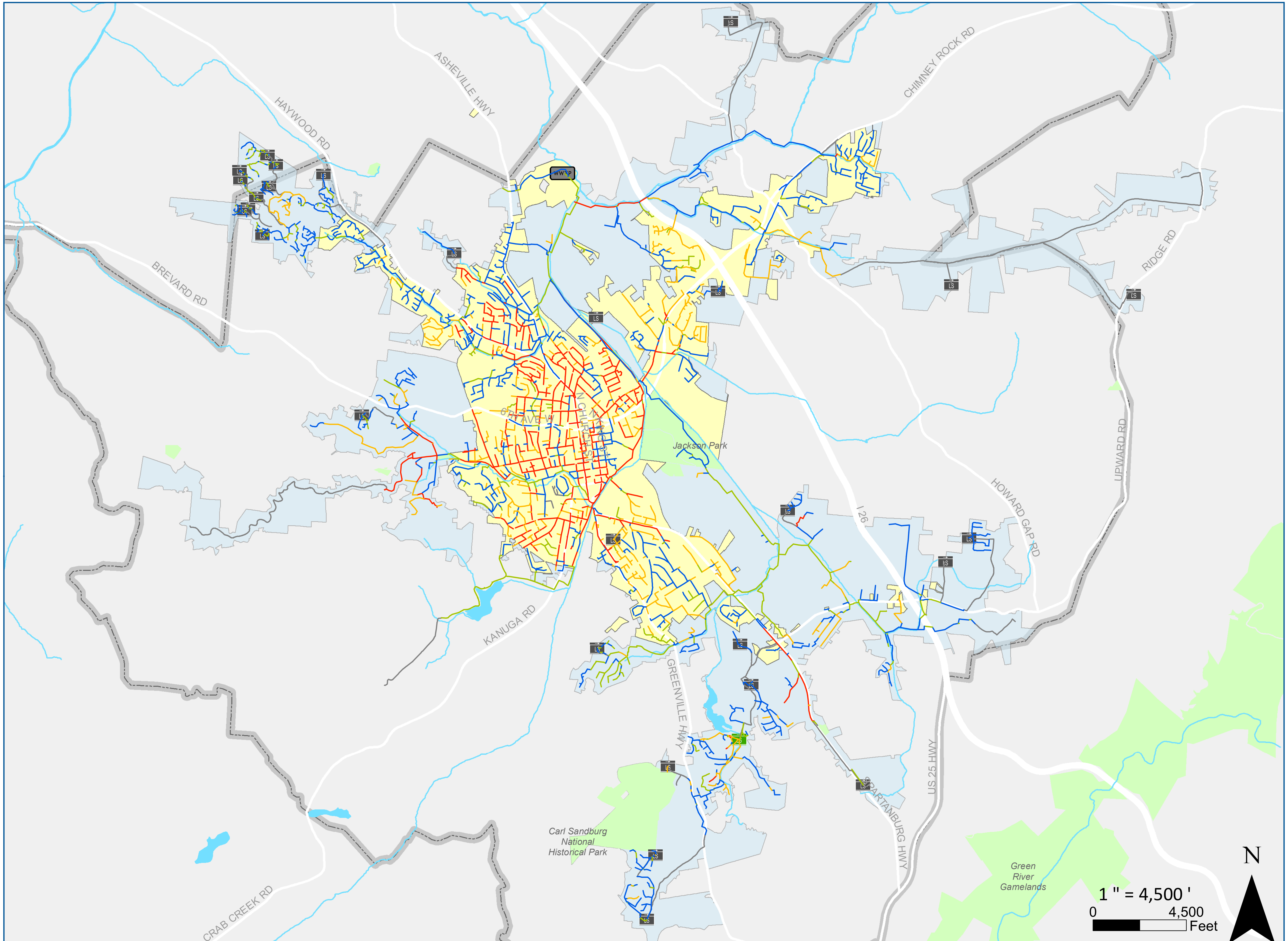
Figure 6-1 LOF Relative Weighting

6.1.1 Pipe Material

Scores from 1 to 4 were assigned based on susceptibility to corrosion, construction practices, and the performance track record of various pipe materials. Table 6-1 represents the assigned scores relative to various pipe materials. The plastic pipe materials (PVC and HDPE) are generally less susceptible to corrosion and have a lower frequency of failure. Therefore, plastic pipe materials are scored as a 1. The ductile and cast-iron pipe are more susceptible to corrosion than plastic pipe so it is scored a 2. A concrete pipe is susceptible to hydrogen sulfide attack that contributes to its deterioration and failure. Although a concrete pipe is strong; however, it is scored a 3 because of the corrosion from hydrogen sulfide sewer gasses. The vitrified clay or clay tile pipe is not susceptible to corrosion but construction practices result in joint failures that allow inflow and infiltration and root intrusion to cause this pipe to fail. Therefore, this material is scored a 4.

Table 6-1 Likelihood of Failure Scores for Material

PIPE MATERIAL*	SCORE	PERCENTAGE
PVC/HDPE	1	43%
Ductile/Cast Iron	2	14%
Concrete/Unknown	3	21%
Vitrified Clay Pipe (VCP)	4	22%



City of Hendersonville
SSAIA
Likelihood of Failure
Pipe Materials Scoring
Figure 6-2

LOF Pipe Material Scoring

- 1 - PVC/HDPE
- 2 - Ductile/Cast Iron
- 3 - Concrete/Unkown
- 4 - VCP
- City Limits
- Existing Service Boundary
- 2040 Service Area

Hendersonville Water Sewer

BLACK & VEATCH

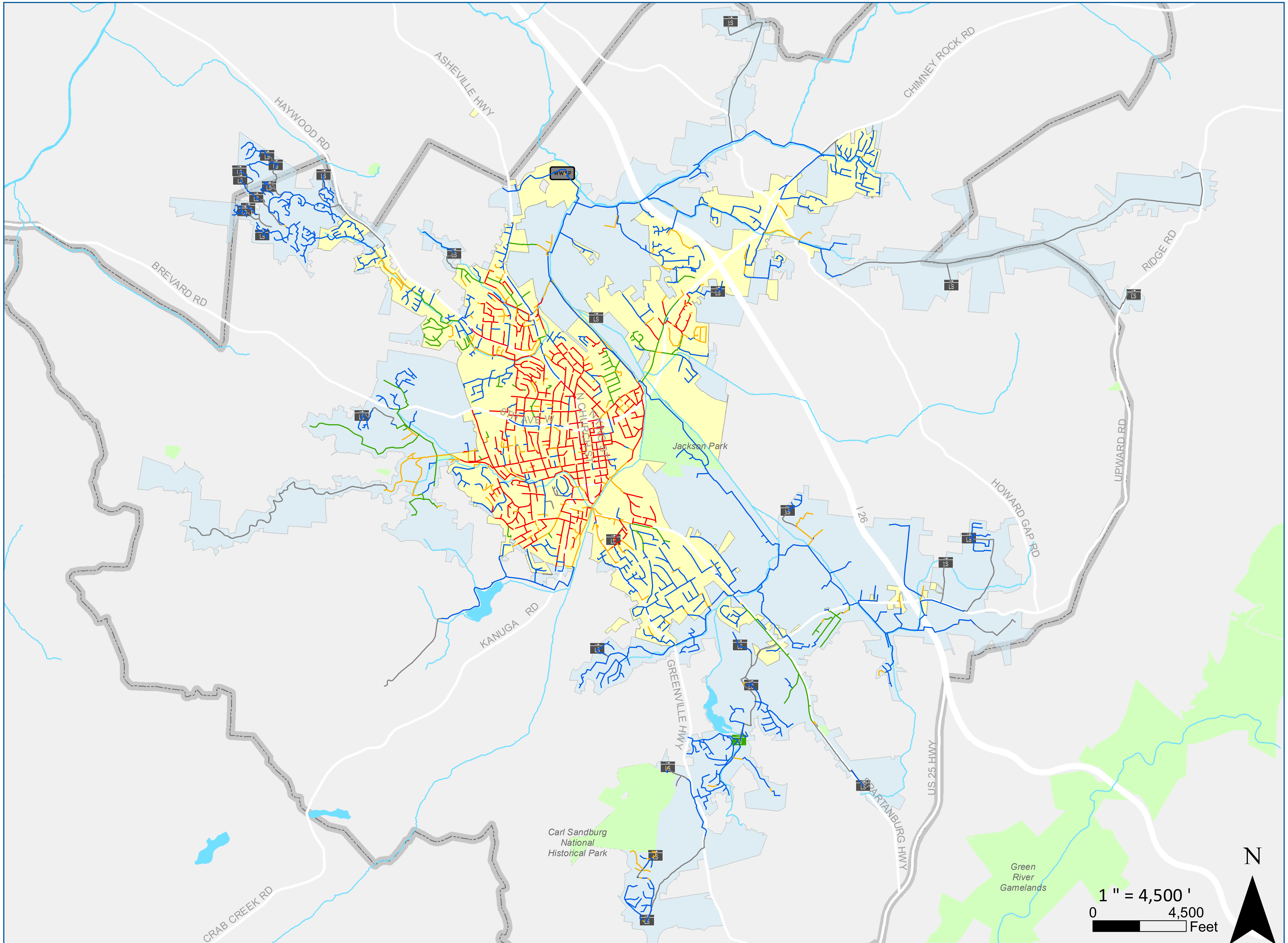
6.1.2 Pipe Age

The LOF of the pipes increases as the pipe ages. The life span of a pipe can vary based on material, installation methods, and soil conditions. While many pipe manufacturers claim life spans of 100 years (PVC, Ductile Iron, etc), many pipes will deteriorate or become otherwise compromised in a much shorter time span. Based on Black & Veatch experience and the existing condition in the pipes observed in the COH system, it is assumed that the maximum useful life of pipes is 75 years. Scores for pipe age are assigned based on the age of the pipe, relative to the maximum useful life of 75 years. With proper maintenance, pipes older than 75 years and in good condition can and should stay in service. Prioritizing the older sections of pipe will ensure that condition issues are resolved and repairs made in a timely manner in order to maximize the longevity of the system. Table 6-2 represents the scores assigned with respect to pipe age. For pipes with unknown pipe age, a score of 3 was assigned.

Table 6-2 Likelihood of Failure Scores for Age

PIPE AGE (YEARS)	SCORE	PERCENTAGE
<30	1	54%
30 - 50	2	9%
50 -75/ Unknown	3	13%
> 75	4	24%

City of Hendersonville
SSAIA
Likelihood of Failure
Pipe Age Scoring
Figure 6-3



LOF Pipe Age Scoring

- 1 - < 30 years
- 2 - 30 - 50 years
- 3 - 50 -75 years/Unknown
- 4 - > 75 years
- City Limits
- Existing Service Boundary
- 2040 Service Area

Hendersonville Water Sewer

BLACK & VEATCH

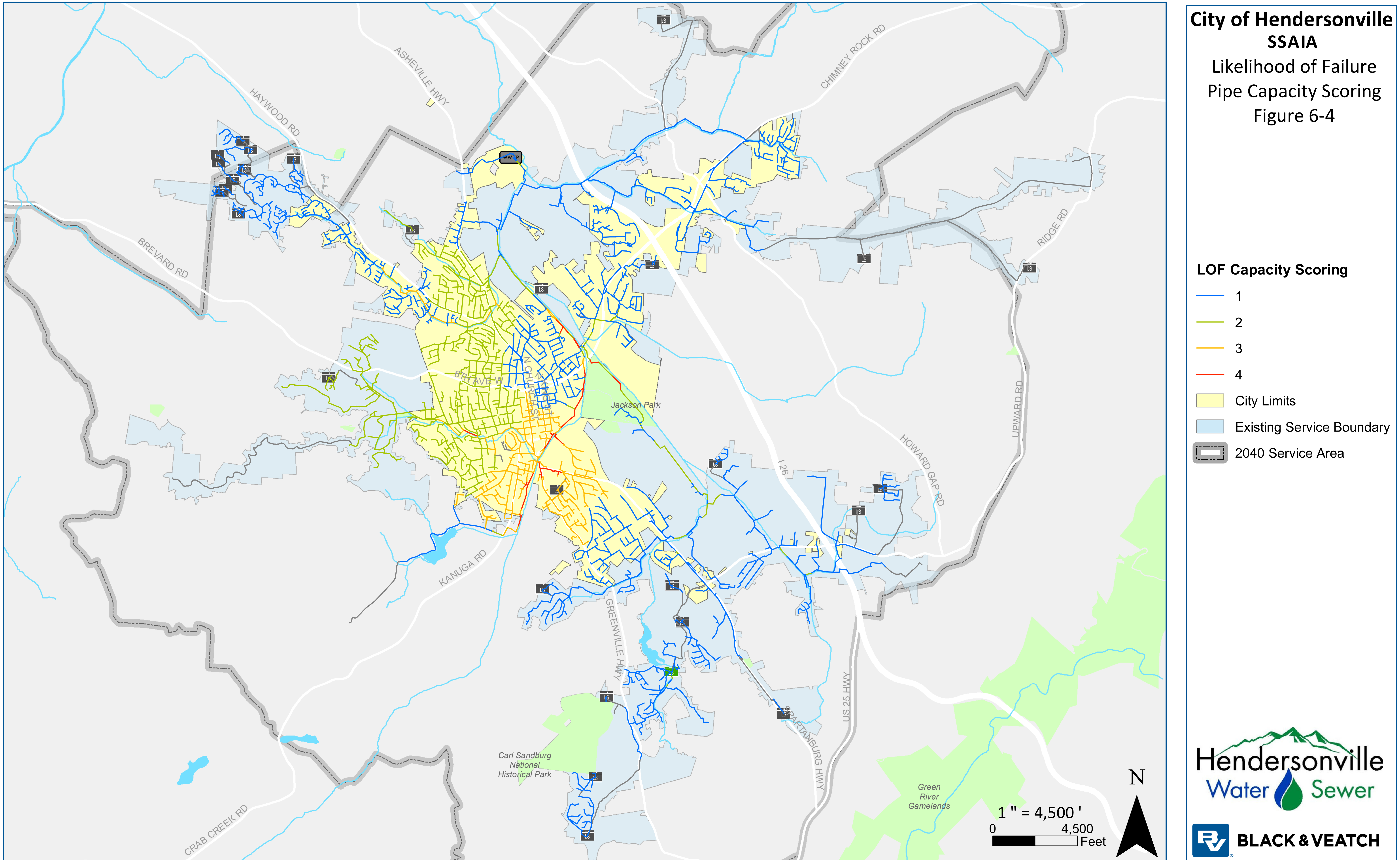
Template for Figures 6- February 28, 2019

6.1.3 Pipe Capacity

Hydraulic capacity and I/I rate were evaluated to assign scores for each pipe segment, with pipe SSOs indicative of pipe failure. All pipes within a basin were assigned a score based on the I/I rate from the calibration. Additionally, the score would be elevated if the pipe showed surcharge or an SSO in the capacity analysis. The segments are considered to have failed if they are determined to be flooding under selected storm events. If the pipes had no surcharge or a basin I/I rate less than three percent they were assigned a score of 1. Pipes that had higher than 3 percent I/I rates or surcharged pipe in a 2-year storm were assigned a score of 2. If pipes flooded during a 10-year event or had a basin I/I rate greater than five percent, the pipes were assigned a score of 3. Higher scores were assigned if a pipe segment was determined to be flooded under a less severe storm event, as shown in Table 6-3.

Table 6-3 Likelihood of Failure Score for Capacity

AVAILABLE CAPACITY	SCORE	PERCENTAGE
No surcharge or basin I/I Rate < 3%	1	57%
Surcharge in a 2-year storm or basin I/I Rate > 3%	2	28%
SSO in a 10-year event or basin I/I Rate >5%	3	13%
SSO in a 2-year event	4	2%

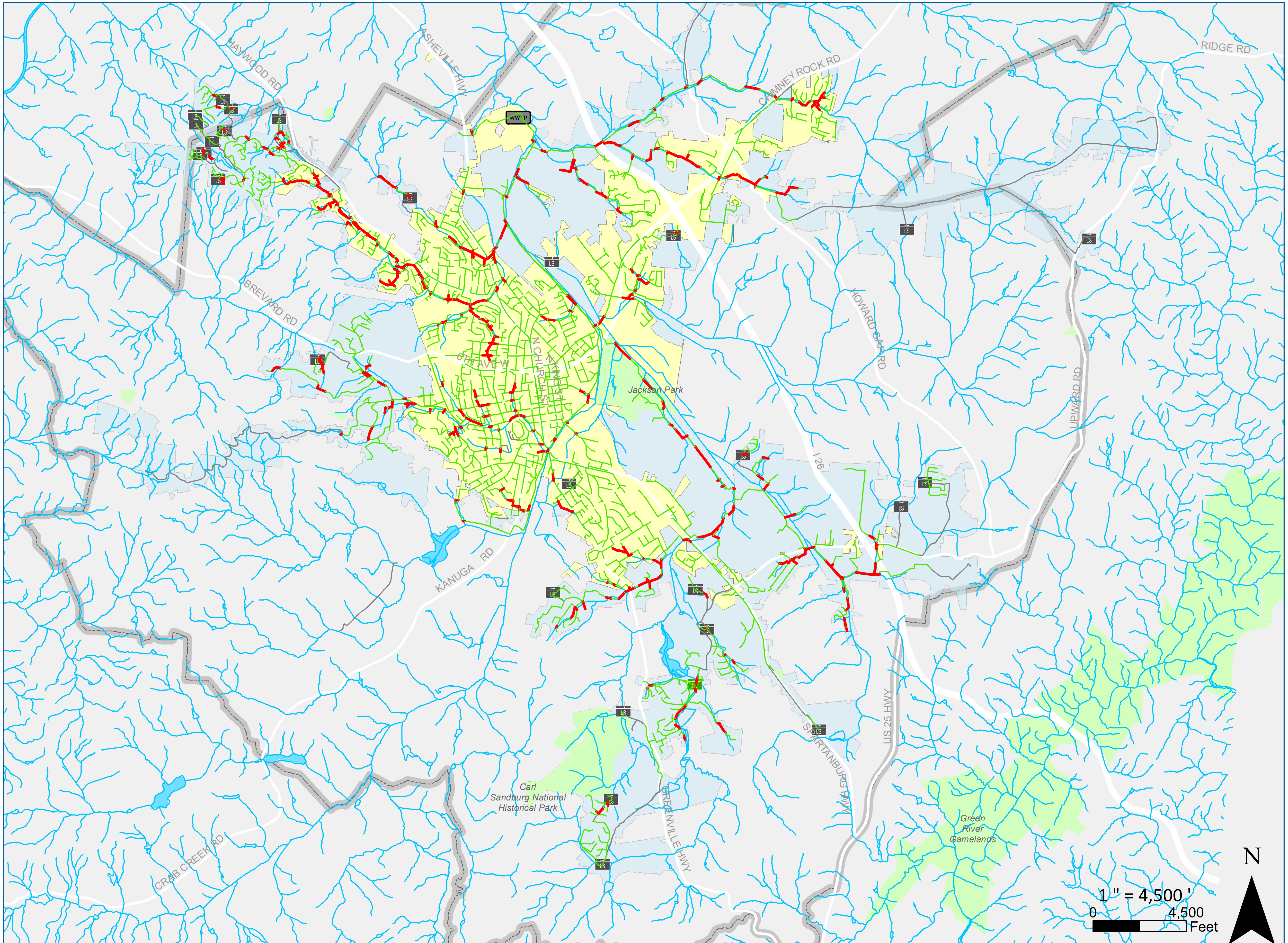


Template for Figures 6- February 12, 2019

6.1.4 Additional LOF Factors – High Priority

The smoke testing, acoustic testing, and proximity to stream banks were all factors included in the prioritization. The smoke and acoustic testing data from Section 2 were incorporated into the LOF. For the acoustic testing, if the scoring for the test was less than a 3, there was a structural defect. The smoke testing also detected defects. These were the highest priority and scored as a 5 for the LOF. Additionally, 6-inch diameter sewers with known defects should be elevated to a LOF score of 5. This will result in current CIP projects being elevated to high priority replacements.

As stream banks erode, they become more unstable and can incur the risk of collapse and damage to any pipes within 20 feet. The streams geodatabase: *Henderson_Effective_PGDB_Final.mdb* was provided by COH to calculate the pipes near streams. Pipes that were near the stream were assigned an additional LOF score of 0.5.



City of Hendersonville

SSAIA

Likelihood of Failure
Streambank Scoring
Figure 6-5

LOF Streambank Addition

- >20 ft from Stream
- Wthin 20 ft of Stream
- City Limits
- Existing Service Boundary
- 2040 Service Area

Hendersonville Water Sewer

BLACK & VEATCH

6.2 CONSEQUENCE OF FAILURE

Consequences of failure factors take into account the location and pipe diameter, and the subsequent environmental and public impact caused as a result of pipe failure. A scale of 1 to 5 was used to represent the various level of the consequence of failure, where 1 represents the lowest level of consequence and 5 represents the most severe consequence of failure. The specific risk factors used were sewer locations, shown in Table 6-4.

The consequence of failure scores based on the location of the pipe alignment relative to the surrounding area is shown in Table 6-4. Alignment score considers proximity to structures, public buildings, commercial areas, and wetlands/creeks.

Table 6-4 Consequence of Failure Scores for Alignment

ALIGNMENT	SCORE	PERCENTAGE
Easement (Assumed 50 feet around Centerline (CL))	1	22%
Easement close to Structures (Assumed 5 feet around CL)	2	20%
Roadway (Assumed 25 feet around CL)	3	37%
Highway (Assumed 75 feet around CL)	4	12%
Pipes Serving Public Building/Commercial Areas (hospitals, schools, and shopping centers)	5	2%
Wetland/Creek	5	7%

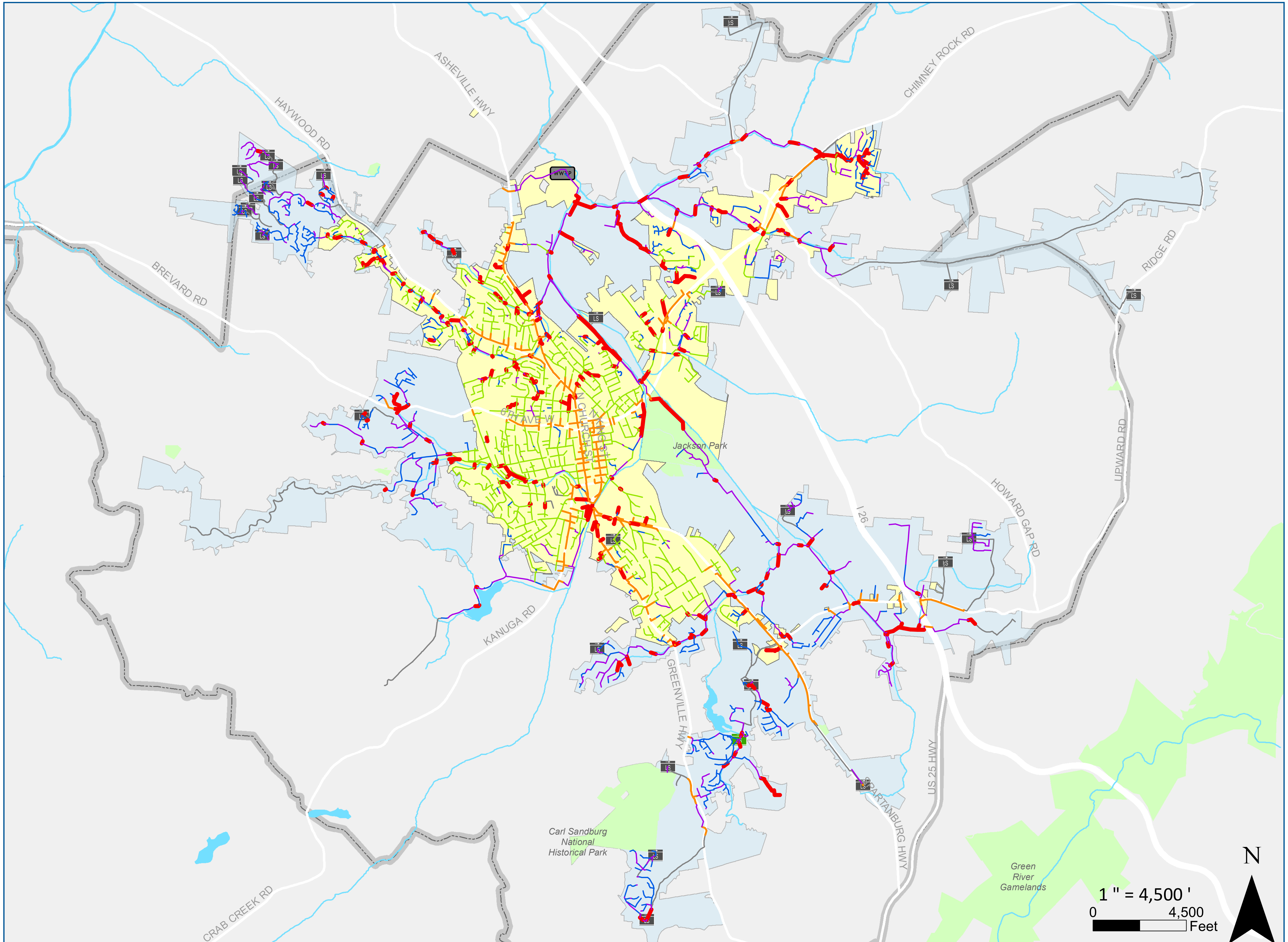
A diameter risk score was assigned to each pipe. Larger pipes carry incur more risk because the larger pipe take longer to replace for significantly more cost. For pipes larger than 24 inches, 1 point was added to the pipe score, and a half point was added for pipes from 12 inches to 24 inches. All pipes less than 12 inches were scored as a zero because these pipes are typically easy and cheaper to replace. A diameter risk factor was an additional 1 point that was added to the COF, shown in Table 6-5.

Table 6-5 Consequence of Failure Score for Diameter

DIAMETER	SCORE	PERCENTAGE
Less than 12 inch	0	75%
12 inch to 24 inch	0.5	14%
More than 24 inch	1	11%

Figure 6-6 below shows the alignment scores for each pipe. The pipes near downtown Hendersonville typically have risk and consequences of failure because of the proximity to the streets and businesses downtown.

City of Hendersonville
SSAIA
Consequences of Failure
Pipe Alignment Scoring
Figure 6-6



Hendersonville Water Sewer

BLACK & VEATCH

Template for Figures 6- February 12, 2019

6.3 FORCE MAIN PRIORITIZATION

The Bonclarken force main was the only force main that was part of the skeletonized model and had sufficient capacity. However, other force mains were not evaluated for capacity and that criteria was left out of the force main LOF. Instead, the force mains were weighted 50 percent on mater, and 50 percent on age. While most of the force mains in the system were plastic pipe with small diameters, the force mains varied in age and alignment risk scoring. Table 6-6 shows the force mains risk scores:

Table 6-6 Force Main Scoring

PUMP STATION NAME	FORCE MAIN RISK SCORE
Blythe St	3
Bonclarken	3
Browning Ave	12.5
Carl Sandburg	12.5
Carriage Park	6.05
Clear Creek School	10
Custom Pac	4.5
Dana Elementary School	10
Dunroy	4.5
Eagle Pointe	6
Garden Lane	12.5
General Electric	7.5
Highland Lake	6
Highland Lake Golf Villas	6
Kenmure Brookwood	6
Kenmure Driving Range	7.5
Lakewood RV	7.5
Crest Road	3
Lower King Creek	8
Outback Restaurant	3
Shaws Creek Farm	6
Somersby Park	6
Sugarloaf School	9
The Orchards	5
Tom’s Hill	6.75

6.4 RISK ANALYSIS RESULTS

An overall risk score was calculated for each pipe segment. This overall score was derived from multiplying the total LOF and COF scores. The individual LOF and COF scores were calculated by multiplying the factors described above by the assigned weighting then summing them together. The risk was calculated by multiplying the LOF and COF together. The formulas used to calculate the risk score for each pipe is shown below:

$$\text{LOF Score} = ((L_{\text{Material}} * 0.2) + (L_{\text{Age}} * 0.2) + (L_{\text{Capacity}} * 0.6)) + L_{\text{Stream Bank}}$$

$$\text{COF Score} = (C_{\text{Alignment}} + C_{\text{Diameter}})$$

$$\text{Risk} = \text{COF} * \text{LOF}$$

For example, pipe 1818 is a pipe segment in critical project G-06. This pipe was a clay pipe ($L_{\text{Material=}}$4), has unknown age ($L_{\text{Age=}}$3), has 2-year SSO potential ($L_{\text{Capacity=}}$4), crossed Mud Creek ($L_{\text{Stream Bank =}}$0.5), has 24 inch diameter (C_{Diameter}=0.5), and was next to S. King Street($C_{\text{Alignment}}$=4). Pipe 1818 has a total risk score of 19.35:

$$\text{LOF Score} = ((4 * 0.2) + (3 * 0.2) + (4 * 0.6)) + 0.5 = 4.3$$

$$\text{COF Score} = (4 + 0.5) = 4.5$$

$$\text{Risk} = 4.5 * 4.3 = 19.35$$

The risk model allows for classification of pipe segments and force mains based on LOF and COF scores, using a risk matrix. The risk matrix classifies the range of LOF and COF scores into 5 levels: Low, Medium Low, Medium, Medium High, and High as shown in Table 6-7. The distribution of pipe segment counts and lengths can be found in Table 6-8. Most of the pipes have a low LOF score (163 miles less than 3). Additionally, a significant number of pipes have high COF scores (110 miles greater or equal 3). Overall, this is a medium level of system risk with a total average non-weighted risk of 5.5.

Table 6-7 Scoring Categories

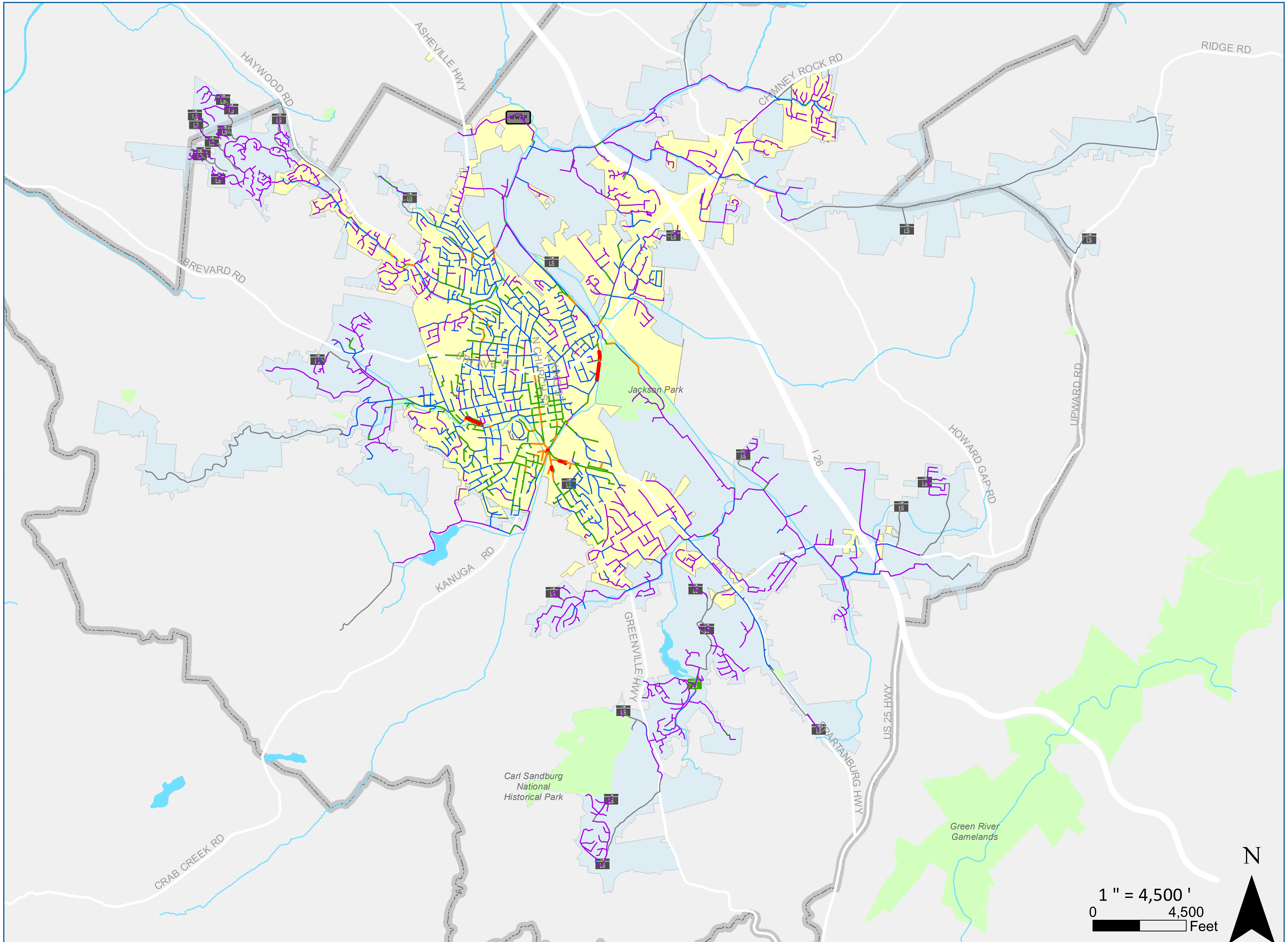
SCORING	COF	LOF
High	>5 (34 miles, 19%)	5 (1 miles, 0%)
Medium-High	4 (26 miles, 14%)	4 (1 miles, 0%)
Medium	3 (60 miles, 34%)	3 (16 miles, 9%)
Medium Low	2 (39 miles, 21%)	2 (125 miles, 69%)
Low	1 (21 miles, 12%)	1 (38 miles, 21%)

Table 6-8 Existing Conditions, Risk Summary

COF_Category	1 LOF - Low	2 LOF - M. Low	3 LOF - Medium	4 LOF - M.High	5 LOF - High
5 COF - High	55 pipes, 2.5 Miles	356 pipes, 26.6 Miles	100 pipes, 4.2 Miles	9 pipes, 0.4 Miles	3 pipes, 0.1 Miles
4 COF - M.High	97 pipes, 3.1 Miles	503 pipes, 17.9 Miles	121 pipes, 4.8 Miles	1 pipes, 0.1 Miles	6 pipes, 0.4 Miles
3 COF - Medium	213 pipes, 9 Miles	1283 pipes, 45.2 Miles	138 pipes, 5.2 Miles	2 pipes, 0.1 Miles	11 pipes, 0.4 Miles
2 COF - M. Low	439 pipes, 13.9 Miles	683 pipes, 23.8 Miles	24 pipes, 1.2 Miles	3 pipes, 0.2 Miles	
1 COF - Low	334 pipes, 9.3 Miles	396 pipes, 11.7 Miles	5 pipes, 0.2 Miles		

Figure 6-7 shows the Total Risk Scores for the existing system, which illustrates the mud creek interceptor has the highest risk and in need of improvements. The list of projects developed in Chapter 5 were prioritized by immediate capacity concerns, NC-DOT projects overlap and risk score. For the NC-DOT projects overlap, improvements were scheduled for design one year before the NC-DOT projects were due for construction so that the following year they could both be constructed in tandem. Figure 6-8 shows the proposed gravity sewer improvements.

City of Hendersonville
SSAIA
Likelihood of Failure
Risk Score Map
Figure 6 - 7



Risk Score

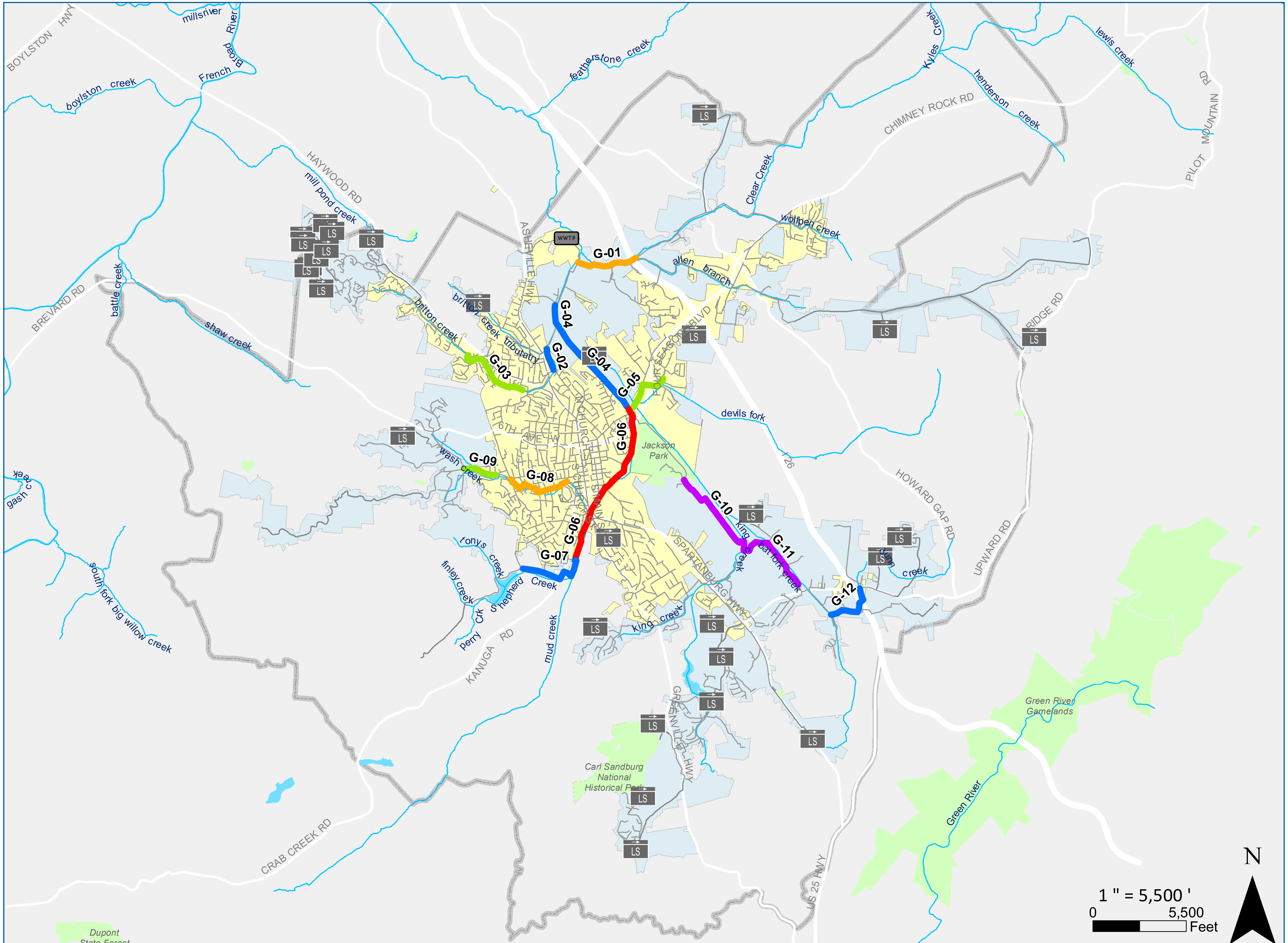
- 1 - 5 - Low
- 6 - 10 - Medium Low
- 11 - 15 - Medium
- 16 - 20 - Medium High
- > 20 - High
- City Limits
- Existing Service Boundary
- 2040 Service Area

Hendersonville Water Sewer

BLACK & VEATCH

City of Hendersonville SSAIA

Prioritized Improvements
Figure 6-8



LS	Pump Station
WWTP	Treatment Plant
	Critical Projects (2019)
	2020 - 2025
	2026 - 2030
	2031 - 2035
	2036 -2040
	COH Collection System
	City Limits
	Existing Service Boundary
	2040 Service Area

Hendersonville Water Sewer

BLACK & VEATCH

7.0 Capital Improvement Plan and Recommendations

The purpose of this chapter is to document the recommended future projects. All improvements were organized into the planning periods based on the future modeled planning years (2018, 2025, 2040). Projects were delegated into 5-year planning periods based on the capacity assessment (Section 5.0) and condition assessment (Section 2.0) results along with the final prioritization scores (Section 6.0).

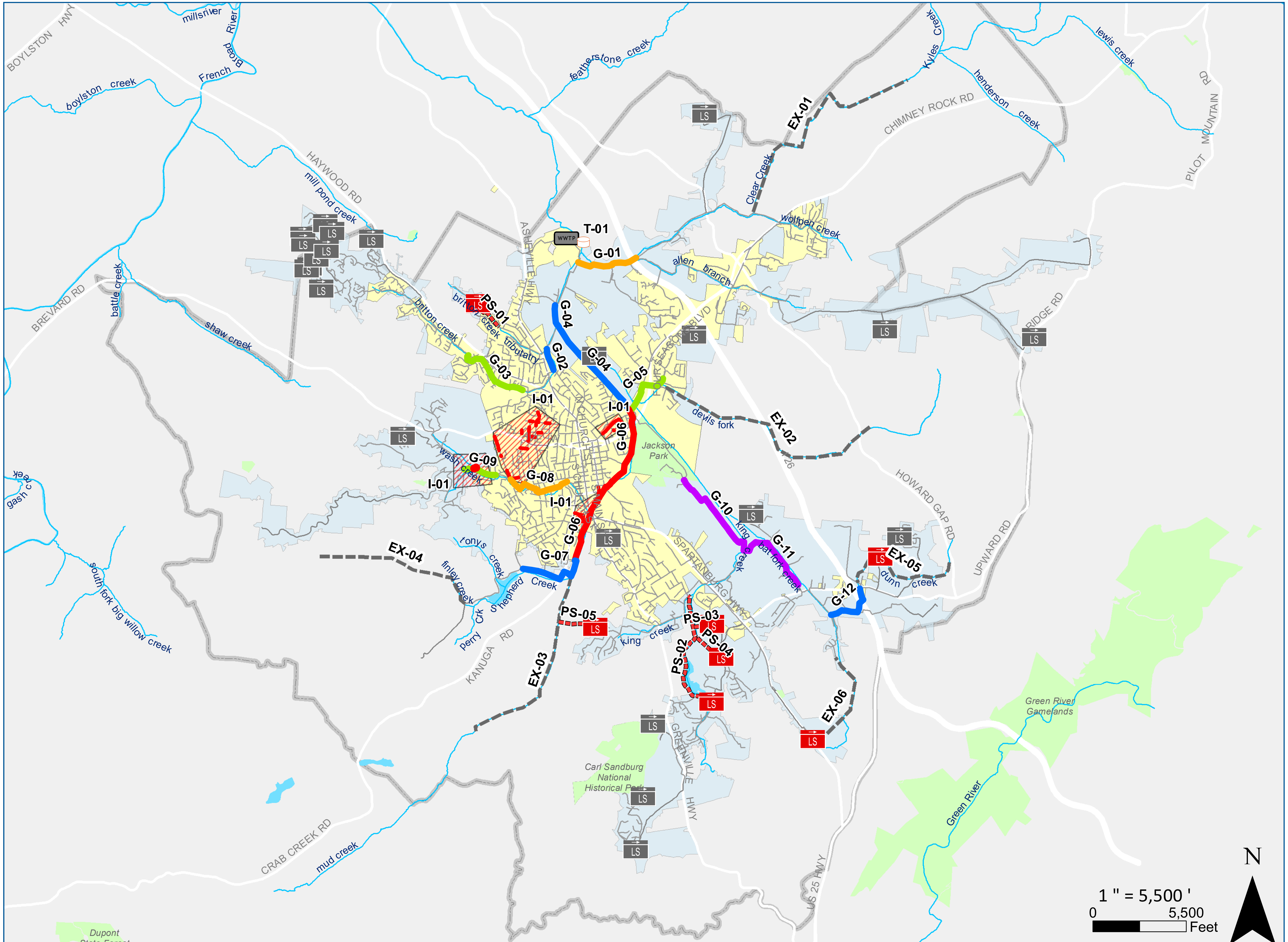
7.1 SUMMARY OF PROJECTS

The improvement projects were programmed to start by ordering them first by capacity during the capacity assessment. For capacity projects recommended between 2025-2040, the CIP projects were prioritized by risk score. Additionally, projects that involved NCDOT road crossings were accelerated ahead of projects with higher risk so that the project can be constructed in tandem with the road crossing. The projects that prevent SSOs are the highest priority. Projects were programmed based on the project start year. For most projects, a one-year duration was designated for planning and design. Construction was assumed to begin after the completion of the design duration. The year design begins and the project details are shown below in Table 7-1. Planning and Design should be completed prior to the construction start year to ensure that construction is completed on time. The location of each project listed in Table 7-1 is shown in Figure 7-1.

Table 7-1 Project Details

Design Start Year		PROJECT ID	PROJECT DESCRIPTIONS	RISK SCORE	TOTAL COST
2019	2019	G-06	Replacement sewer along Mud Creek near Railroad	11.9	$9,225,500
		Manhole Inspection Plan	Complete the manhole inventory and inspection with a concentrated effort in the next year	N/A	To be completed by City Staff
		T-01	Equalization Basins and WWTP Capacity Study	N/A	Projects to be provided by Addendum to this report
		Lift Station Maintenance	Support the slope at lift station 037 Carriage Park. Repair or replace the check valve, update the disconnect, and repair or replace the pump rail system at 003 Garden Lane	N/A	To be completed by City Staff
2020 – 2025	2021	G-08	Wash Creek Replacement Sewer	12.3	$3,266,300
	2022	Force Main Inspection Plan	Force main inventory and inspection based on prioritization	N/A	To be completed by City Staff
	2022	G-01	Clear Creek Sewer Replacement near Future Greenway	5.5	$3,669,600
	2024	EX-01	New sewer line extension along Clear Creek	N/A	$8,555,900
2026 - 2030	2027	G-05	Devils Fork Sewer Replacement near MLK Jr Blvd	8.0	$2,390,700
	2028	G-03	Brittain Creek Sewer Replacement near Haywood Rd	7.9	$2,503,200

Design Start Year		PROJECT ID	PROJECT DESCRIPTIONS	RISK SCORE	TOTAL COST
	2029	EX-02	New sewer line extension along Devils Fork	N/A	$6,409,300
	2029	G-09	Wash Creek Replacement Sewer near Wash Creek Dr	7.9	$1,267,200
2031 - 2035	2031	PS-01	Browning Avenue Pump Station Abandonment	N/A	$517,700
	2031	G-04	Mud Creek Parallel Replacement	7.2	$7,511,700
	2032	G-02	Brittain Creek Sewer Replacement near Patton Park	7.9	$1,007,300
	2033	G-12	Dunn Creek Replacement Sewer near I-26	5.5	$2,060,400
	2033	EX-05	New sewer line extension along Dunn Creek	N/A	$3,244,700
	2034	PS-02	Bonclarken Pump Station Abandonment	N/A	$2,339,000
	2035	G-07	Shepherd Creek Replacement Sewer near Kanuga Rd	4.9	$2,277,300
2036 - 2040	2036	EX-04	New sewer line extension along Finley Creek	N/A	$2,877,400
	2037	EX-06	New sewer line extension along Bat Fork Creek	N/A	$1,870,600
	2037	PS-03	Highland Lake Golf Villas Pump Station Abandonment	N/A	$381,400
	2037	PS-04	Highland Lake Pump Station Abandonment	N/A	$862,400
	2037	PS-05	Donroy Pump Station Abandonment	N/A	$673,900
	2038	EX-03	New sewer line extension along Mud Creek	N/A	$3,447,700
	2039	G-10	King Creek Replacement Sewer near Airport	3.3	$4,900,700
	2040	G-11	Bat Fork Replacement Sewer near Blue Ridge Community College	2.4	$3,158,800
Annual Inspection Plan		I-01	Annual System Inspection. Continue system inspections annually. In the next 3 years, the City should plan to complete inspections of all pipes with high LOF scores. Field testing should be a combination of smoke, acoustic and CCTV testing.	N/A	To be completed by City Staff



City of Hendersonville
SSAIA

Recommended Improvement Projects
Figure 7-1

- EQ Tank
- Pump Station
- Treatment Plant
- Abandoned Pump Station
- Pump Station Abandonment
- Critical Projects (2019)
- 2020 - 2025
- 2026 - 2030
- 2031 - 2035
- 2036 -2040
- CCTV Investigation
- Gravity Extensions
- COH Collection System
- City Limits
- Existing Service Boundary
- 2040 Service Area

Hendersonville Water Sewer

BLACK & VEATCH

7.2 RECOMMENDATIONS FOR FURTHER ANALYSES AND UPDATES

7.2.1 Capacity and CIP Prioritization

Based on the future year analysis, capacity assessment, and project prioritization work, it is recommended that the City:

- Install critical project G-06 as soon as possible. The base year model showed this stretch of sewer was undersized for the current loadings. Project G-06 is needed to mitigate the risk of an SSO occurring during a 2-year event in the base year. Additionally, this pipe also has a high-risk score (11.9) further highlighting the need to perform this project.
- Make improvements to the WWTP to increase the hydraulic capacity of the plant to treat additional flows during a wet weather event. An alternative analysis has been initiated to determine the best strategy for operational improvements. In addition, the City should study adequate EQ capacity to enable the WWTP to handle 2-year I/I rates. The EQ should be sized as part of the alternatives analysis currently in progress. The recommended operational improvements and EQ sizing will be included in an addendum to this report.
- Upgrade the WWTP capacity to meet future demands. The current discharge permit is 6 MGD; however, the plant is rated for 4.8 MGD. The system flows are approaching the 4.8 MGD maximum month flow and the City should begin to investigate opportunities to upgrade their plant to 6 MGD in the near future. By the end of the planning period (2040), the projected flows are expected to exceed 6 MGD, so the next plant design should also address future expansion beyond 6 MGD.
- Revisit the modeling software selection. The InfoSewer software package has limited capabilities. The hydraulic engine provides poor results when calculating flow depths through adverse slope pipes and parallel pipes. InfoSewer also does not allow for the addition of sediment in pipes. The City should consider switching to Innovyse's InfoSWMM in the next model update. InfoSWMM is less cost-prohibitive than InfoWorks, but still provides a more robust hydraulic engine than InfoSewer for dealing with parallel and adverse slope pipes, both of which are seen in the City's modeled collection system.
- Update the Master Plan every 5 years to reevaluate system growth and to continue to recommend proper improvement projects to provide excellent service for the future.

7.2.2 Condition Assessment

Recommendations derived from Phase 1 and Phase 2 work are to address the deficiencies noted during the inspections and to maintain or improve the condition of the piping. The following recommendations are made:

- Conduct CCTV inspection of the segments identified by the smoke testing as having severe and moderate defects and the segments with scores of blocked or poor from the acoustic testing and acoustic testing.
- Continue in-house smoke testing in areas identified in the Inspection Plan and as indicated by flow data.
- Complete the manhole inventory and inspection with a concentrated effort in the next year.
- Implement a program to inspect the 16 miles of pipelines in the system with a high LOF score within the next 3 years. This will provide a baseline inspection of these pipelines that can be used to measure performance within the collection system in the future. The

inspections can be smoke testing, acoustic testing, or CCTV, depending on the prioritization of the pipeline. If in-house inspections have been completed of these priority pipelines, the work should have been completed within the past 5 years.

- Incorporate acoustic testing using SL-RAT used in Phase 2 as part of the inspection procedures.
- Continue to update the Inspection Tracker tool with new inspection data collected in the future.
- Complete the following maintenance needs identified in the lift station inspections performed by COH:
  - Support the slope at lift station 037 Carriage Park.
  - Repair or replace the check valve, update the disconnect, and repair or replace the pump rail system at 003 Garden Lane.

The force mains were not included in the condition assessment work but should be inspected within the next 5 years to document their condition and determine whether repair and replacement are required as part of the capital plan. However, force mains were scored as part of the risk analysis. The results from this analysis can be now be used to prioritize the force main inspections plan. A force main inspection plan would include the following:

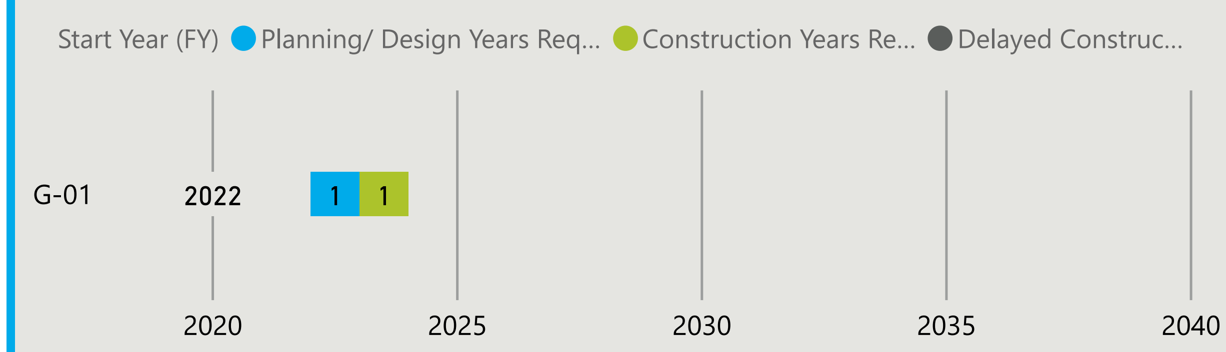
- Develop an inventory of the pipe material, age, diameter, and length from the GIS.
- Prioritize the force mains using a risk analysis approach that utilizes the LOF multiplied by the COF to create a risk-based ranking. A preliminary ranking is included in Chapter 6.
- Identify inspection technologies (leak detection, ultrasonic testing for wall thickness, or electromagnetic if the pipe is out of service) for gathering data on the condition of the force mains.
- Conduct inspections of the force mains according to the prioritized rankings. The higher ranking force mains would be inspected in more detail than the lower ranking force mains.

Project Name

G-01

Clear Creek Sewer Replacement near Future Greenway

Start Year	Total Project Cost	
2022	$3.67M	$16,300 Pipeline Easement Cost
	Weighted Risk Score	$609,000 Engineering Cost
	5.7	$2,537,300 Construction Cost
		$507,000 Scope Contingency



Pipe Details	Pipe #1
Downstream Manhole	195
Upstream Manhole	5190
Diameter	30
Length	3630

Scope:

Approximately 6,730 feet of 30-inch gravity sewer will replace the existing 18-inch and 24-inch that parallels Clear Creek. The sewer will follow the existing alignment along Clear Creek and will cross Clear Creek once and Allen Branch once. This project will require 3 road crossings: Clear Creek Road, I-26, Nix Rd.

Project Objective and Benefit:

This improvement will relieve surcharging and potential overflows along Clear Creek. This improvement is driven by future flows. The model predicts SSOs during a 2-year storm starting in 2040. The LOF risk ranged from 1.5-2.5. This project should be coordinated with the NCDOT I-26 (25-64) project.

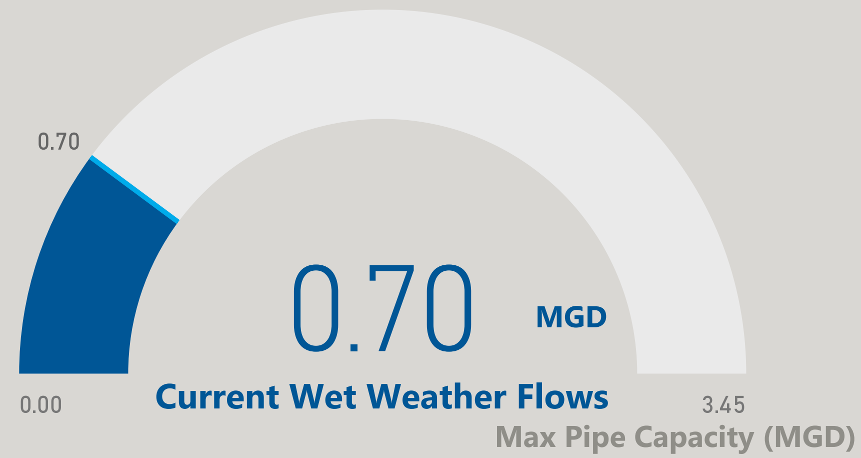
Hendersonville Water Sewer

Capacity Tracking

Master Plan Trigger

2040 2YR SSO

	Flow (MGD)
Available Capacity	2.75
Development Flow	0.00
Existing Pipe Capacity	3.45
Existing 2 YR Storm Flow	0.70

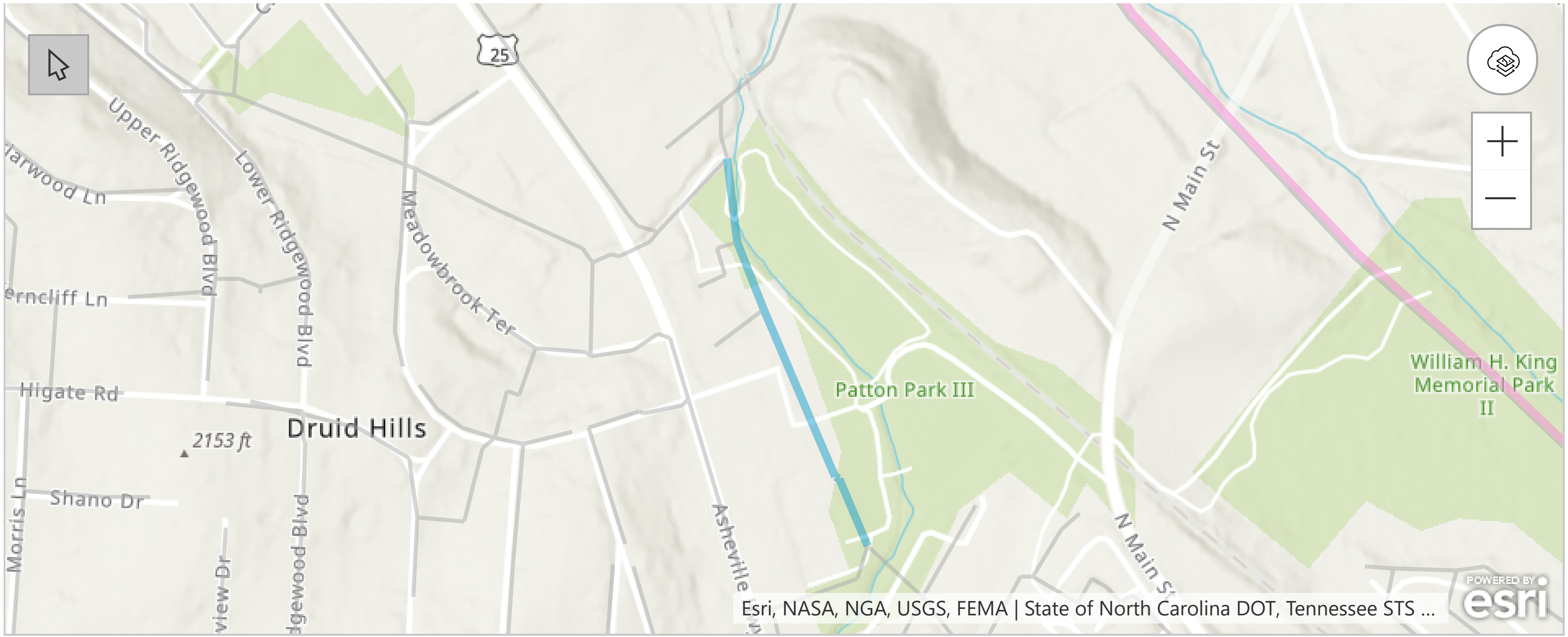


Future Improvement

Project Name

G-02

Brittain Creek Sewer Replacement near Patton Park



Pipe Details	Pipe #1
Downstream Manhole	2007
Upstream Manhole	2118
Diameter	24
Length	1700

Hendersonville Water Sewer

Scope:

Approximately 1,700 feet of 24-inch gravity sewer will replace the existing Brittain Creek outfall. The new sewer has 1 road crossing: E Clairmont Dr.

Project Objective and Benefit:

This improvement will relieve surcharging and potential overflows along Brittain Creek. This improvement is driven by wet weather flows. The model predicted SSOs in a 10 year event, but only surcharging in a 2 year event. The improvement will reduce the LOF risk, which ranges from 2.0-2.9 because of the 92 year old clay and ductile iron pipe.

Start Year

2032

Total Project Cost

$1.01M

Weighted Risk Score

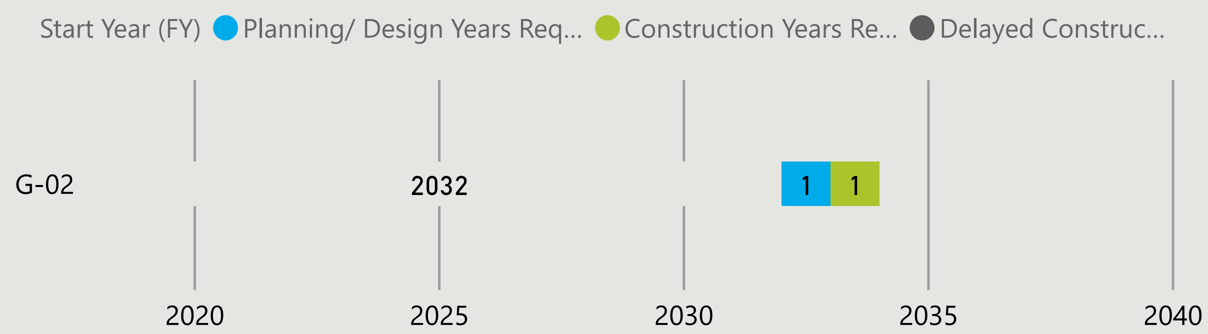
7.9

$7,700
Pipeline Easement Cost

$167,000
Engineering Cost

$693,600
Construction Cost

$139,000
Scope Contingency

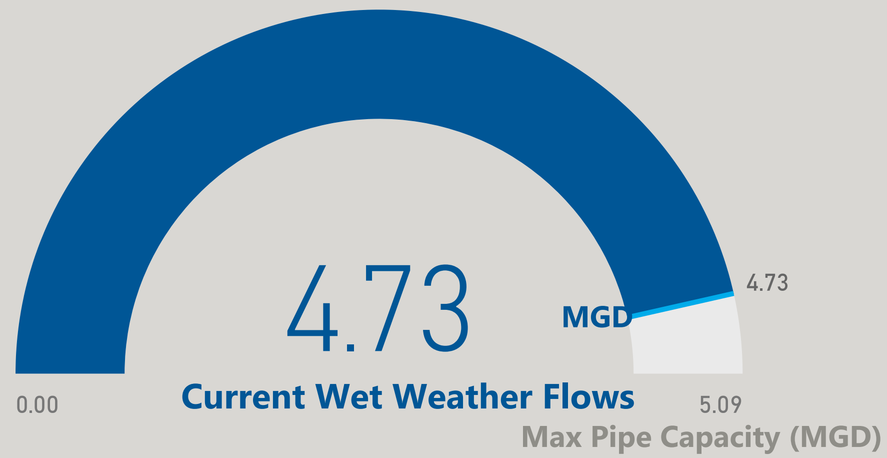


Capacity Tracking

Master Plan Trigger

BASE 10YR SSO

	Flow (MGD)
Available Capacity	0.36
Development Flow	0.00
Existing Pipe Capacity	5.09
Existing 2 YR Storm Flow	4.73

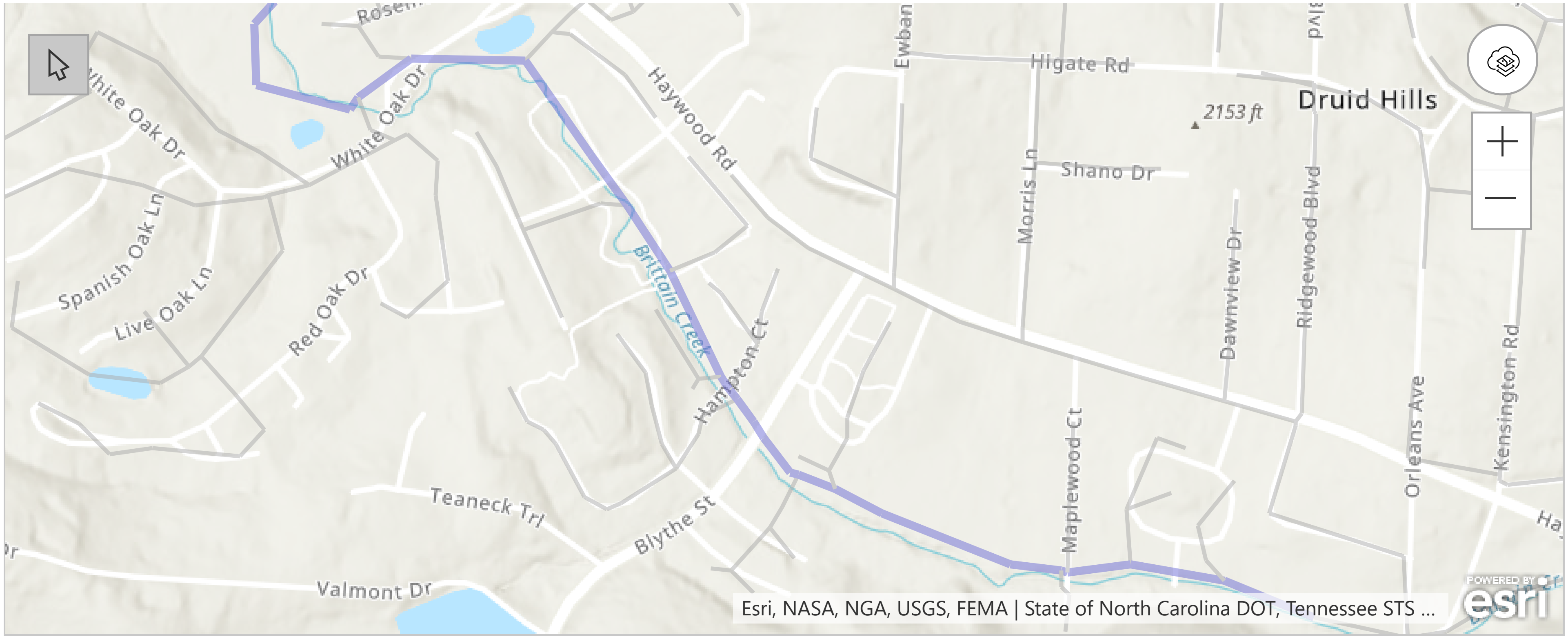


Future Improvement

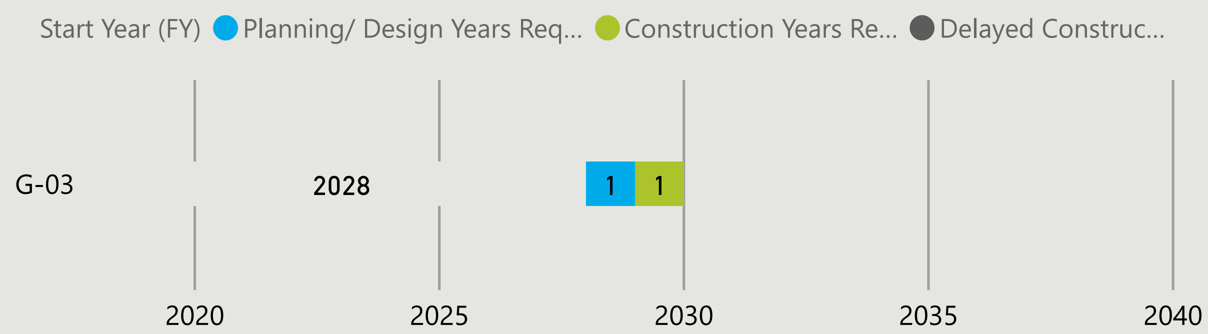
Project Name

G-03

Brittain Creek Sewer Replacement near Haywood Rd



Start Year	Total Project Cost	
2028	**$2.5M**	$20,200 Pipeline Easement Cost
	Weighted Risk Score	$414,000 Engineering Cost
	7.9	$1,724,000 Construction Cost
		$345,000 Scope Contingency



Pipe Details	Pipe #1
Downstream Manhole	1476
Upstream Manhole	2699
Diameter	15
Length	4480

Scope:

Approximately 4,480 feet of gravity sewer will replace the existing Brittain Creek Outfall. The gravity sewer will follow the existing alignment along Brittain Creek. This project will include 4,480 feet of 15-inch sewer. This project will have five road crossings: Maplewood Ct, Blythe St, Hampton Ct, Haywood Townes Dr, and White Oaks Dr.

Project Objective and Benefit:

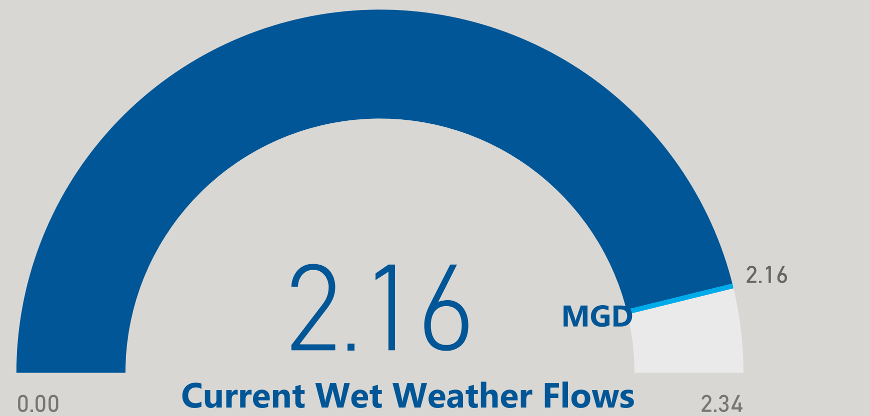
This improvement will relieve surcharging and potential overflows along Brittain Creek. This improvement is driven by wet weather flows. The model predicts SSOs during a 2-year storm starting in 2040 and during a 10-year storm as early as the base year. The LOF risk ranged from 1-4 because of the 40 year old ductile iron and pvc pipe. The improvement will reduce the LOF by eliminating the capacity constraints.

Capacity Tracking

Master Plan Trigger

BASE 10YR SSO or 2040 2 yr

	Flow (MGD)
Available Capacity	0.18
Development Flow	0.00
Existing Pipe Capacity	2.34
Existing 2 YR Storm Flow	2.16



Current Wet Weather Flows

Max Pipe Capacity (MGD)

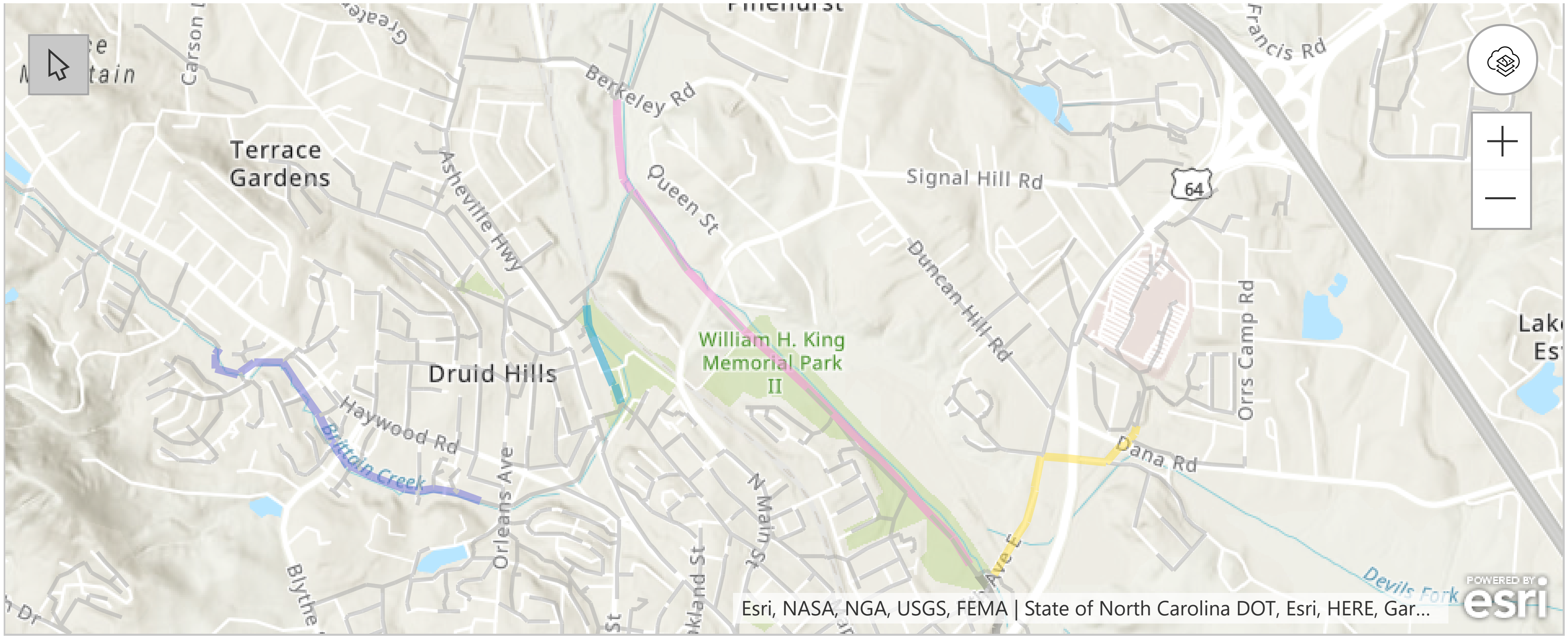
Future Improvement

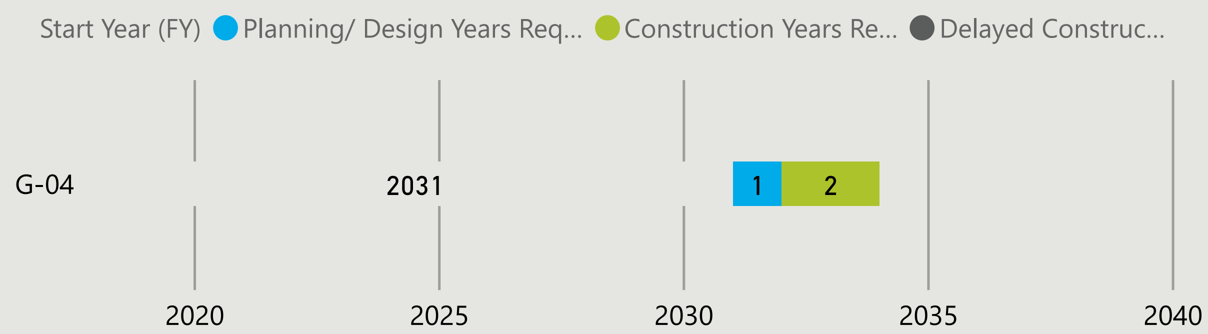
Project Name

G-04

Mud Creek Parallel Replacement

Start Year	Total Project Cost	
2031	**$7.51M**	$33,700 Pipeline Easement Cost
	Weighted Risk Score	$1,246,000 Engineering Cost
	7.5	$5,193,000 Construction Cost
		$1,039,000 Scope Contingency





Pipe Details	Pipe #1	Pipe #2
Downstream Manhole	2345	207
Upstream Manhole	5319	2343
Diameter	36	54
Length	6310	1180

Scope:

Approximately 7,490 feet of gravity sewer will replace the existing Mud Creek outfall. The sewer will follow the alignment of the existing sewer along Mud Creek. The project will include 6,310 feet of 36-inch and 1,180 feet of 54-inch gravity sewer.

Project Objective and Benefit:

This improvement will relieve surcharging and potential overflows along Mud Creek. This improvement is driven by future development and wet weather flows. The model predicts SSOs during a 2-year storm starting in 2040 and during a 10-year storm as early as 2025. The LOF risk, which ranged from 1.5-3.5 because of the 44 year old clay pipe.

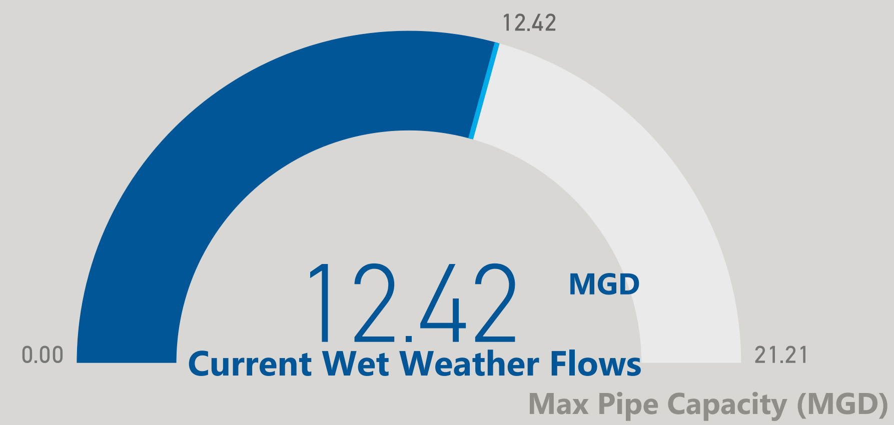
Hendersonville Water Sewer

Capacity Tracking

Master Plan Trigger

2025 10YR SSO or 2040 2 yr

	Flow (MGD)
Available Capacity	8.79
Development Flow	0.00
Existing Pipe Capacity	21.21
Existing 2 YR Storm Flow	12.42

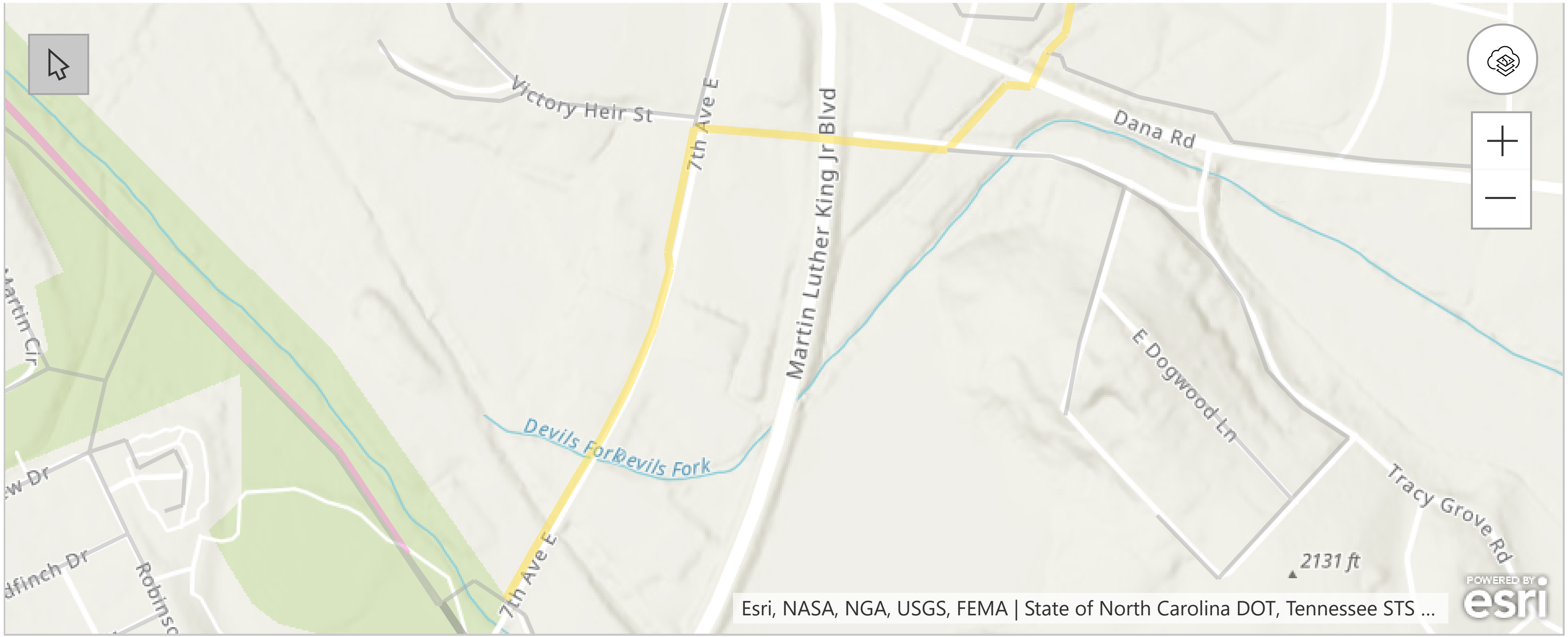


Future Improvement

Project Name

G-05

Devils Fork Sewer Replacement near MLK Jr Blvd



Pipe Details	Pipe #1
Downstream Manhole	5317
Upstream Manhole	772
Diameter	21
Length	3070

Scope:

Approximately 3,100 feet of 21-inch gravity sewer will replace the existing Devils Fork outfall. The alignment will follow the existing sewer along 7th Ave. This project has one stream crossing. This project will require three road crossings: Dana Rd, Tracy Grove Rd, and 7th Ave.

Project Objective and Benefit:

This improvement will relieve surcharging and potential overflows along the Devils Fork Outfall. This improvement is driven by future development. The model predicts SSOs during a 2-year storm starting in 2040. The improvement will reduce the LOF risk, which ranged from 2-3 because of the 40 year old clay pipe.

Hendersonville Water Sewer

Start Year

2027

Total Project Cost

$2.39M

Weighted Risk Score

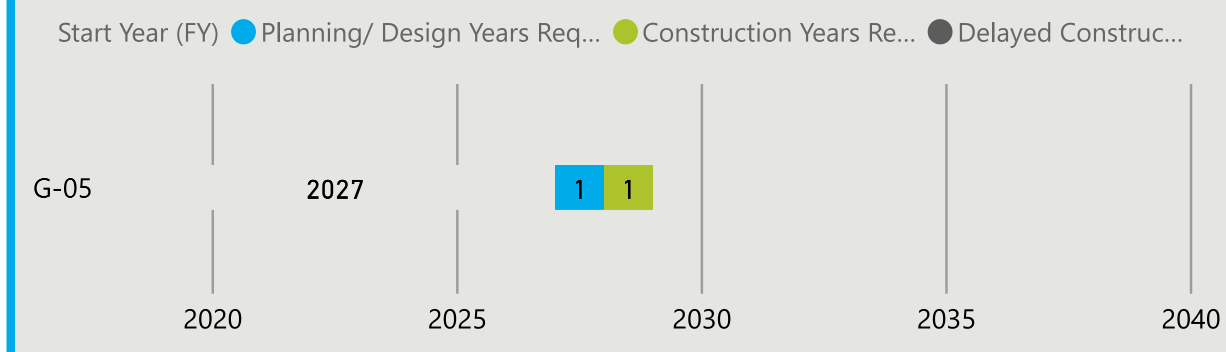
8.4

$13,800
Pipeline Easement Cost

$396,000
Engineering Cost

$1,650,900
Construction Cost

$330,000
Scope Contingency

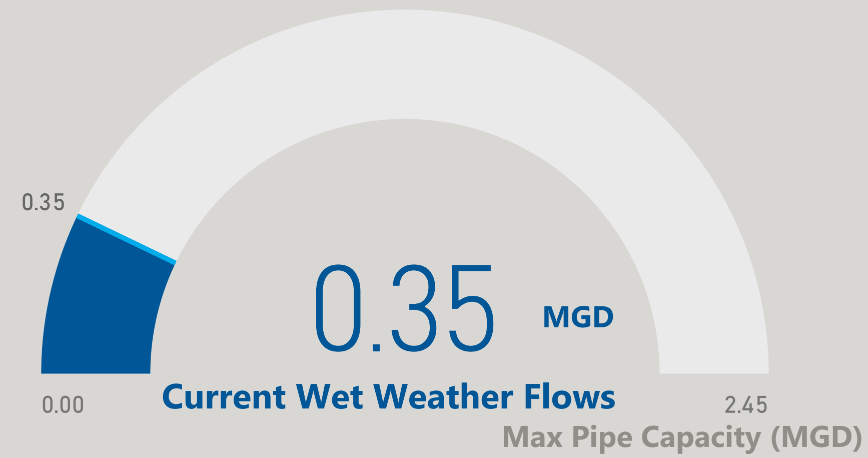


Capacity Tracking

Master Plan Trigger

2040 2YR SSO

	Flow (MGD)
Available Capacity	2.10
Development Flow	0.00
Existing Pipe Capacity	2.45
Existing 2 YR Storm Flow	0.35

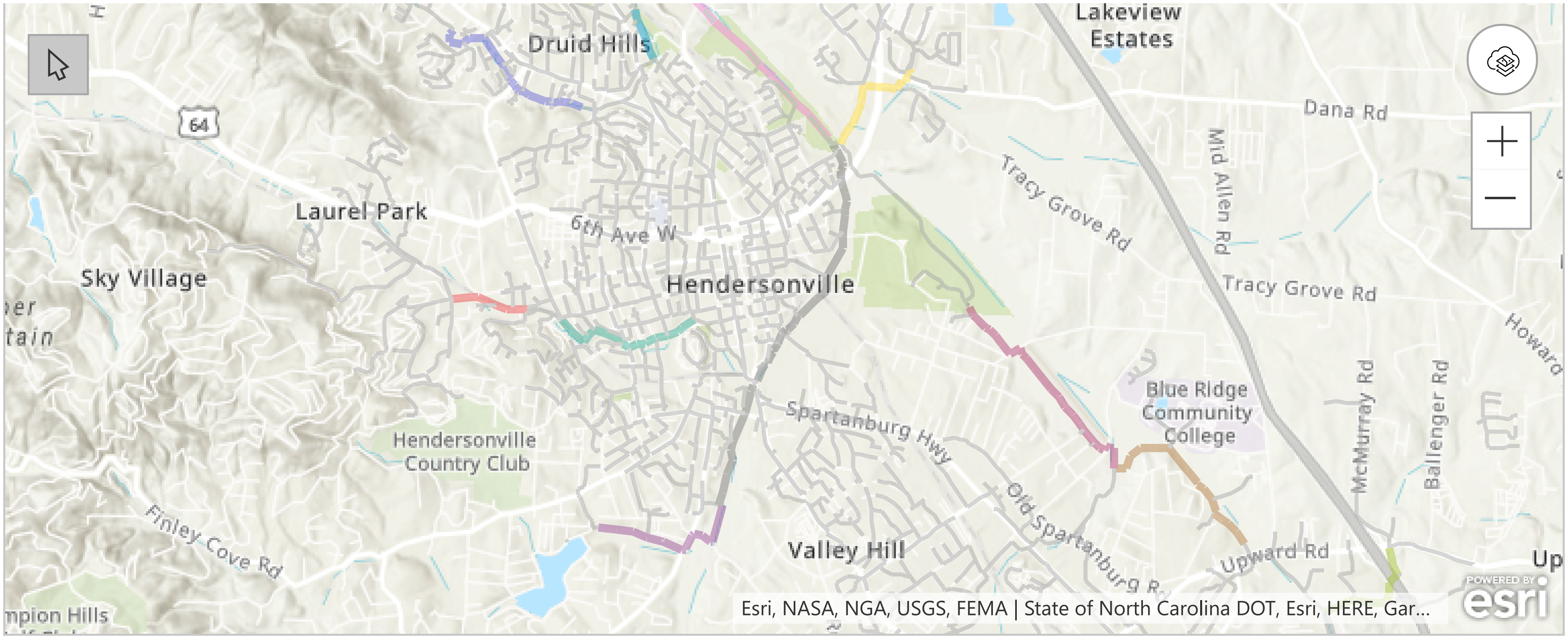


Future Improvement

Project Name

G-06

Replacement sewer along Mud Creek near Railroad



Start Year
2019

Total Project Cost

$9.23M

Weighted Risk Score

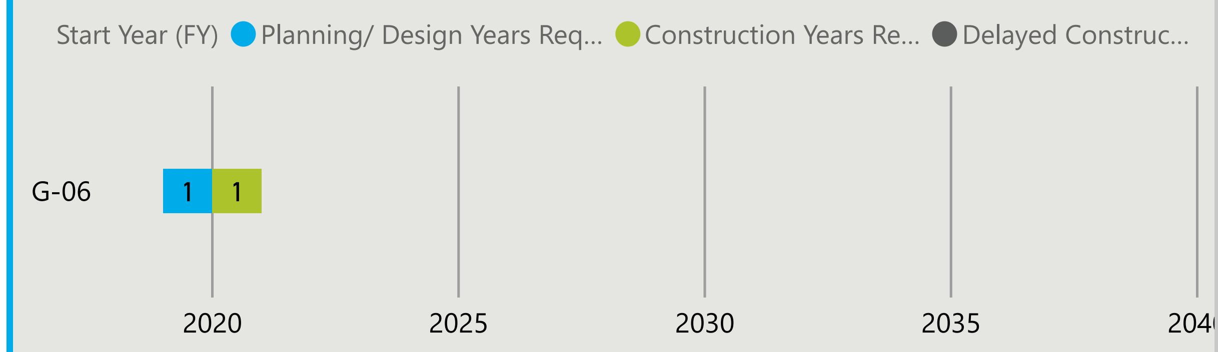
12.0

$44,900
Pipeline Easement Cost

$1,530,000
Engineering Cost

$6,375,600
Construction Cost

$1,275,000
Scope Contingency



Pipe Details	Pipe #1	Pipe #2
Downstream Manhole	5319	1770
Upstream Manhole	443	524
Diameter	21	36
Length	3150	6820

Hendersonville Water Sewer

Scope:

Approximately 9,960 feet of gravity sewer will parallel or replace the existing Mud Creek interceptor. The sewer will follow the alignment of the existing sewer along Mud Creek, however, the alignment should be evaluated to avoid the railroad crossing near South Kind St. The project will include 3,150 feet of 21-inch and 6,820 feet of 36-inch. The project has several stream crossings, three potential rail road crossings and six road crossings: White St., S. Main St., S. Grove St. 4th Ave., and Highway 64. The project should be evaluated to reroute the alignment through the existing 36 inch jack and bore previously performed for the Jackson Park sewer line to avoid crossing 64. This project is a critical project and should be started immediately.

Project Objective and Benefit:

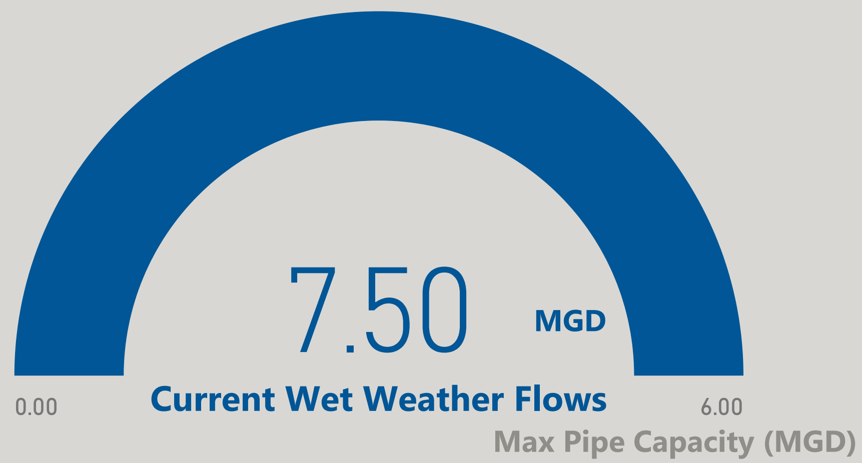
This improvement will relieve surcharging and potential overflows on the Mud Creek Outfall. The model predicts SSOs during a 2-year storm starting in the base year. The improvement will reduce the LOF risk, which ranged from 2-4.5 because of the 90 year old clay pipe, and capacity constraints.

Capacity Tracking

Master Plan Trigger

BASE 2YR SSO

	Flow (MGD)
Available Capacity	0.00
Development Flow	0.00
Existing Pipe Capacity	6.00
Existing 2 YR Storm Flow	7.50

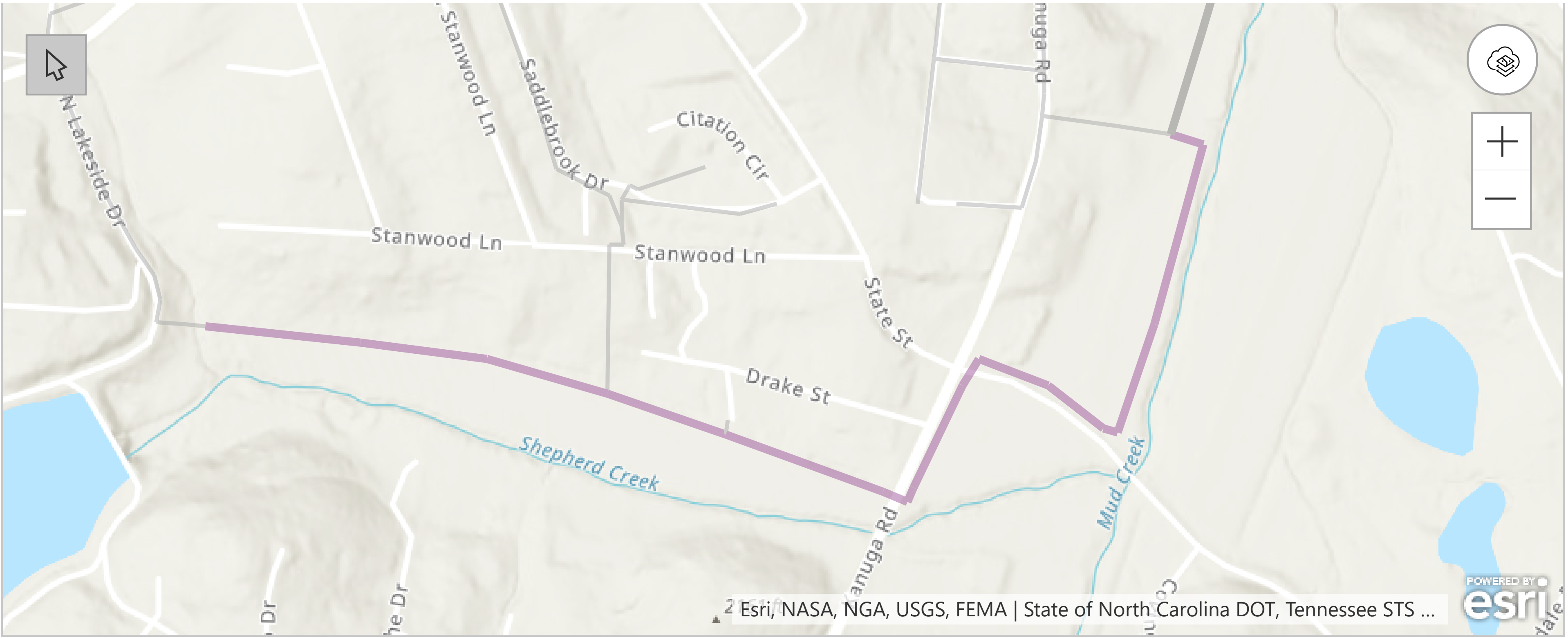


Critical Improvement

Project Name

G-07

Shepherd Creek Replacement Sewer near Kanuga Rd



Pipe Details	Pipe #1
Downstream Manhole	524
Upstream Manhole	5242
Diameter	18
Length	4320

Scope:

Approximately 4,320 feet of gravity sewer will parallel or replace the existing Shepherd Creek outfall. The sewer will follow the alignment of the existing sewer along Shepherd Creek. The project will include 4,320 feet of 18-inch. This project will include two road crossings: Erkwood Dr., and Kanuga Rd. This project is planned for 2025.

Project Objective and Benefit:

This improvement will relieve surcharging and potential overflows on the Shepherd Creek Outfall upstream of project G-06. The model predicts SSOs during a 2-year storm starting in 2025. The LOF risk ranged from 1-3, and the pipe material consists of 3 year old DIP.

Hendersonville Water Sewer

Start Year

2035

Total Project Cost

$2.28M

Weighted Risk Score

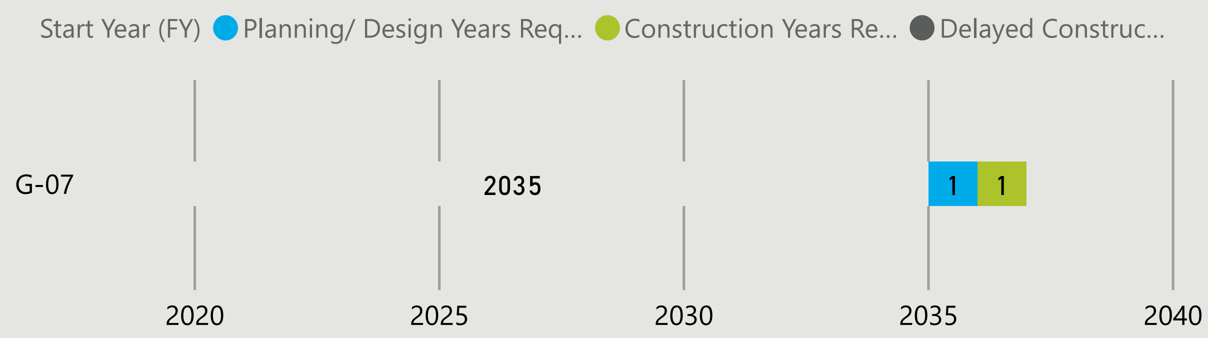
4.9

$19,400
Pipeline Easement Cost

$376,000
Engineering Cost

$1,567,900
Construction Cost

$314,000
Scope Contingency

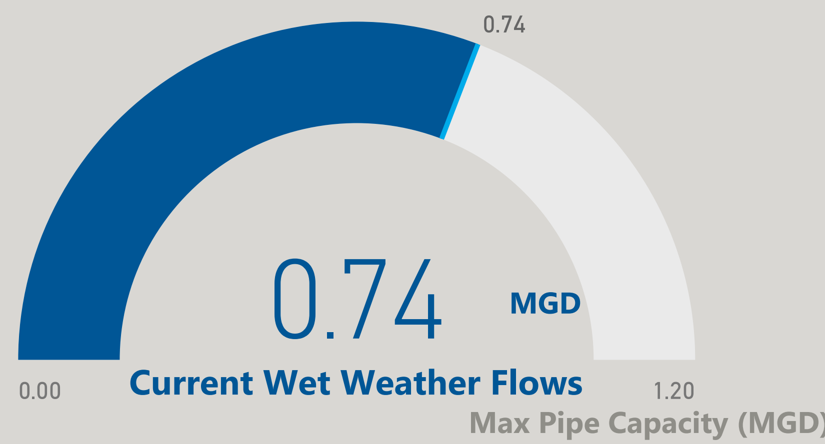


Capacity Tracking

Master Plan Trigger

2040 2YR SSO

	Flow (MGD)
Available Capacity	0.46
Development Flow	0.00
Existing Pipe Capacity	1.20
Existing 2 YR Storm Flow	0.74

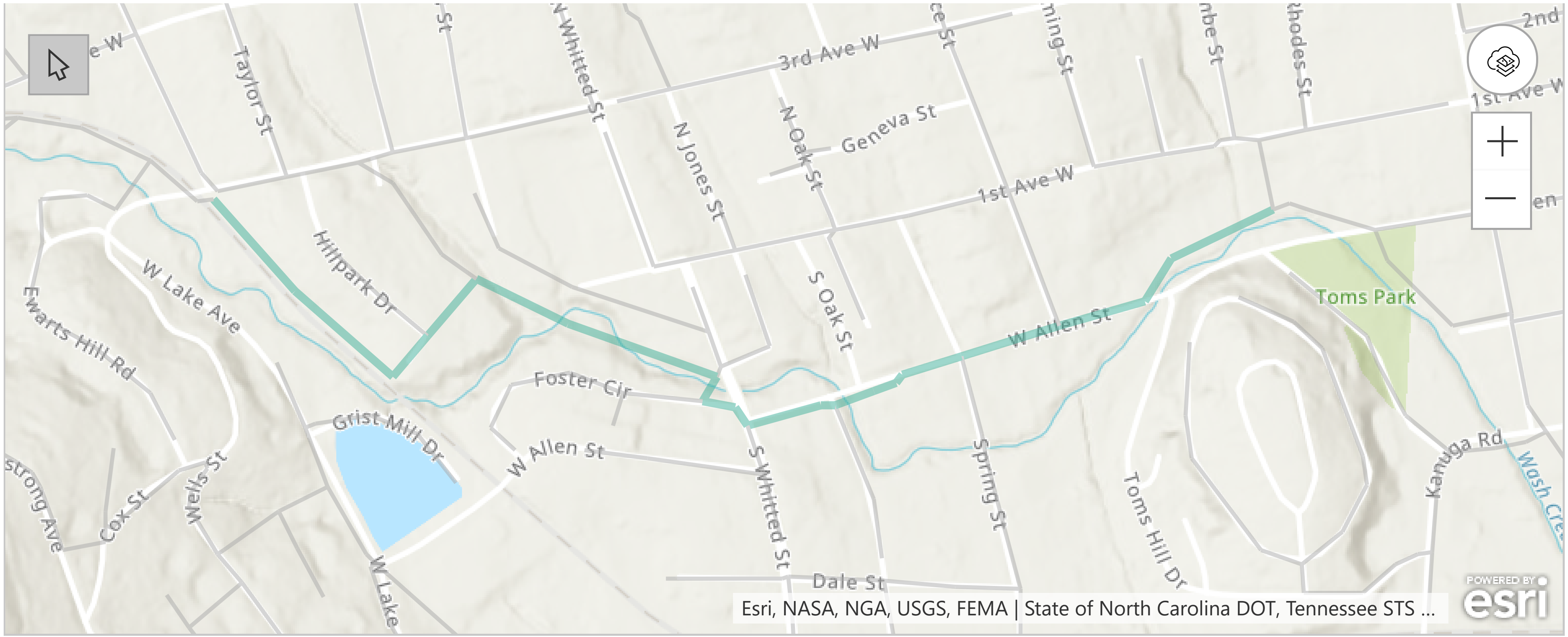


Future Improvement

Project Name

G-08

Wash Creek Replacement Sewer



Pipe Details	Pipe #1
Downstream Manhole	1611
Upstream Manhole	4941
Diameter	21
Length	4150

Scope:

Approximately 4,150 feet of 21-inch gravity sewer will replace the existing Wash Creek outfall. The sewer will follow the exiting alignment along Wash Creek. This project will include three road crossings: Kanuga Rd, W. Barwell St, and S. Washington St.

Project Objective and Benefit:

This improvement will relieve surcharging and potential overflows along Wash Creek. This improvement is driven by wet weather flows. The model predicts SSOs during a 2-year storm starting in 2040 and during a 10-year storm as early as the base year. The improvement will reduce the LOF risk, which ranged from 2-4 because of the 92 year old clay pipe, by removing capacity constraints and replacing the clay pipe.

Hendersonville Water Sewer

Start Year

2021

Total Project Cost

$3.27M

Weighted Risk Score

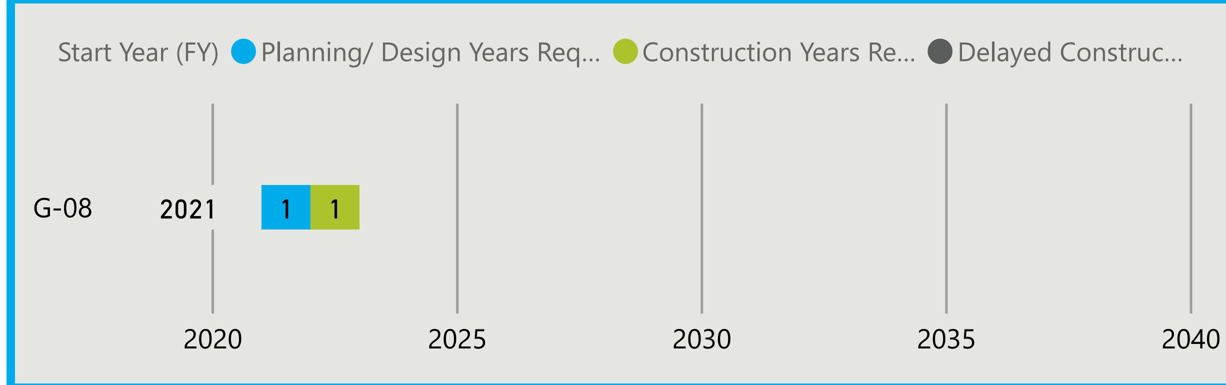
12.2

$18,700
Pipeline Easement Cost

$541,000
Engineering Cost

$2,255,600
Construction Cost

$451,000
Scope Contingency

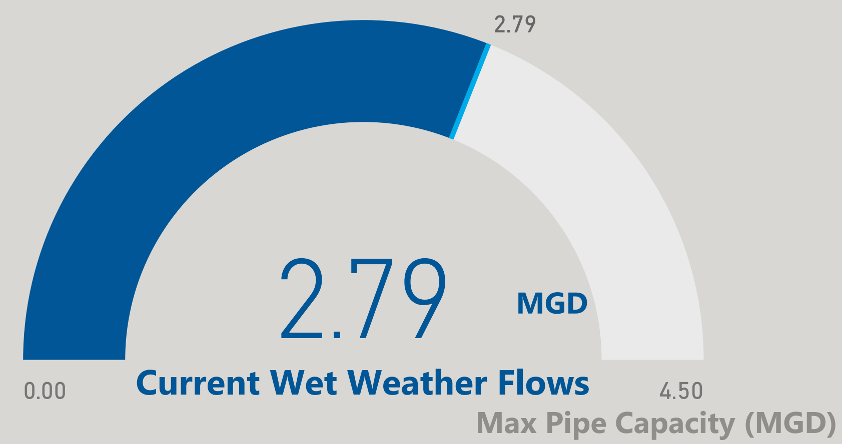


Capacity Tracking

Master Plan Trigger

BASE 10YR SSO

	Flow (MGD)
Available Capacity	1.71
Development Flow	0.00
Existing Pipe Capacity	4.50
Existing 2 YR Storm Flow	2.79



Future Improvement

Project Name

G-09

Wash Creek Replacement Sewer near Wash Creek Dr

Pipe Details	Pipe #1
Downstream Manhole	1736
Upstream Manhole	1734
Diameter	15
Length	1950

Scope:

Approximately 1,950 feet of 15-inch gravity sewer will replace the existing Wash Creek outfall. The sewer will follow the existing alignment along Wash Creek. The sewer crosses Wash Creek once. This project will include one road crossing on Wash Creek Dr.

Project Objective and Benefit:

This improvement will relieve surcharging and potential overflows along Wash Creek. This improvement is driven by future development and wet weather flows. The model predicts SSOs during a 2-year storm starting in 2040 and during a 10-year storm as early as 2025.The improvement will reduce the LOF risk, which ranged from 2-3 because of the 92 year old clay pipe, by removing the capacity constraints and replacing the clay pipes.

Hendersonville Water Sewer

Start Year

2029

Total Project Cost

$1.27M

Weighted Risk Score

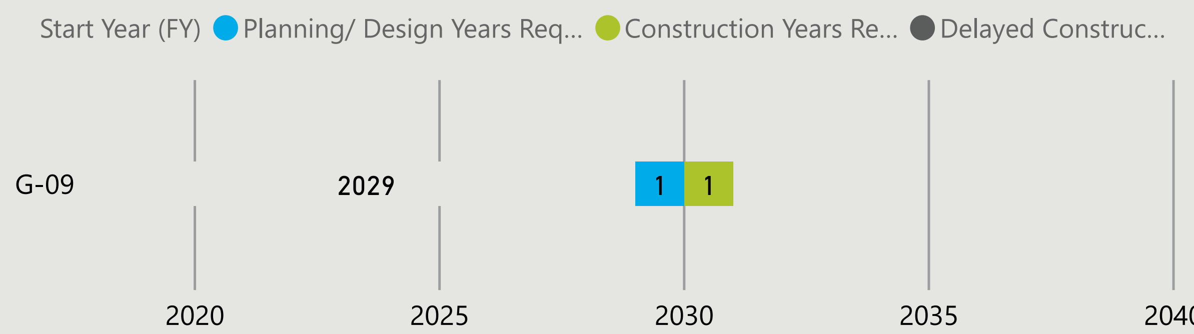
11.5

$8,800
Pipeline Easement Cost

$210,000
Engineering Cost

$873,400
Construction Cost

$175,000
Scope Contingency

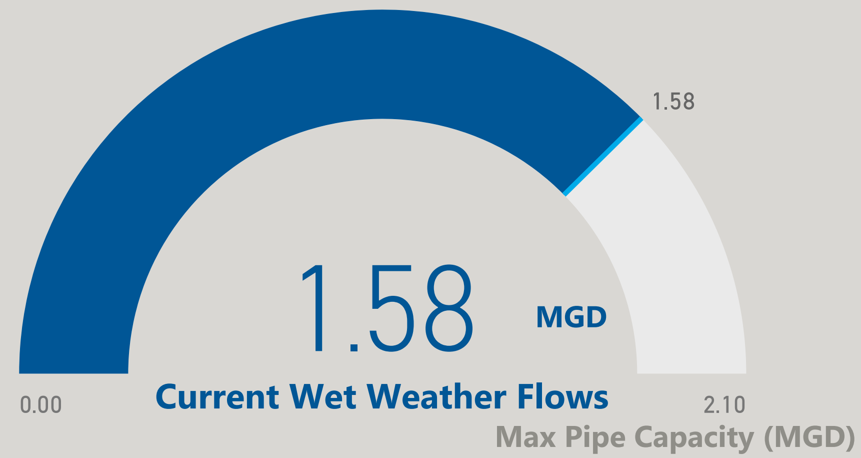


Capacity Tracking

Master Plan Trigger

2025 10YR SSO

	Flow (MGD)
Available Capacity	0.52
Development Flow	0.00
Existing Pipe Capacity	2.10
Existing 2 YR Storm Flow	1.58

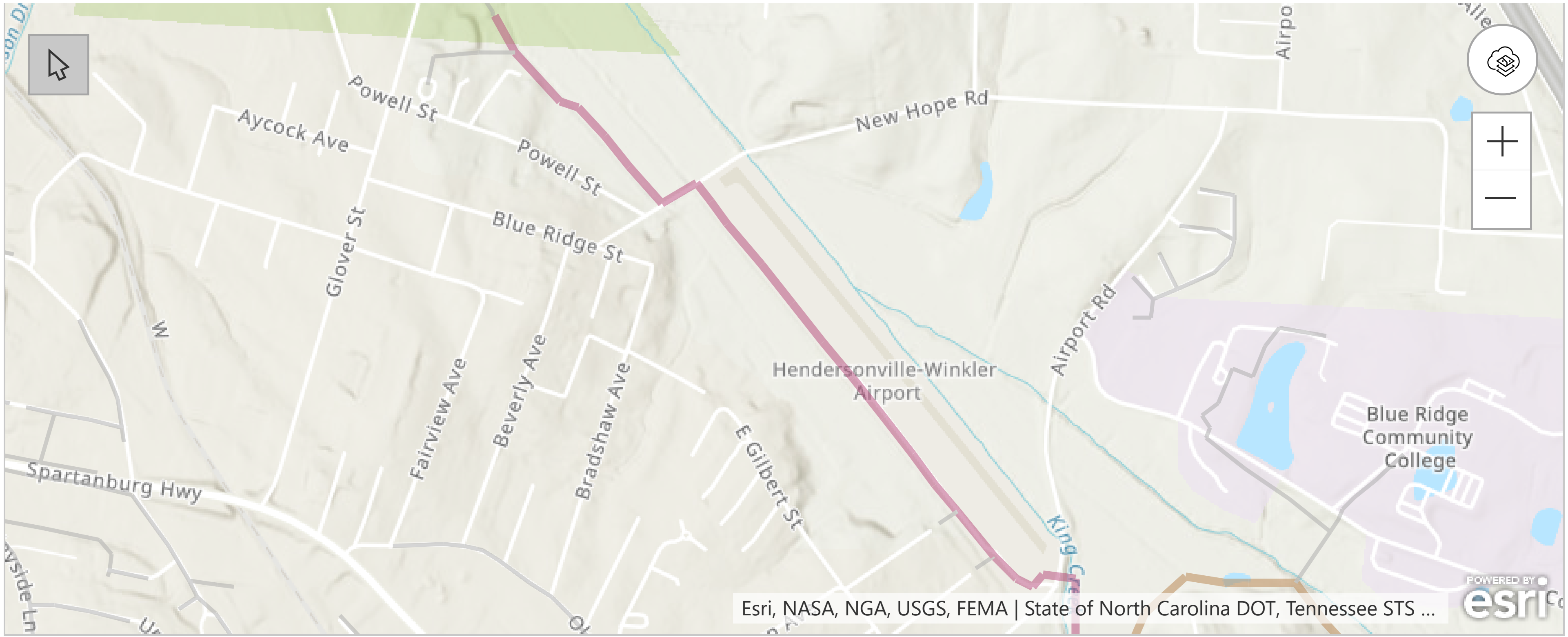


Future Improvement

Project Name

G-10

King Creek Replacement Sewer near Airport



Pipe Details	Pipe #1
Downstream Manhole	5297
Upstream Manhole	506
Diameter	30
Length	5970

Scope:

Approximately 5,970 feet of gravity sewer will replace the existing Bat Fork outfall. The sewer will follow the existing alignment along Bat Fork. The new sewer will cross Kings Creek 1 time. The project will include 5,970 feet of 30-inch gravity sewer. The project will have 3 road Crossings: Airport Rd, Grandeur Ln, and New Hope Rd.

Project Objective and Benefit:

This improvement will relieve surcharging and potential overflows along the King Creek Outfall. This improvement is driven by future development and wet weather flows. The model predicts SSOs during a 2-year storm starting in 2040 and during a 10-year storm as early as 2025. The LOF risk ranged from 1-2.5 because of the 27 year old DIP.

Start Year

2039

Total Project Cost

$4.9M

Weighted Risk Score

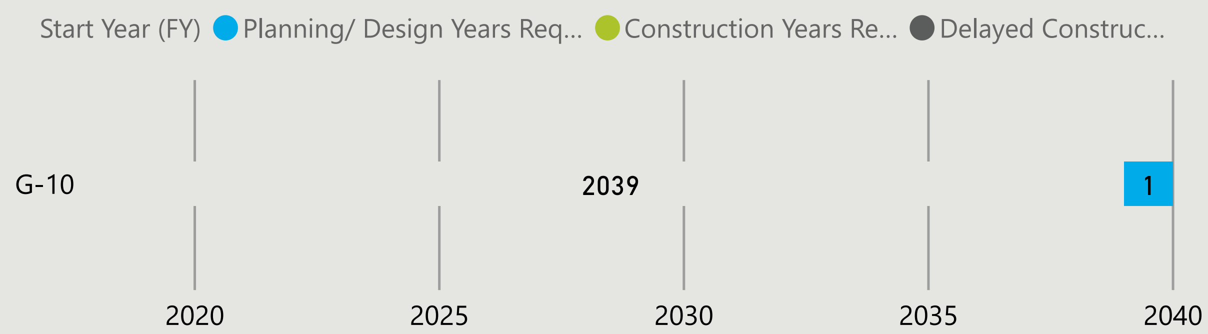
3.6

$26,900
Pipeline Easement Cost

$812,000
Engineering Cost

$3,384,800
Construction Cost

$677,000
Scope Contingency

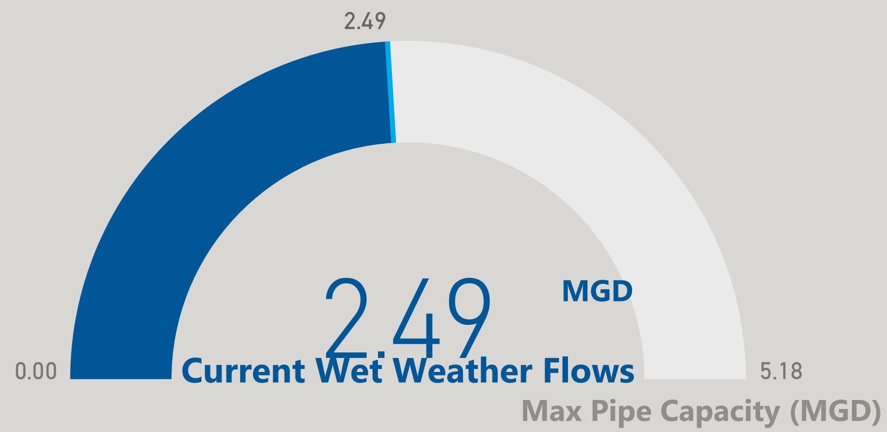


Capacity Tracking

Master Plan Trigger

2040 2YR SSO

	Flow (MGD)
Available Capacity	2.69
Development Flow	0.00
Existing Pipe Capacity	5.18
Existing 2 YR Storm Flow	2.49

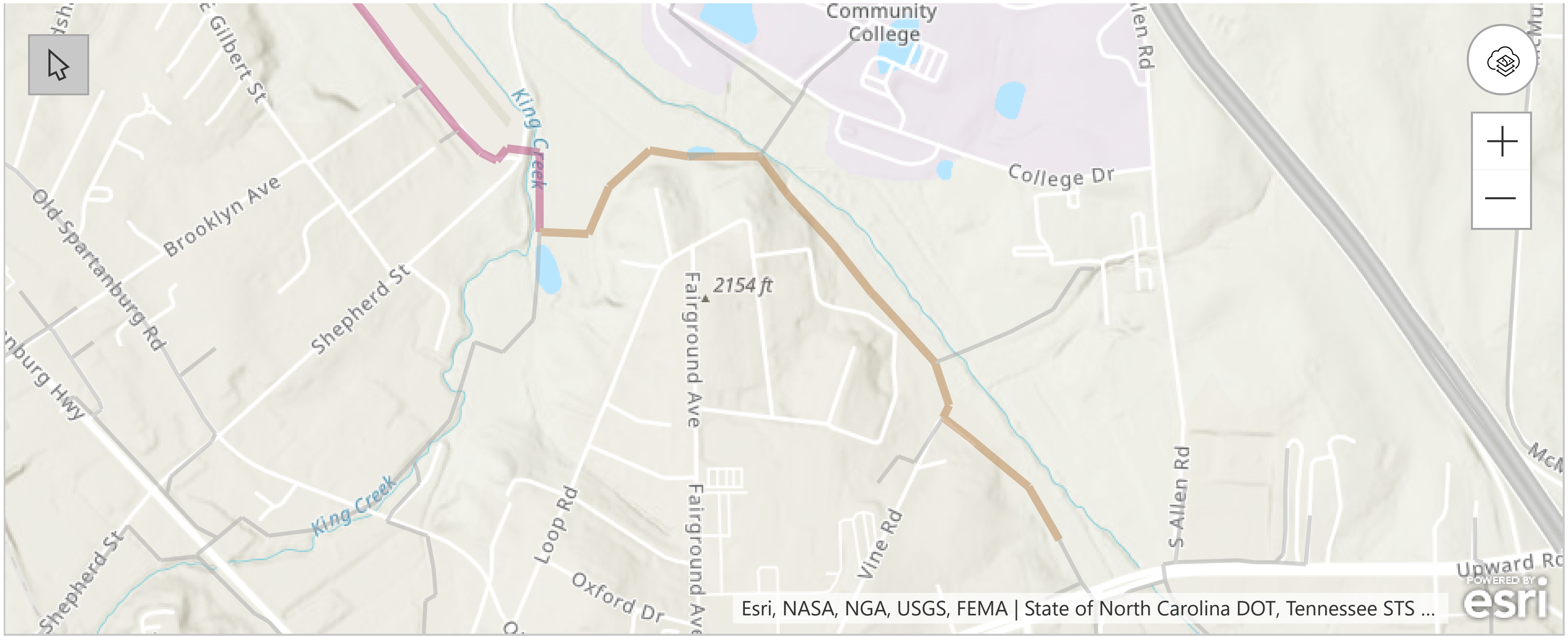


Future Improvement

Project Name

G-11

Bat Fork Replacement Sewer near Blue Ridge Community College



Pipe Details	Pipe #1
Downstream Manhole	506
Upstream Manhole	576
Diameter	24
Length	4810

Scope:

Approximately 4,810 feet of 24-inch gravity sewer will replace the existing Bat Fork outfall.

Project Objective and Benefit:

This improvement will relieve surcharging and potential overflows along the Bat Fork Outfall. This improvement is driven by future development. The model predicts SSOs during a 2-year storm starting in 2040. The LOF risk ranged from 1-2 because of the 27 year old DIP.

Hendersonville Water Sewer

Start Year

2040

Total Project Cost

$3.16M

Weighted Risk Score

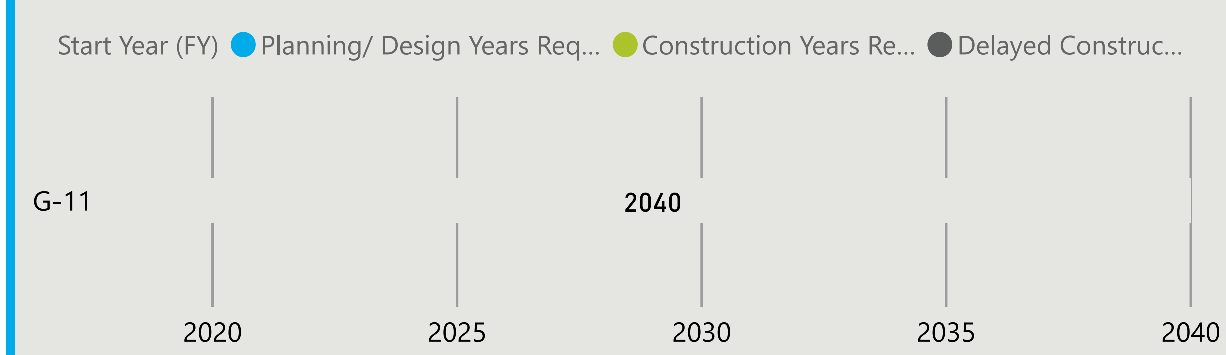
2.4

$21,600
Pipeline Easement Cost

$523,000
Engineering Cost

$2,178,200
Construction Cost

$436,000
Scope Contingency

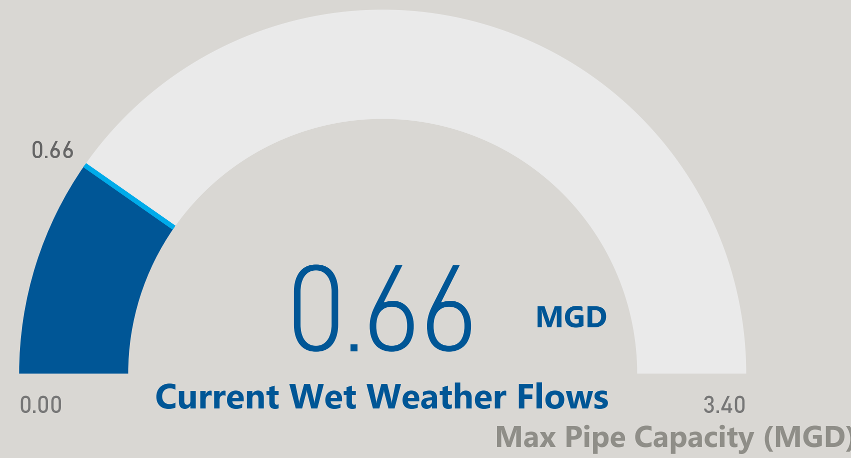


Capacity Tracking

Master Plan Trigger

2040 2YR SSO

	Flow (MGD)
Available Capacity	2.74
Development Flow	0.00
Existing Pipe Capacity	3.40
Existing 2 YR Storm Flow	0.66



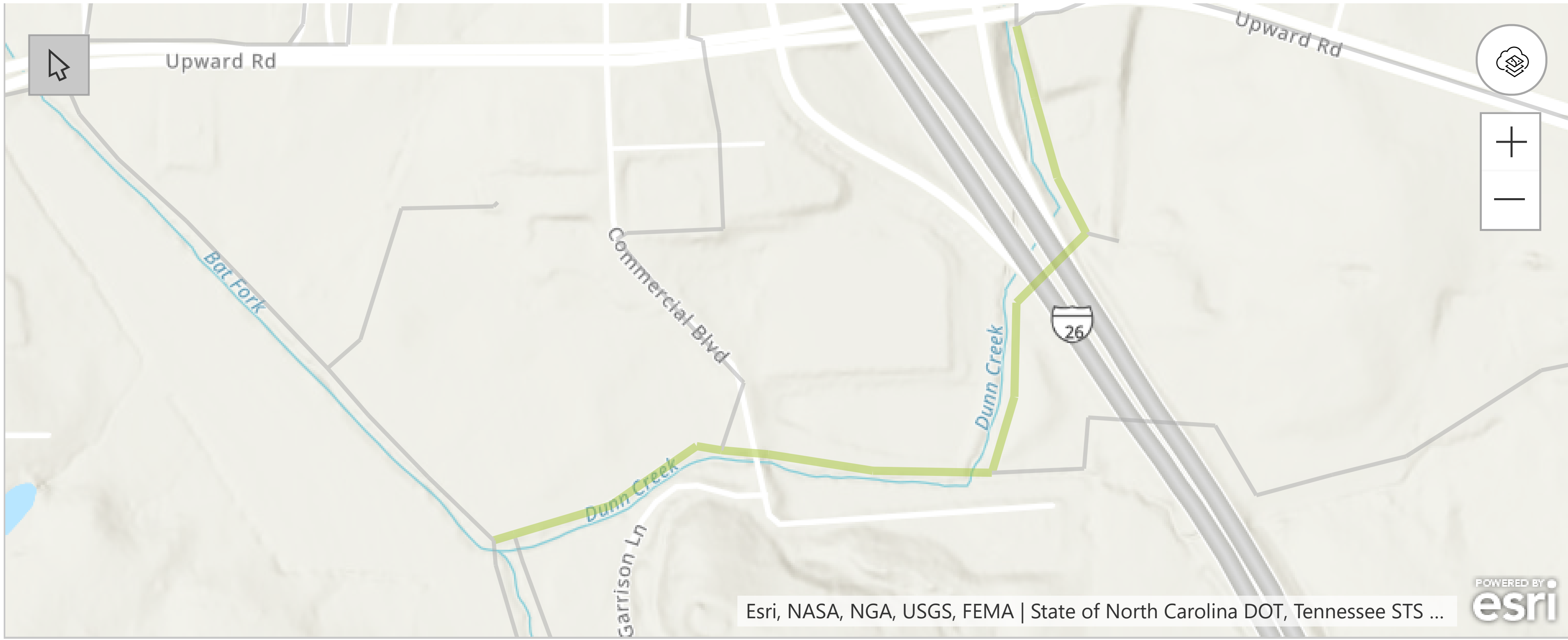
Future Improvement

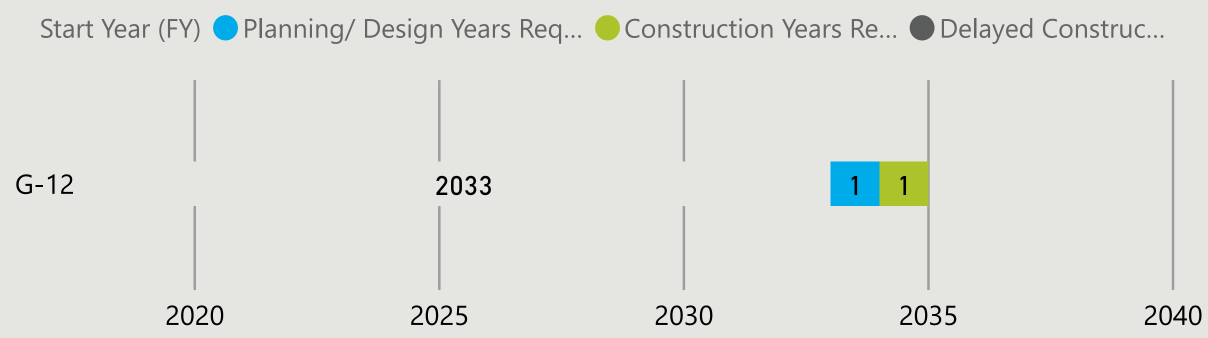
Project Name

G-12

Dunn Creek Replacement Sewer near I-26

Start Year	Total Project Cost	
2033	**$2.06M**	$14,300 Pipeline Easement Cost
	Weighted Risk Score	$341,000 Engineering Cost
	5.7	$1,421,100 Construction Cost
		$284,000 Scope Contingency





Pipe Details	Pipe #1	Pipe #2
Downstream Manhole	628	623
Upstream Manhole	519	628
Diameter	15	18
Length	1530	1640

Scope:

Approximately 3,170 feet of gravity sewer will replace the existing Dunn Creek outfall. The project will include 1,530 feet of 15-inch and 1,640 feet of 18-inch gravity sewer. The new sewer will cross Dunn Creek once. The project will have two road crossings: I-26 and Commercial Blvd.

Project Objective and Benefit:

This improvement will relieve surcharging and potential overflows along Dunn Creek Outfall. This improvement is driven by future development and wet weather flows. The model predicts SSOs during a 2-year storm starting in 2040 and during a 10-year storm as early as 2025. The LOF risk ranged from 1-2 because of the 25 year old ductile iron and pvc pipe.

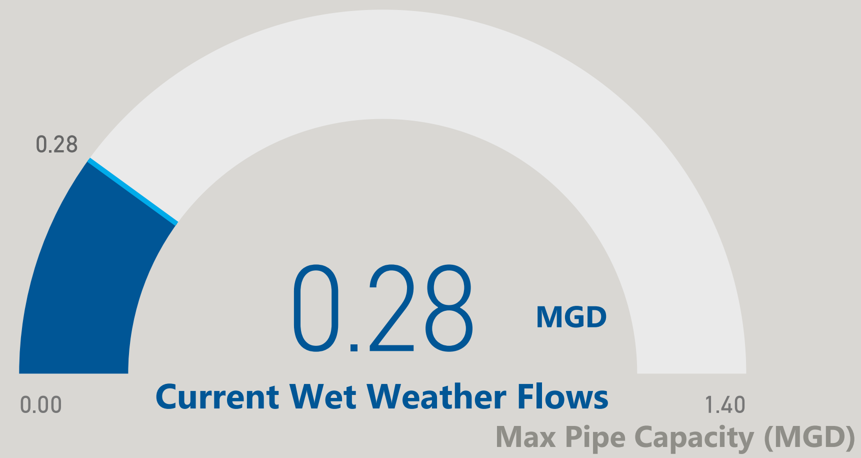
Hendersonville Water Sewer

Capacity Tracking

Master Plan Trigger

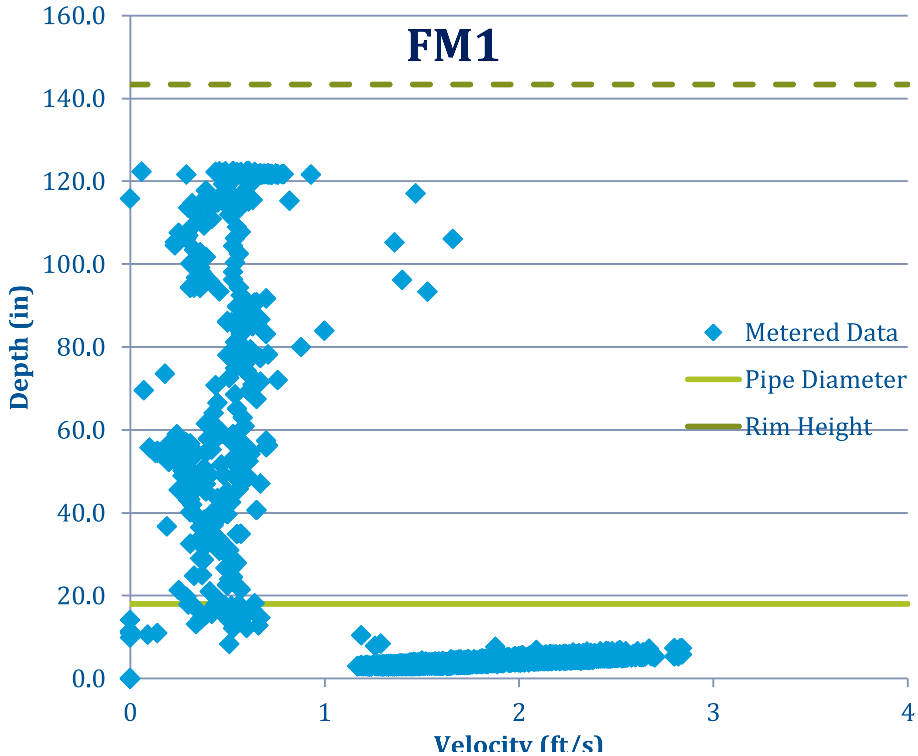
2025 10YR SSO

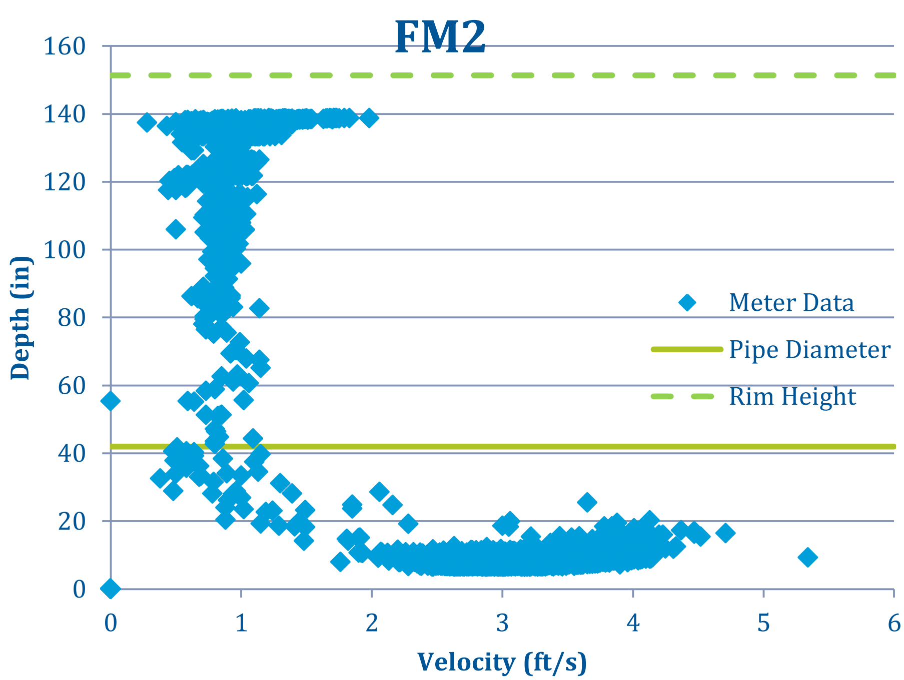
	Flow (MGD)
Available Capacity	1.12
Development Flow	0.00
Existing Pipe Capacity	1.40
Existing 2 YR Storm Flow	0.28

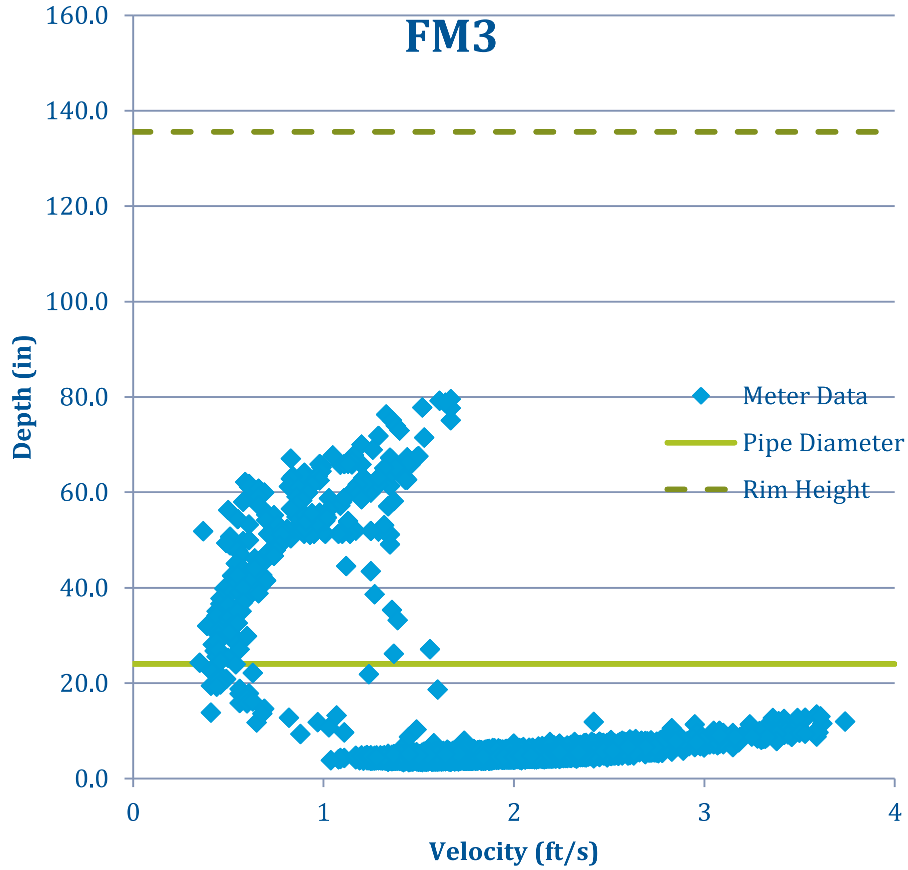


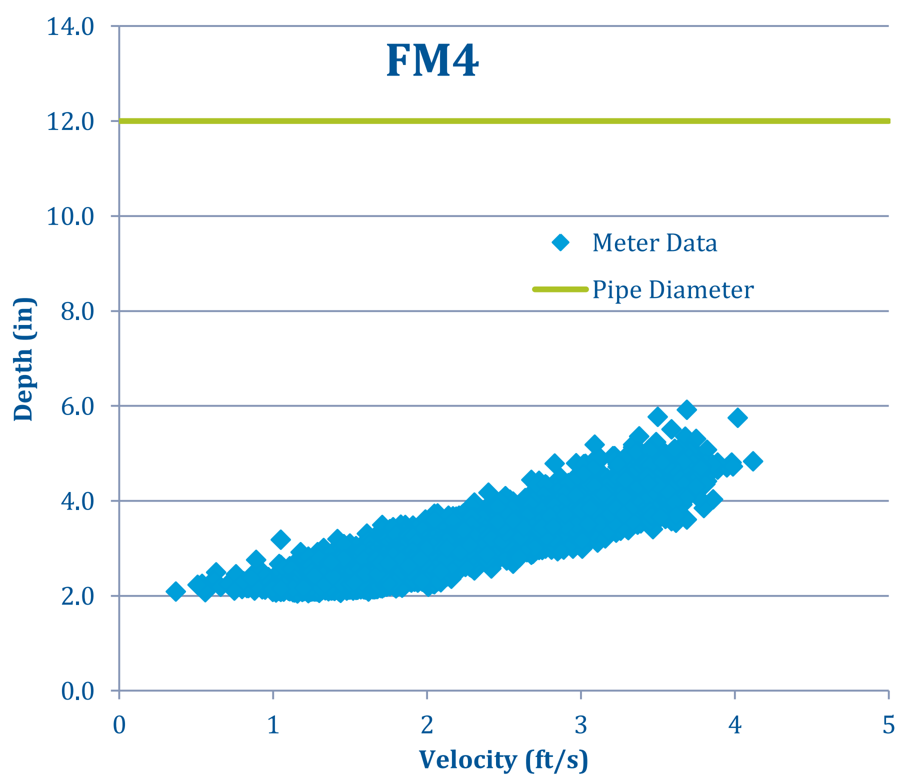
Future Improvement

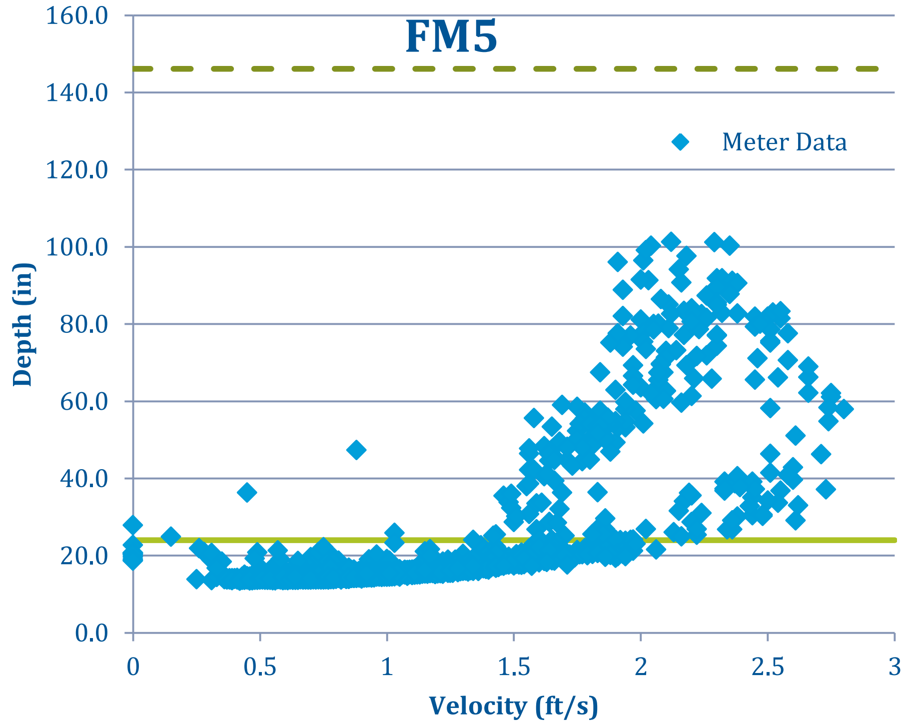
Appendix A: Scatter Plots

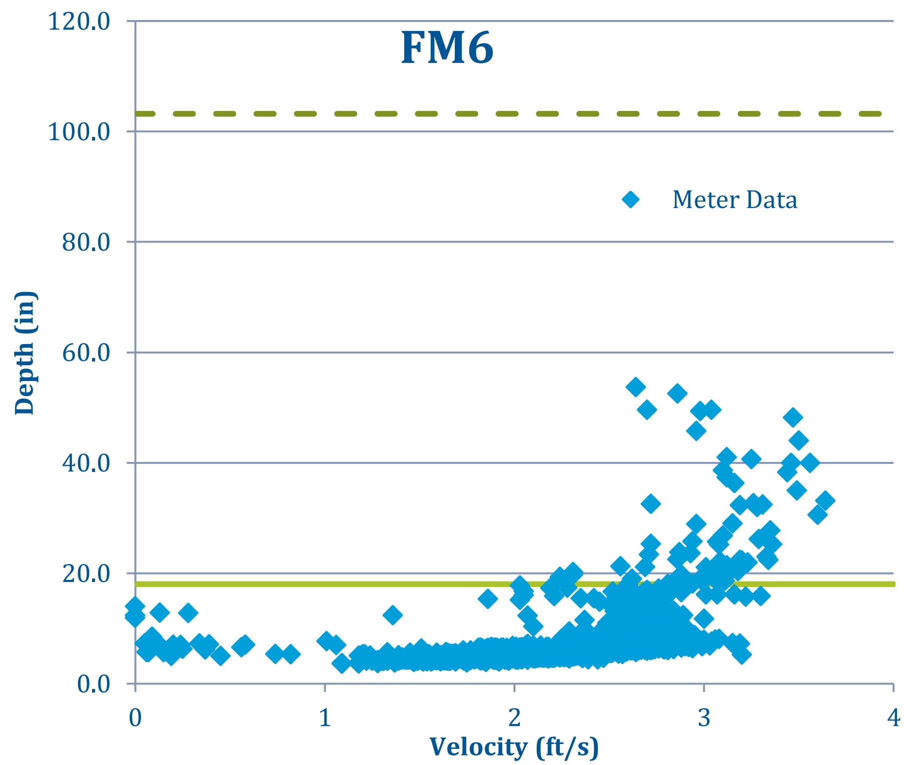


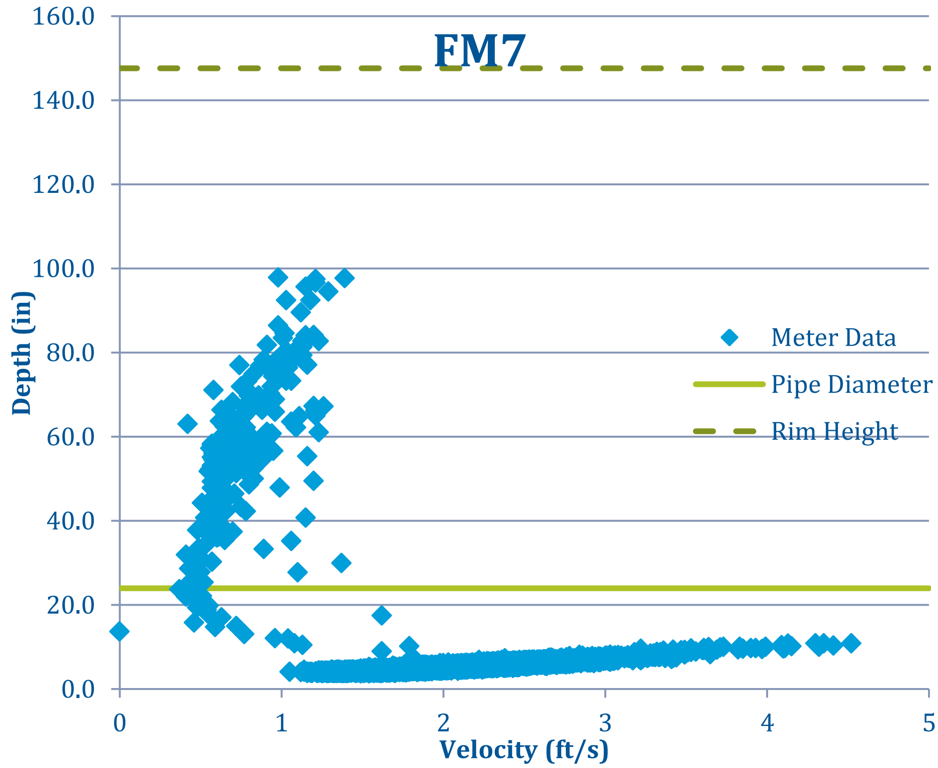


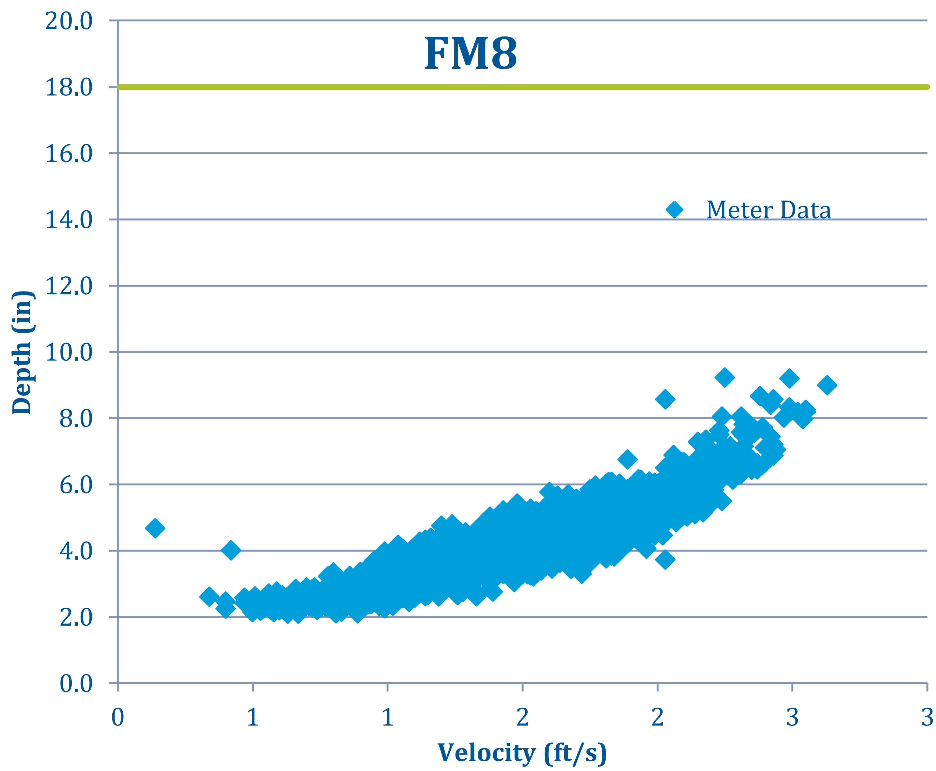

FM3
160.0
140.0
120.0
100.0
80.0
60.0
40.0
20.0
0.0
Depth (in)
0
1
2
3
4
Velocity (ft/s)
Meter Data
Pipe Diameter
Rim Height


FM4
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
Depth (in)
0
1
2
3
4
5
Velocity (ft/s)
Meter Data
Pipe Diameter


FM5
Meter Data
Depth (in)
160.0
140.0
120.0
100.0
80.0
60.0
40.0
20.0
0.0
0
0.5
1
1.5
2
2.5
3
Velocity (ft/s)

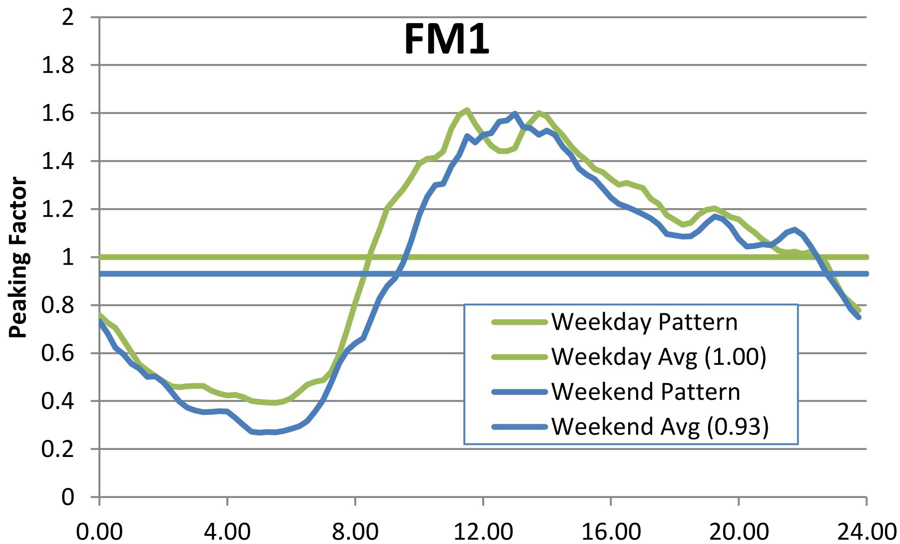

FM6
Meter Data
Depth (in)
120.0
100.0
80.0
60.0
40.0
20.0
0.0
0
1
2
3
4
Velocity (ft/s)

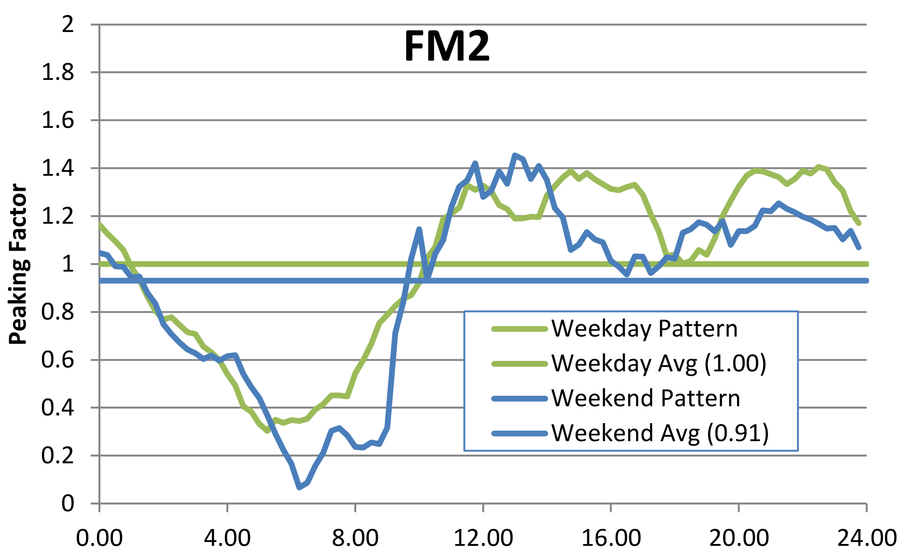

FM7
Depth (in)
160.0
140.0
120.0
100.0
80.0
60.0
40.0
20.0
0.0
0
1
2
3
4
5
Velocity (ft/s)
Meter Data
Pipe Diameter
Rim Height

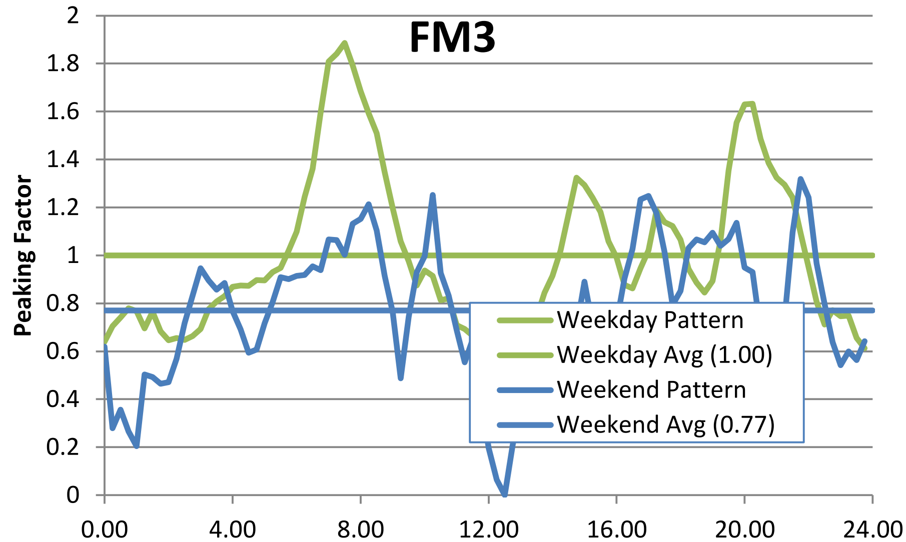

FM8
Depth (in)
20.0
18.0
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
0
1
1
2
2
3
3
Velocity (ft/s)
Meter Data

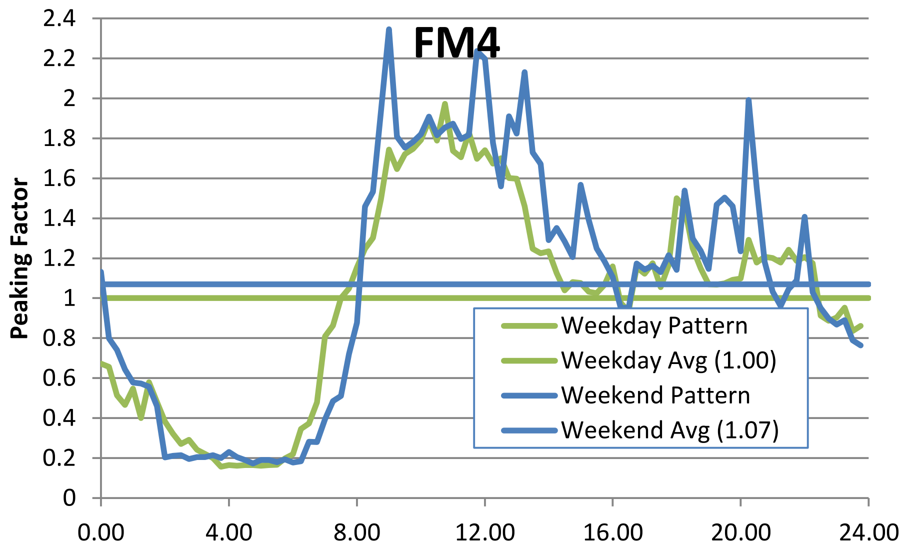
Appendix B: Diurnal Patterns

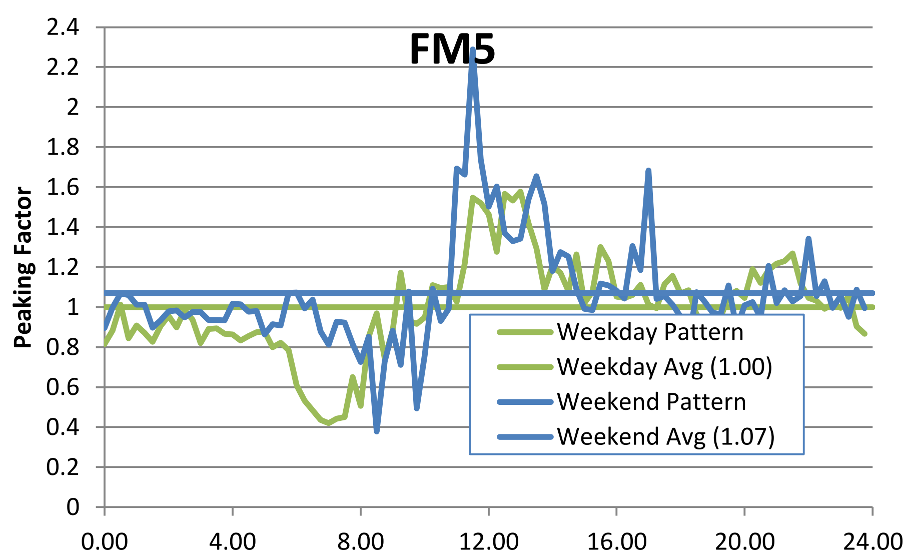
The following plots show the weekday and weekend diurnal patterns for the calibrated meters. The unit hydrographs serve to replicate the diurnal variation seen in municipal wastewater systems. Weekday patterns always average to a value of 1. Weekend patterns are based on a fraction of the weekday pattern. A flow meter that records higher dry weather flows on the weekends will have a weekend average greater than 1. Alternatively, weekend averages less than 1 indicate lower weekend flows.

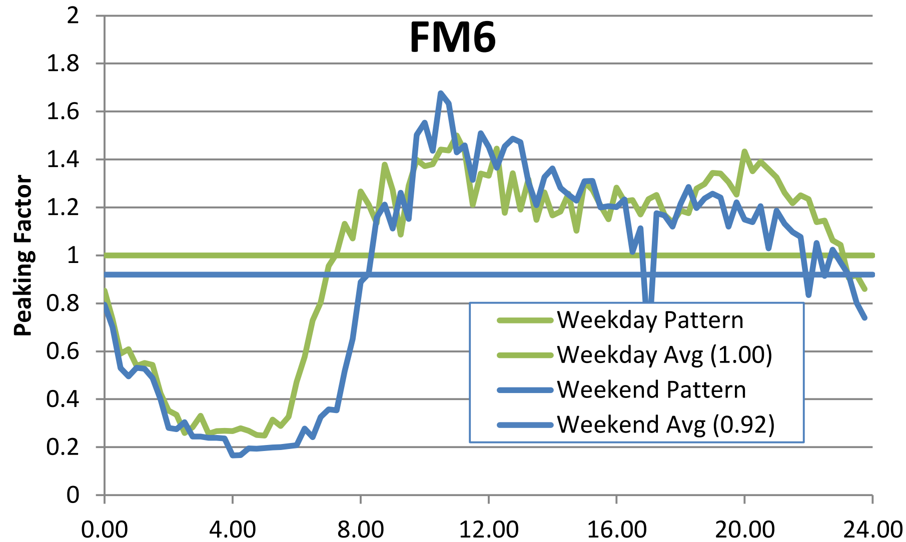


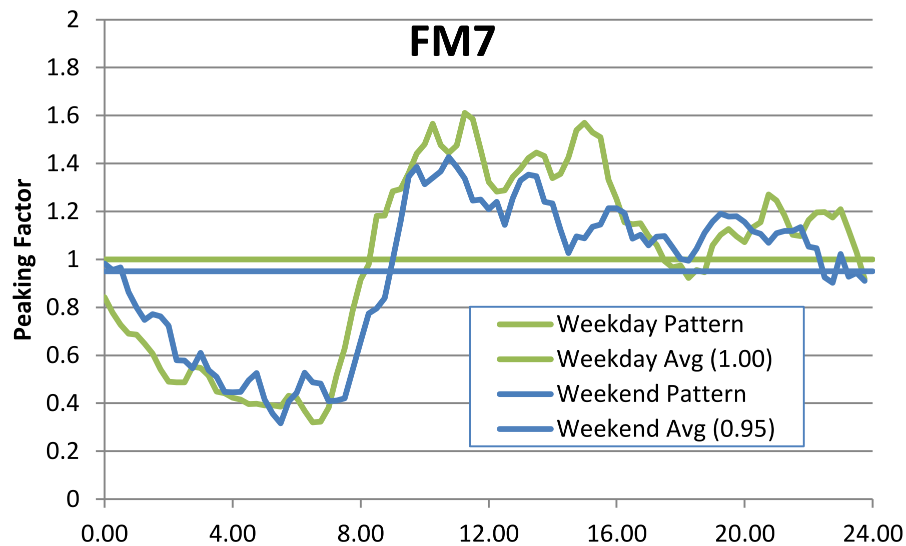


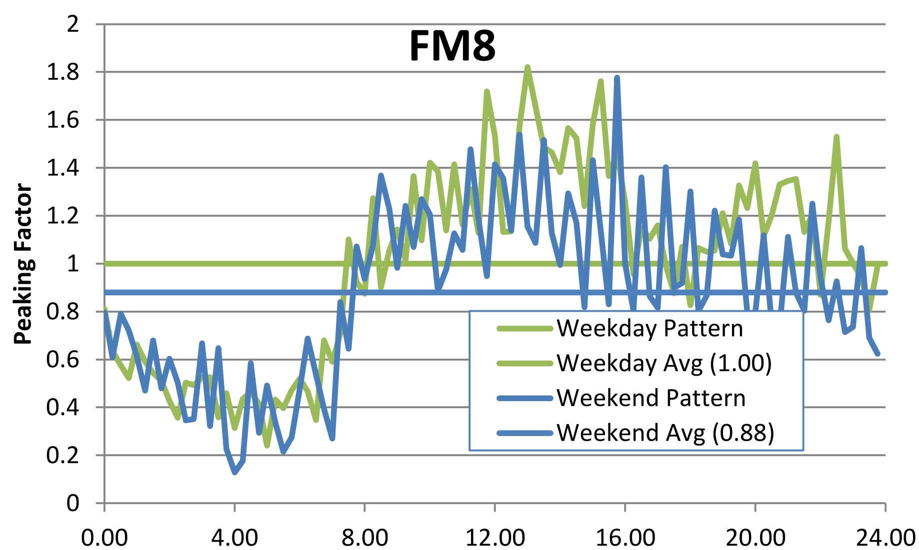

FM3
Peaking Factor
2
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0.00
4.00
8.00
12.00
16.00
20.00
24.00
Weekday Pattern
Weekday Avg (1.00)
Weekend Pattern
Weekend Avg (0.77)


FM4
Peaking Factor
2.4
2.2
2
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0.00
4.00
8.00
12.00
16.00
20.00
24.00
Weekday Pattern
Weekday Avg (1.00)
Weekend Pattern
Weekend Avg (1.07)


FM5
Peaking Factor
2.4
2.2
2
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0.00
4.00
8.00
12.00
16.00
20.00
24.00
Weekday Pattern
Weekday Avg (1.00)
Weekend Pattern
Weekend Avg (1.07)

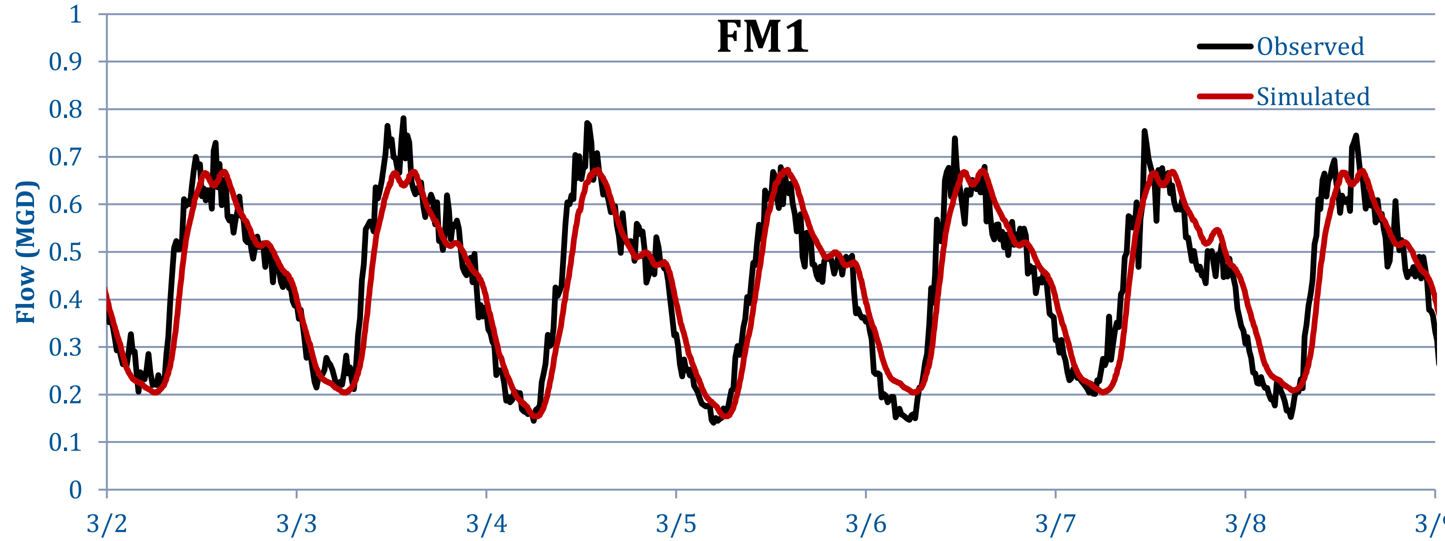

FM6
Peaking Factor
2
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0.00
4.00
8.00
12.00
16.00
20.00
24.00
Weekday Pattern
Weekday Avg (1.00)
Weekend Pattern
Weekend Avg (0.92)

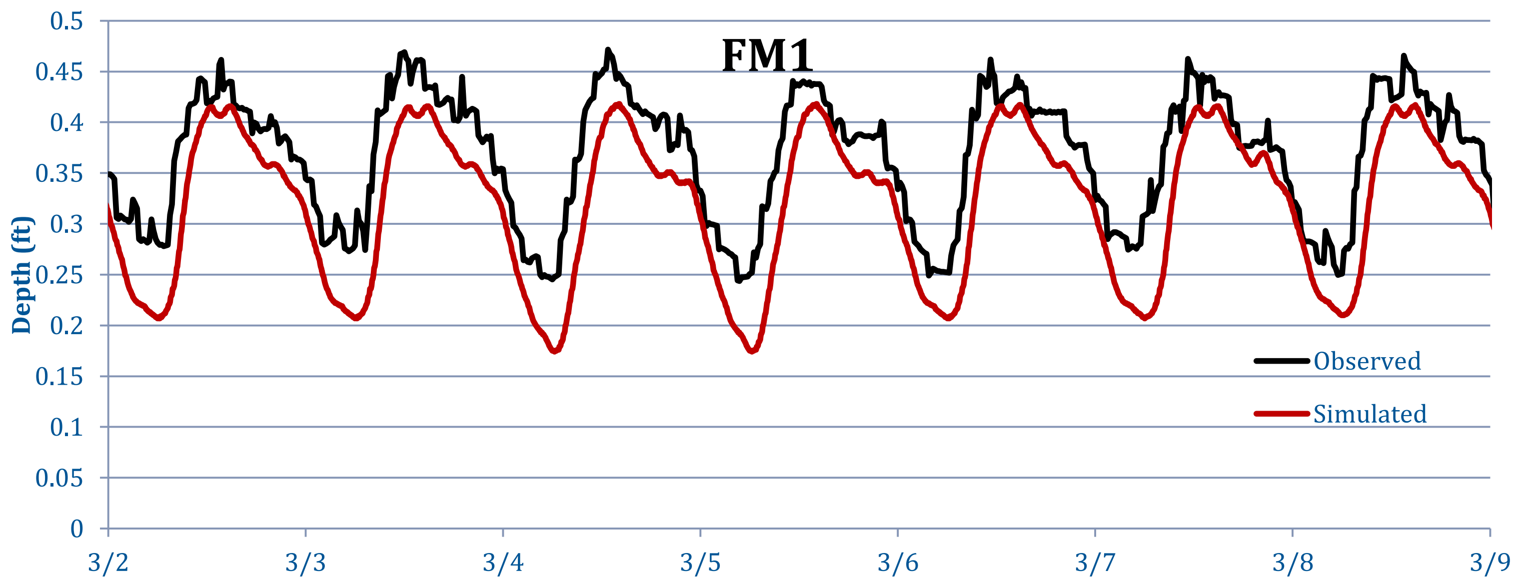

FM7
Peaking Factor
2
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0.00
4.00
8.00
12.00
16.00
20.00
24.00
Weekday Pattern
Weekday Avg (1.00)
Weekend Pattern
Weekend Avg (0.95)

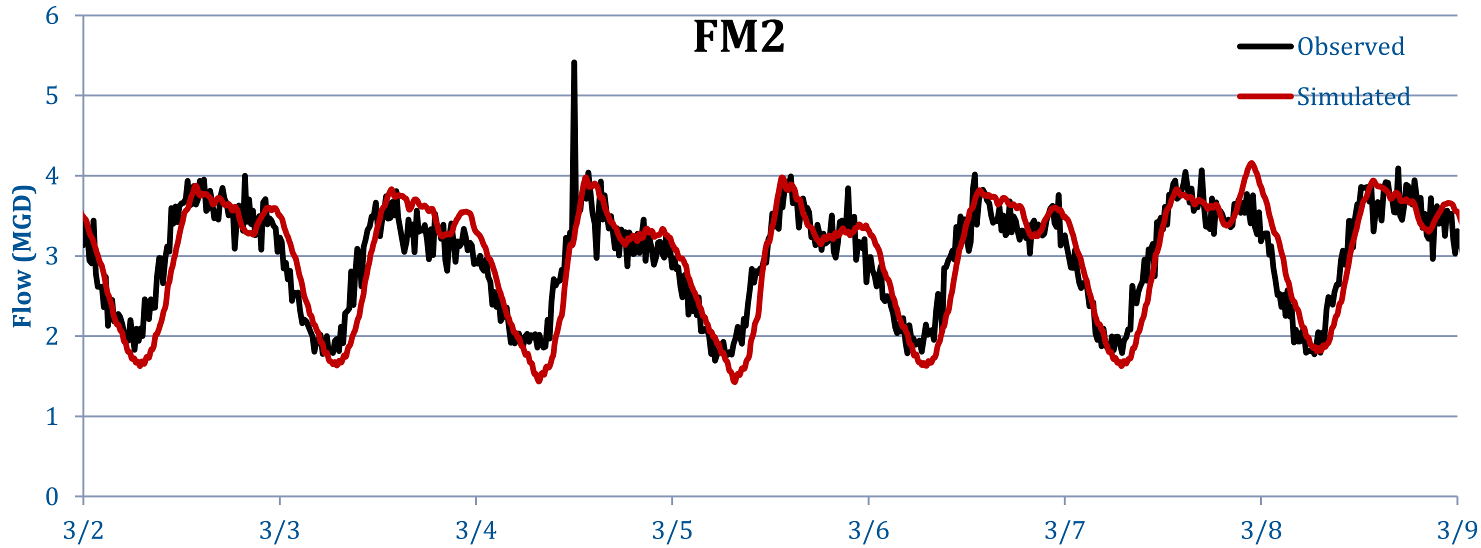

FM8
Peaking Factor
2
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0.00
4.00
8.00
12.00
16.00
20.00
24.00
Weekday Pattern
Weekday Avg (1.00)
Weekend Pattern
Weekend Avg (0.88)

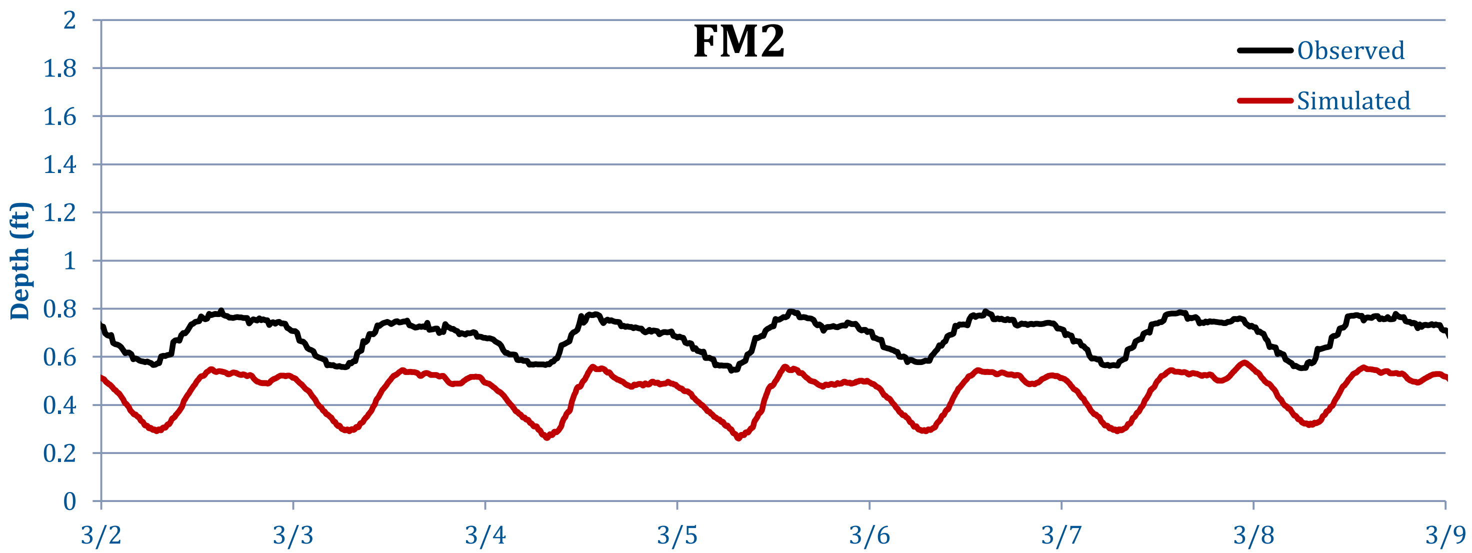
Appendix C: Dry Weather Calibration Plots

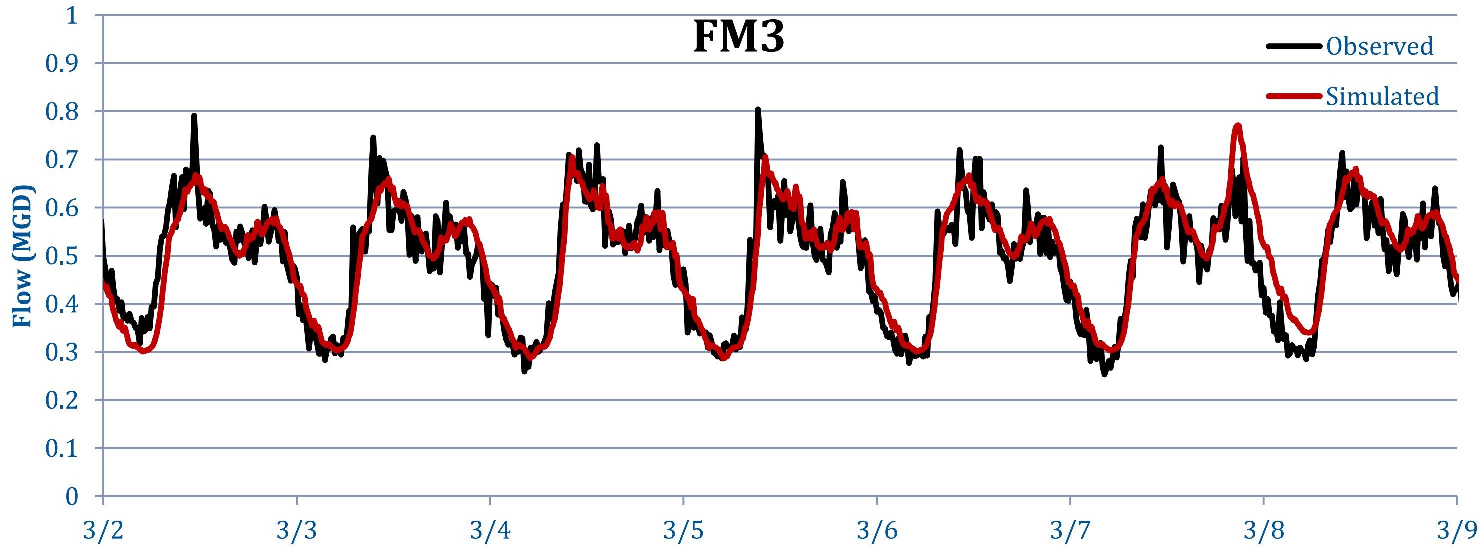
The following table details the calibration statistics for the dry weather calibration period. The attached plots show the weekday and weekend calibration for each calibrated flow meter. The primary calibration goal for the dry weather calibration is the shape and timing of the modeled and metered curves shown in the calibration plots.

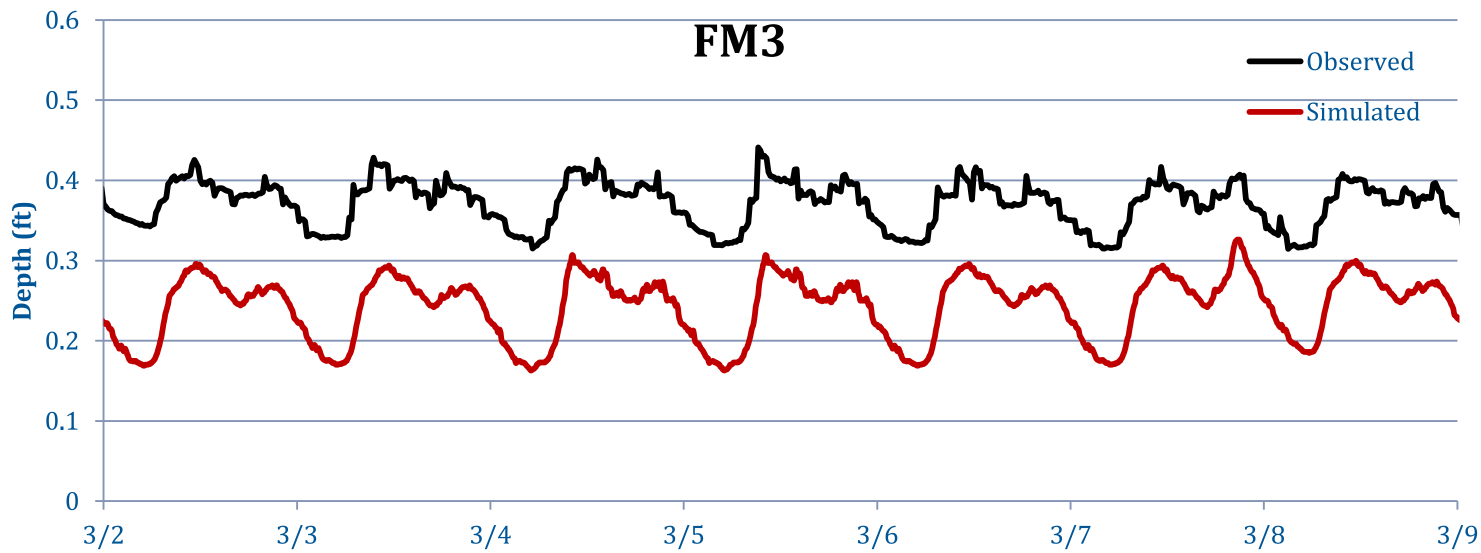

FM1
Observed
Simulated
Flow (MGD)
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
3/2
3/3
3/4
3/5
3/6
3/7
3/8
3/9

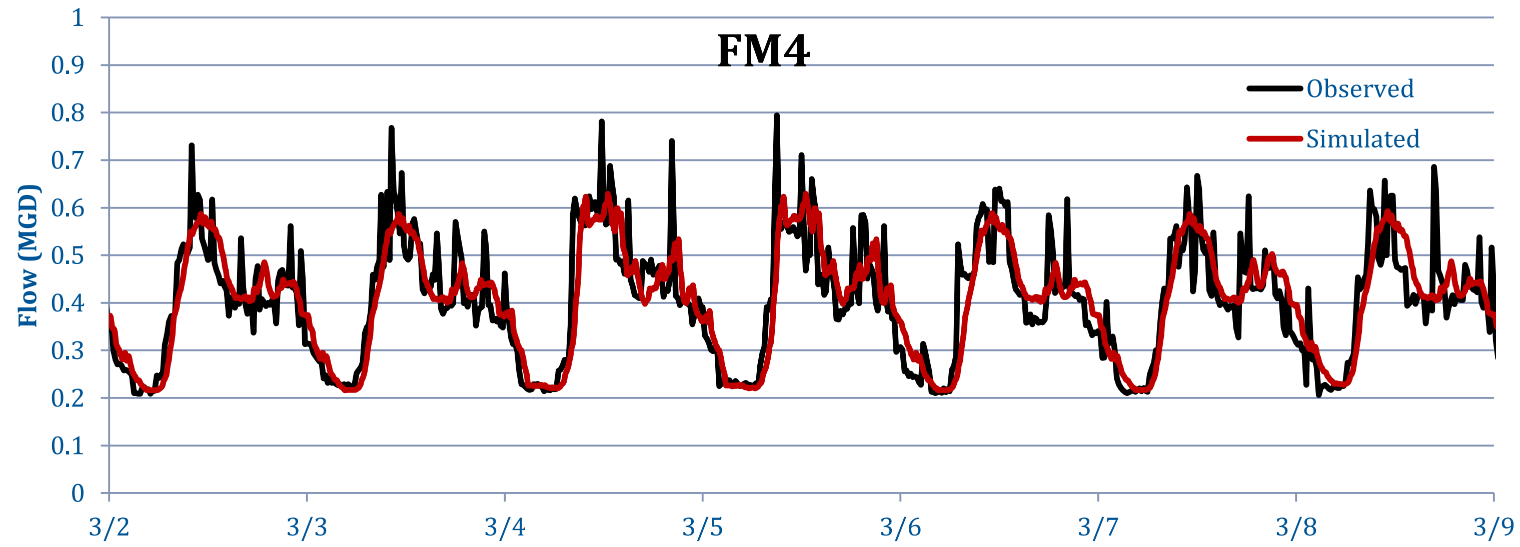

FM1
Observed
Simulated
Depth (ft)
0.5
0.45
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
3/2
3/3
3/4
3/5
3/6
3/7
3/8
3/9

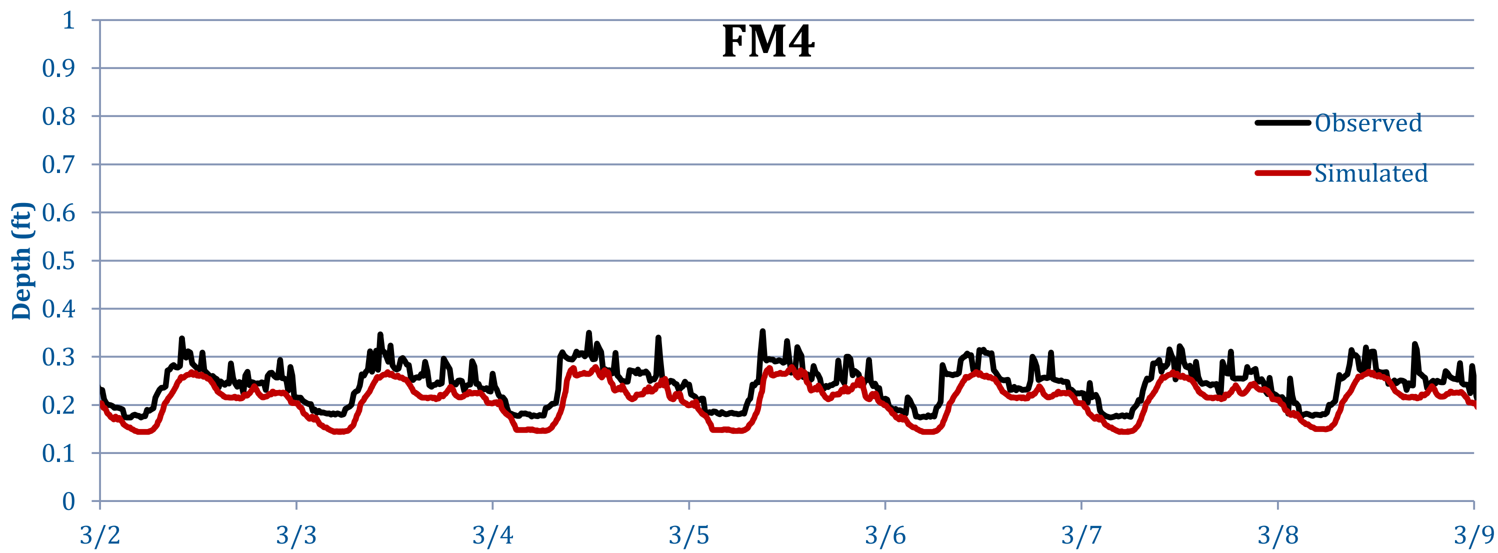

FM2
Observed
Simulated
Flow (MGD)
6
5
4
3
2
1
0
3/2
3/3
3/4
3/5
3/6
3/7
3/8
3/9

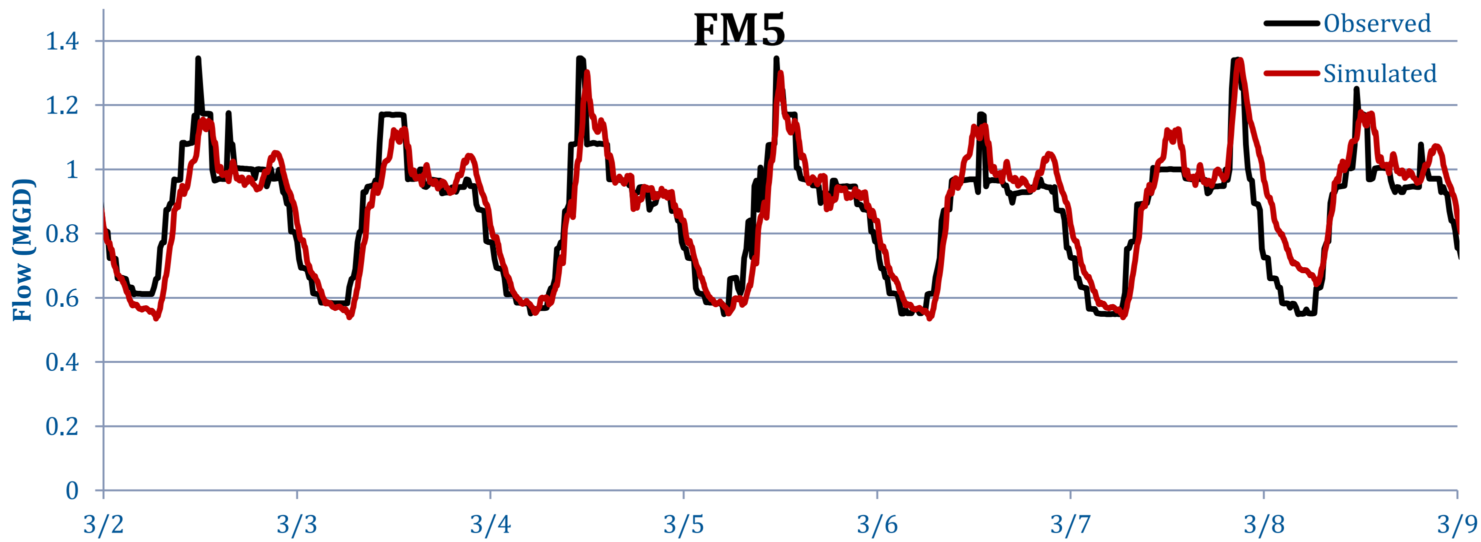

FM2
Observed
Simulated
Depth (ft)
2
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
3/2
3/3
3/4
3/5
3/6
3/7
3/8
3/9

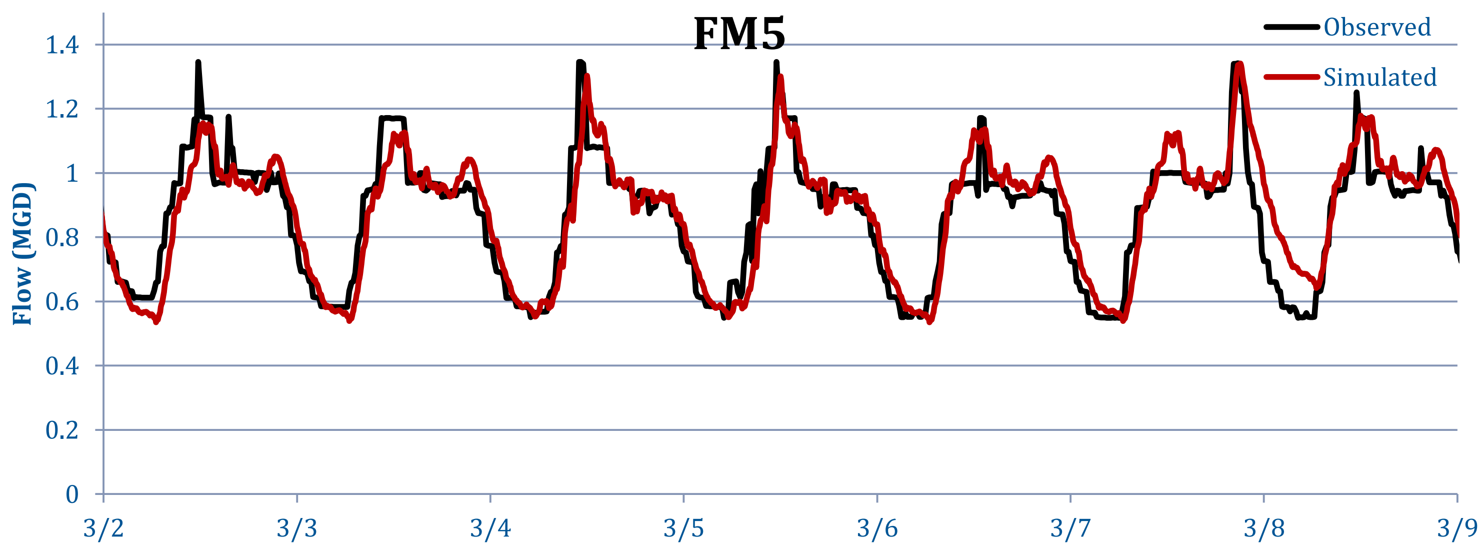

FM3
Observed
Simulated
Flow (MGD)
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
3/2
3/3
3/4
3/5
3/6
3/7
3/8
3/9

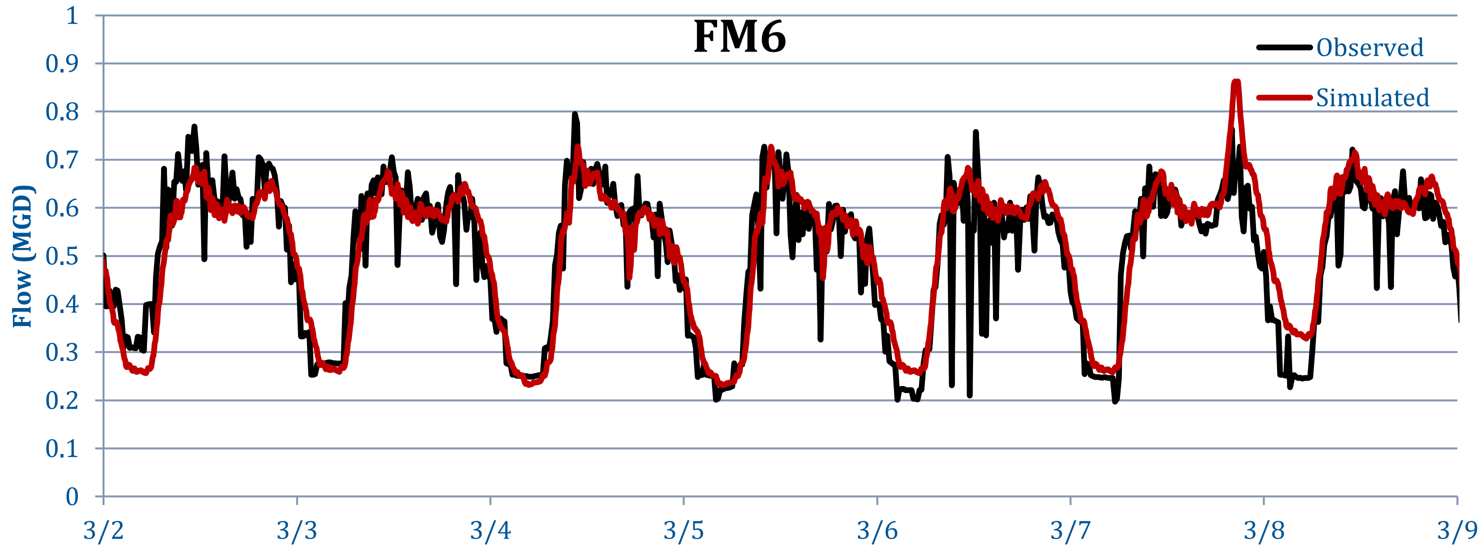

FM3
Observed
Simulated
Depth (ft)
0.6
0.5
0.4
0.3
0.2
0.1
0
3/2
3/3
3/4
3/5
3/6
3/7
3/8
3/9

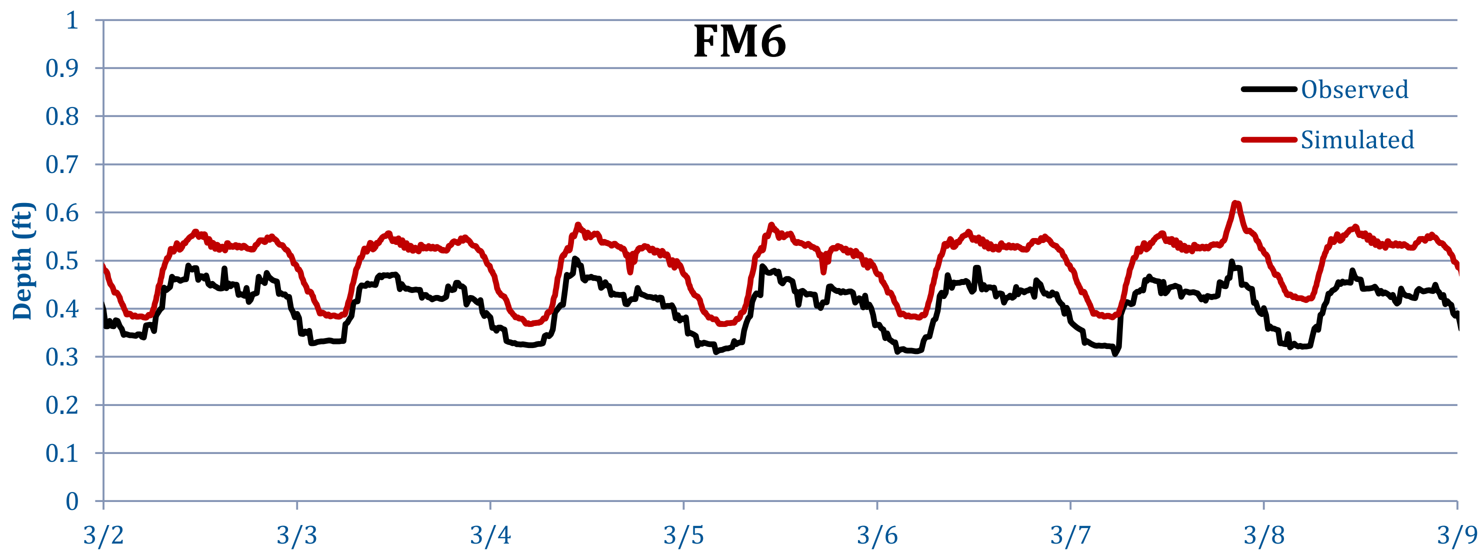

FM4
Observed
Simulated
Flow (MGD)
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
3/2
3/3
3/4
3/5
3/6
3/7
3/8
3/9

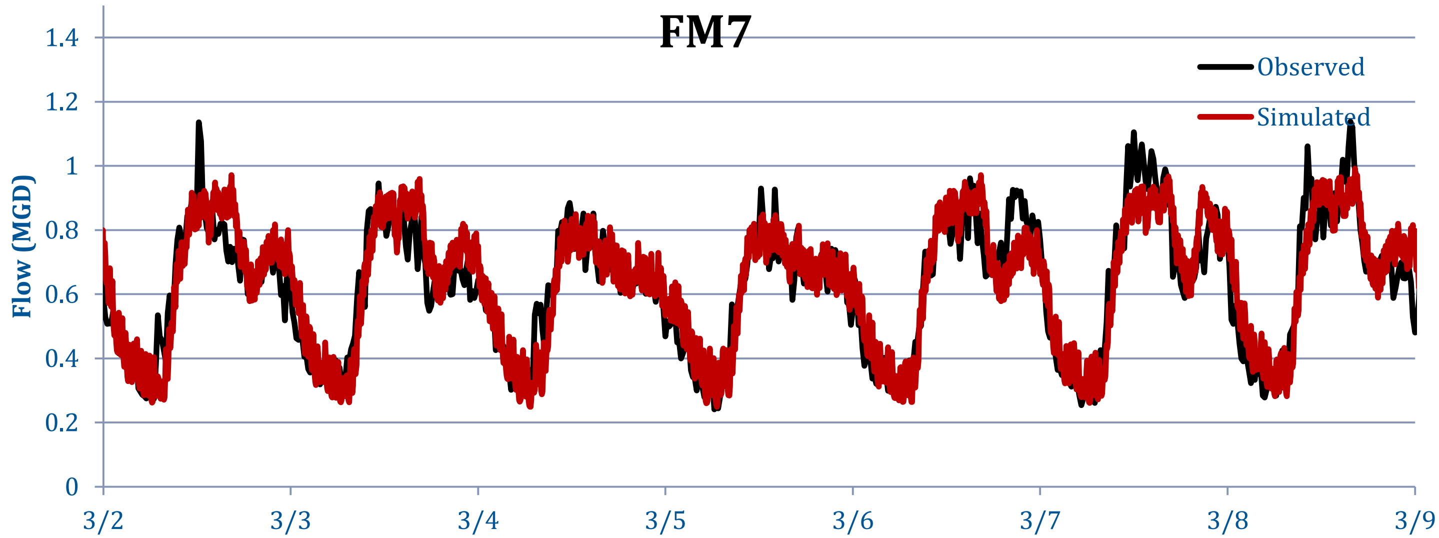

FM4
Observed
Simulated
Depth (ft)
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
3/2
3/3
3/4
3/5
3/6
3/7
3/8
3/9

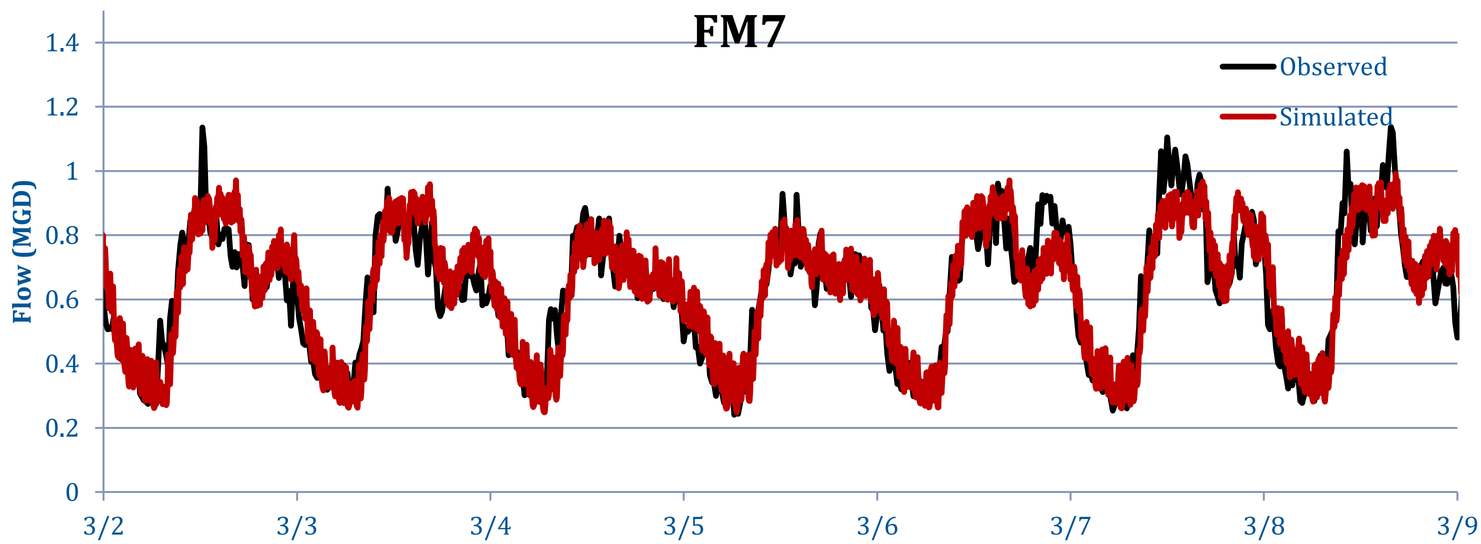

FM5
Observed
Simulated
Flow (MGD)
1.4
1.2
1
0.8
0.6
0.4
0.2
0
3/2
3/3
3/4
3/5
3/6
3/7
3/8
3/9

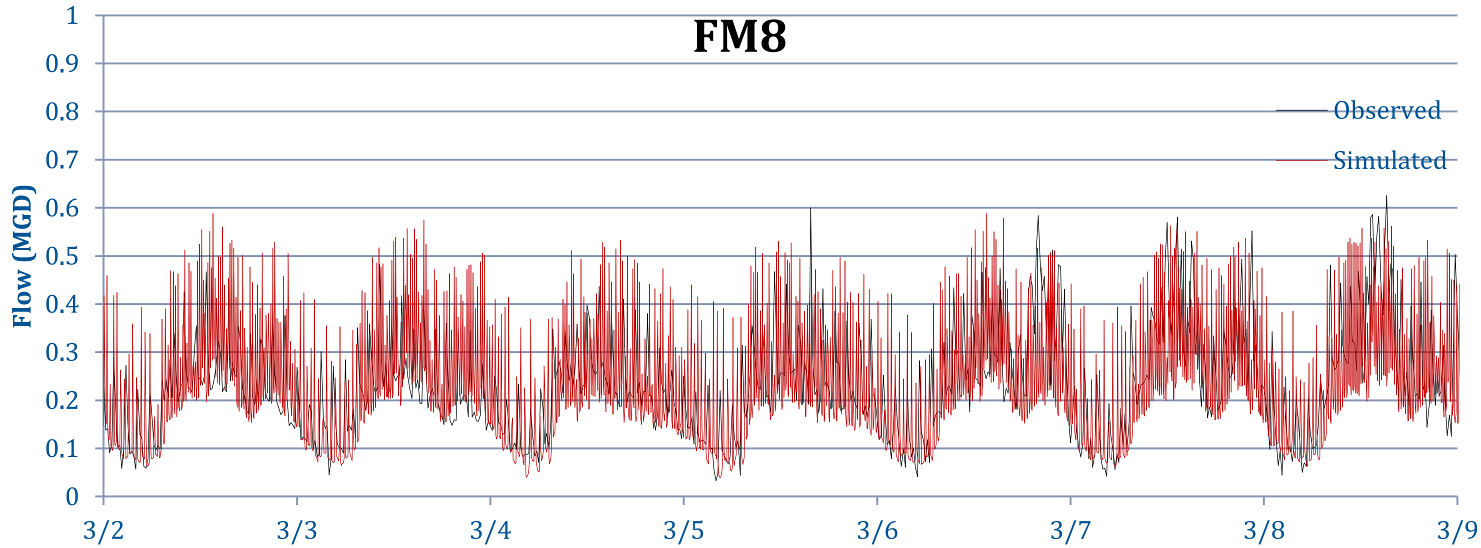

FM5
Observed
Simulated
Flow (MGD)
1.4
1.2
1
0.8
0.6
0.4
0.2
0
3/2
3/3
3/4
3/5
3/6
3/7
3/8
3/9

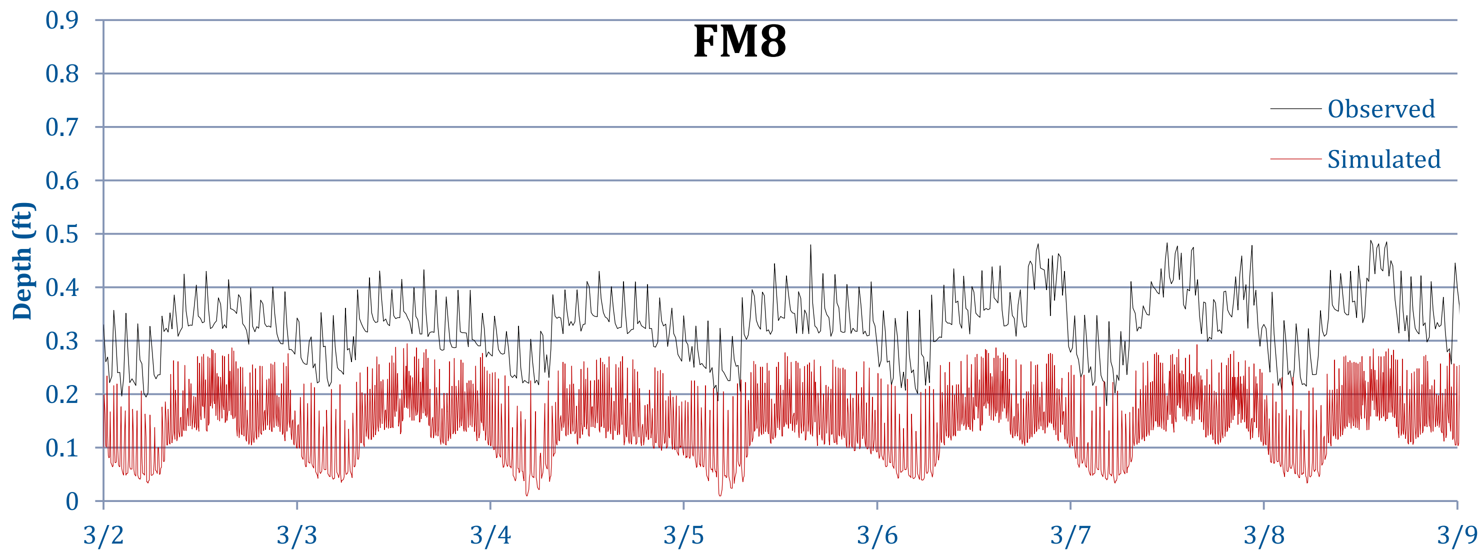

FM6
Observed
Simulated
Flow (MGD)
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
3/2
3/3
3/4
3/5
3/6
3/7
3/8
3/9


FM6
Observed
Simulated
Depth (ft)
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
3/2
3/3
3/4
3/5
3/6
3/7
3/8
3/9


FM7
Observed
Simulated
Flow (MGD)
0
0.2
0.4
0.6
0.8
1
1.2
1.4
3/2
3/3
3/4
3/5
3/6
3/7
3/8
3/9

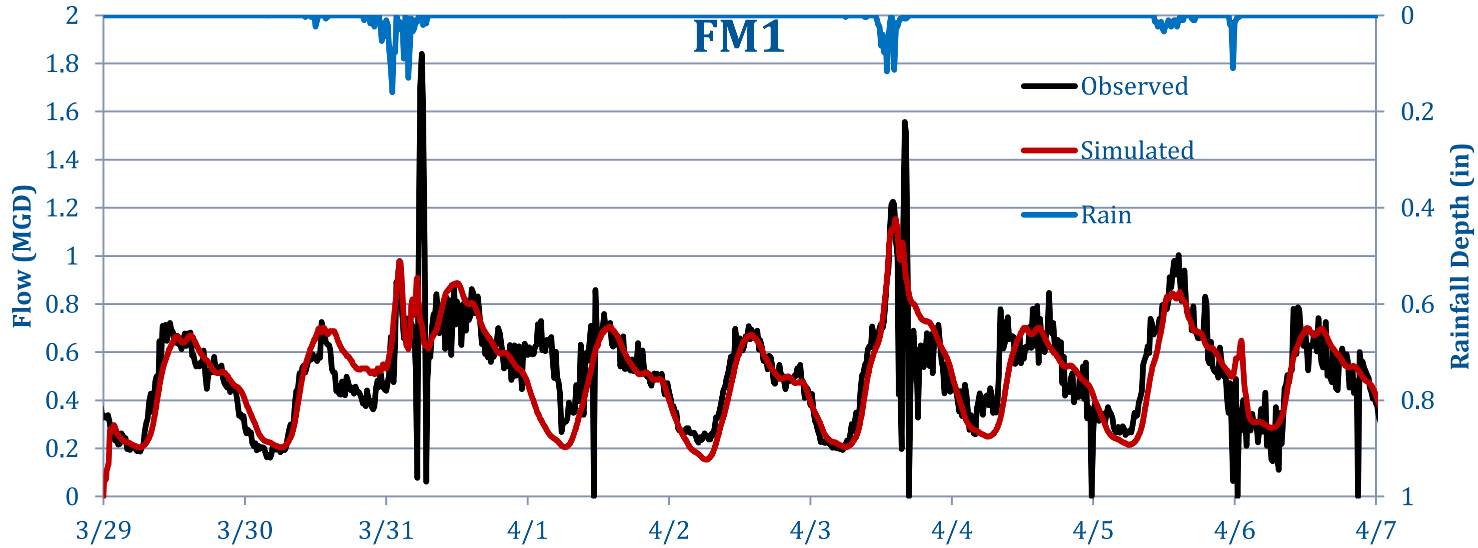

FM7
Observed
Simulated
Flow (MGD)
0
0.2
0.4
0.6
0.8
1
1.2
1.4
3/2
3/3
3/4
3/5
3/6
3/7
3/8
3/9

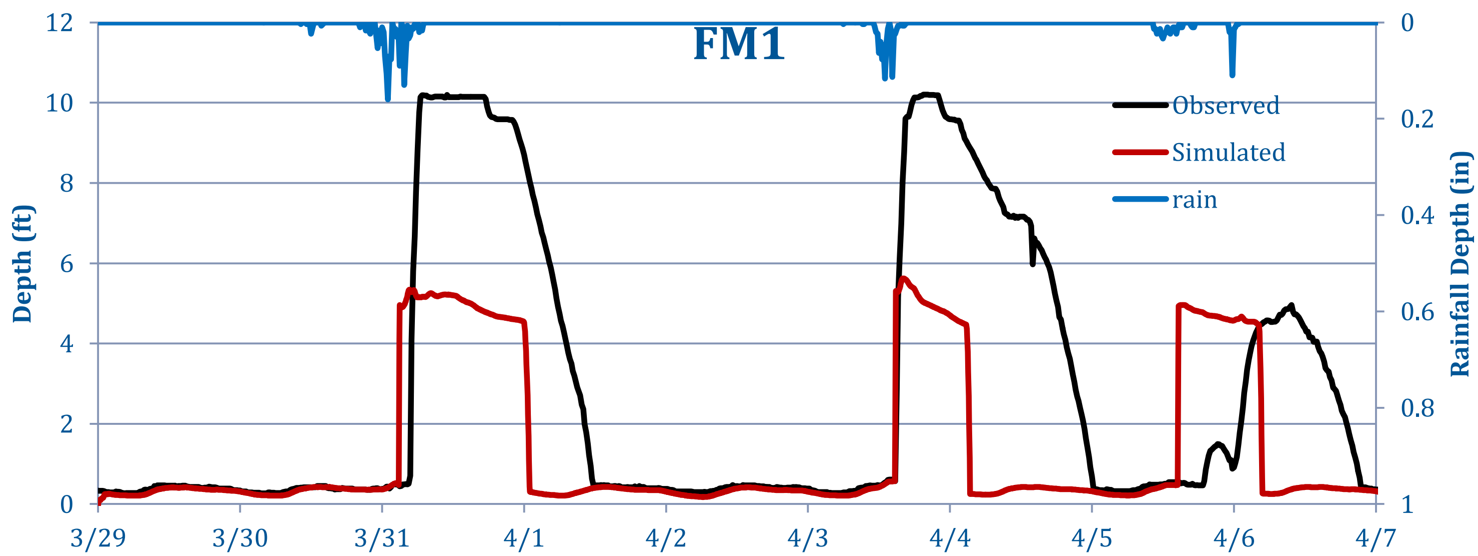

FM8
Flow (MGD)
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
3/2
3/3
3/4
3/5
3/6
3/7
3/8
3/9
Observed
Simulated

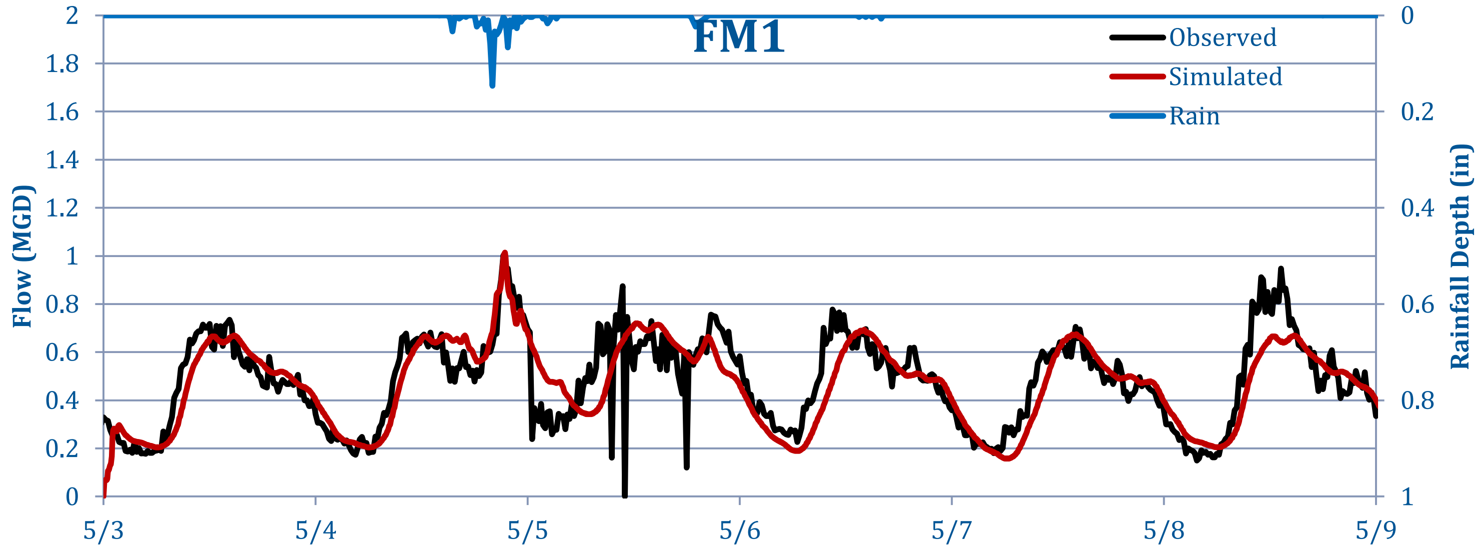

FM8
Depth (ft)
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
3/2
3/3
3/4
3/5
3/6
3/7
3/8
3/9
Observed
Simulated

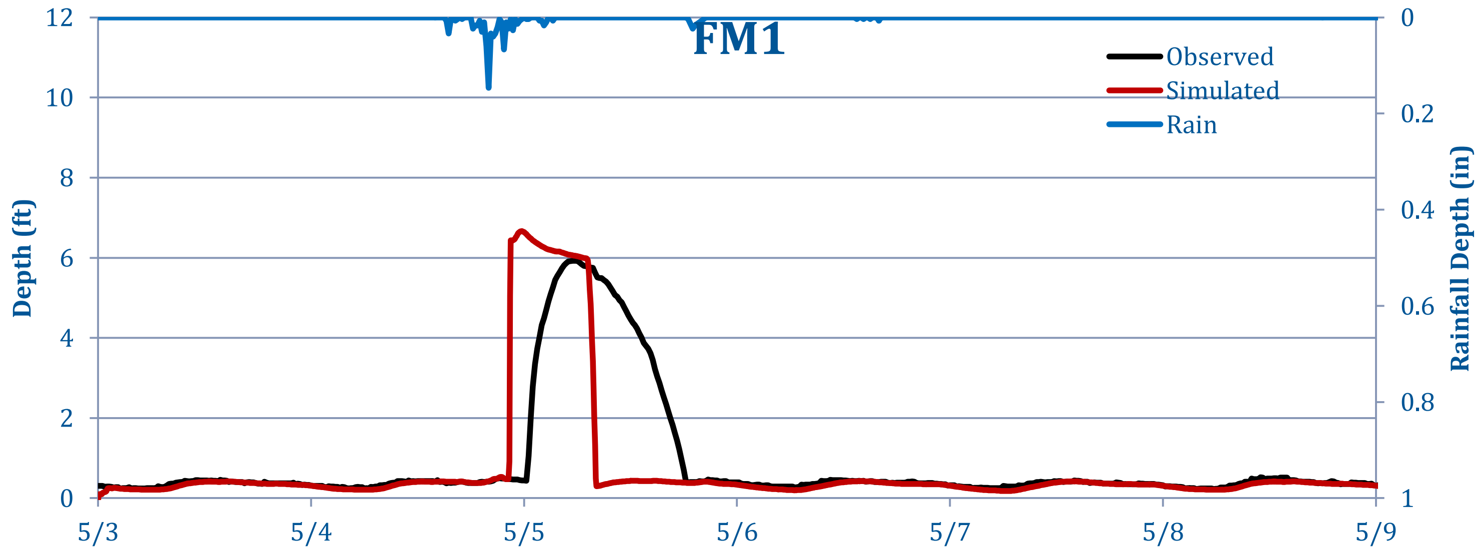
Appendix D: Wet Weather Calibration Plots

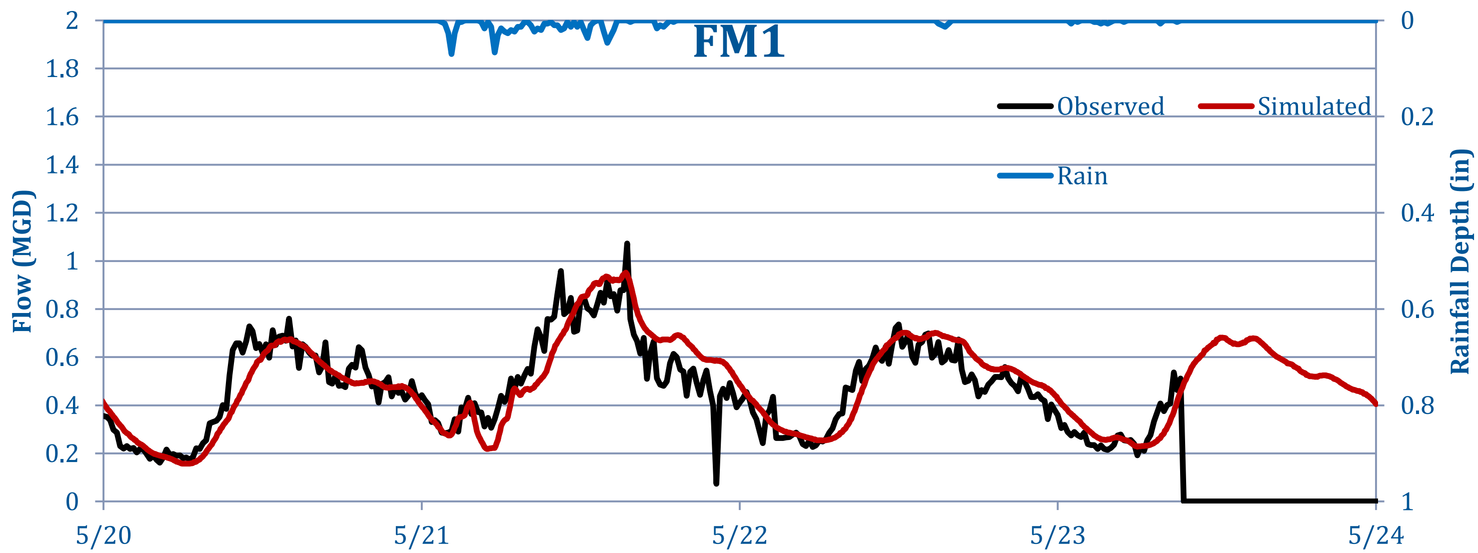
The following table details the calibration statistics for the wet weather calibration events. The attached plots show three storm calibrations for each calibrated flow meter. The primary calibration goal for the dry weather calibration is the shape and timing of the modeled and metered curves shown in the calibration plots.

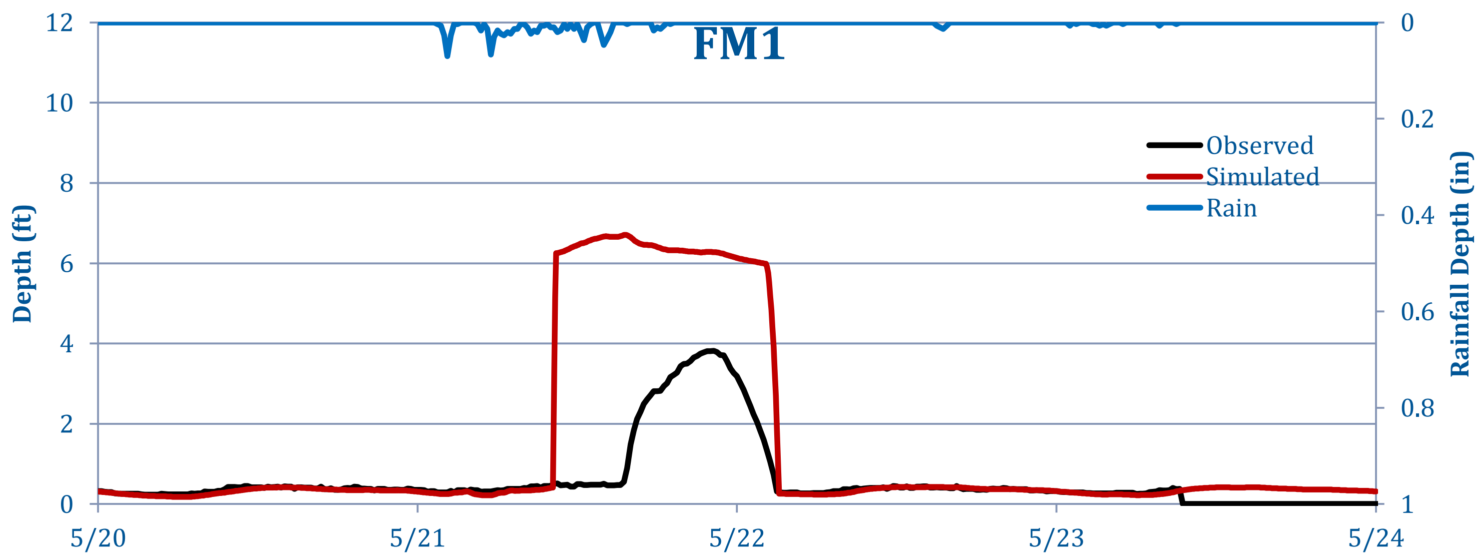

FM1
Flow (MGD)
Rainfall Depth (in)
Observed
Simulated
Rain
2
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0.2
0.4
0.6
0.8
1
3/29
3/30
3/31
4/1
4/2
4/3
4/4
4/5
4/6
4/7

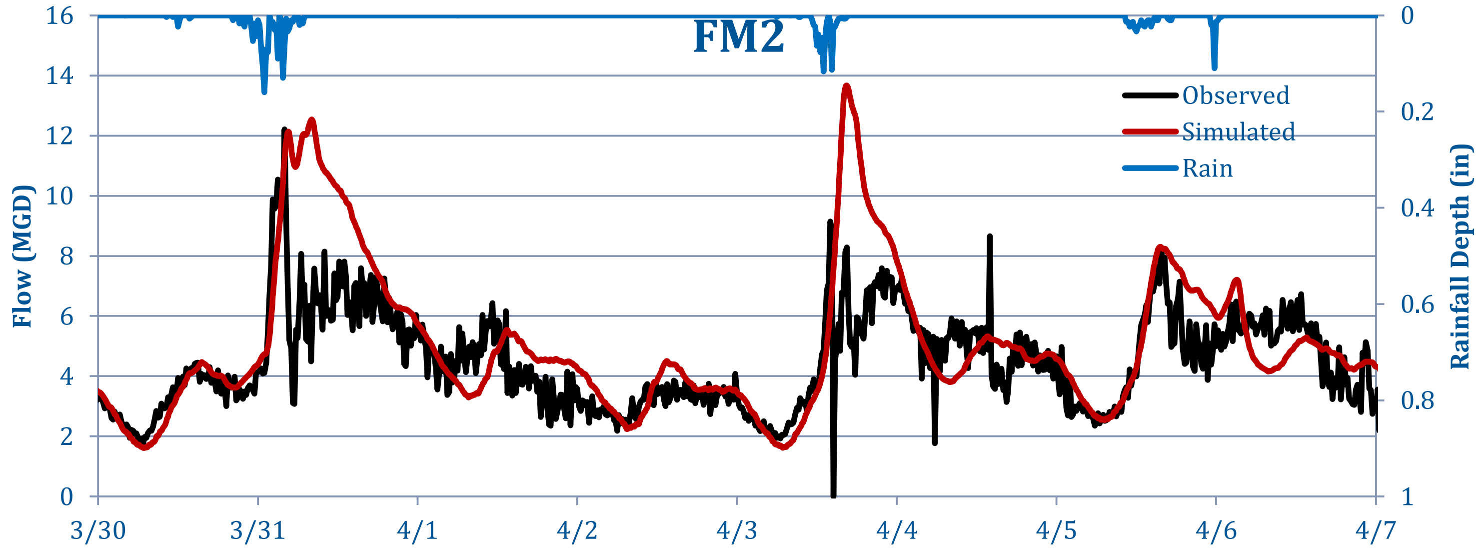

FM1
Depth (ft)
Rainfall Depth (in)
Observed
Simulated
rain
12
10
8
6
4
2
0
0.2
0.4
0.6
0.8
1
3/29
3/30
3/31
4/1
4/2
4/3
4/4
4/5
4/6
4/7


FM1
Observed
Simulated
Rain
Flow (MGD)
Rainfall Depth (in)
2
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0
0.2
0.4
0.6
0.8
1
5/3
5/4
5/5
5/6
5/7
5/8
5/9

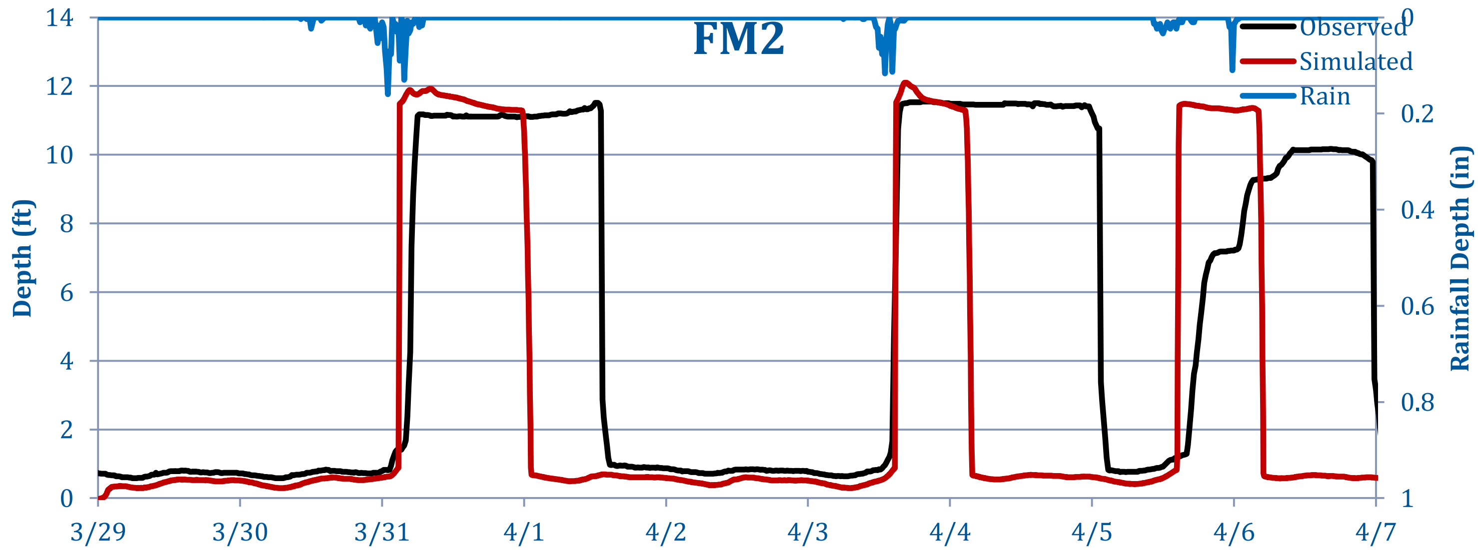

FM1
Observed
Simulated
Rain
Depth (ft)
Rainfall Depth (in)
12
10
8
6
4
2
0
0
0.2
0.4
0.6
0.8
1
5/3
5/4
5/5
5/6
5/7
5/8
5/9


FM1
Observed
Simulated
Rain
Flow (MGD)
Rainfall Depth (in)
2
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0
0.2
0.4
0.6
0.8
1
5/20
5/21
5/22
5/23
5/24

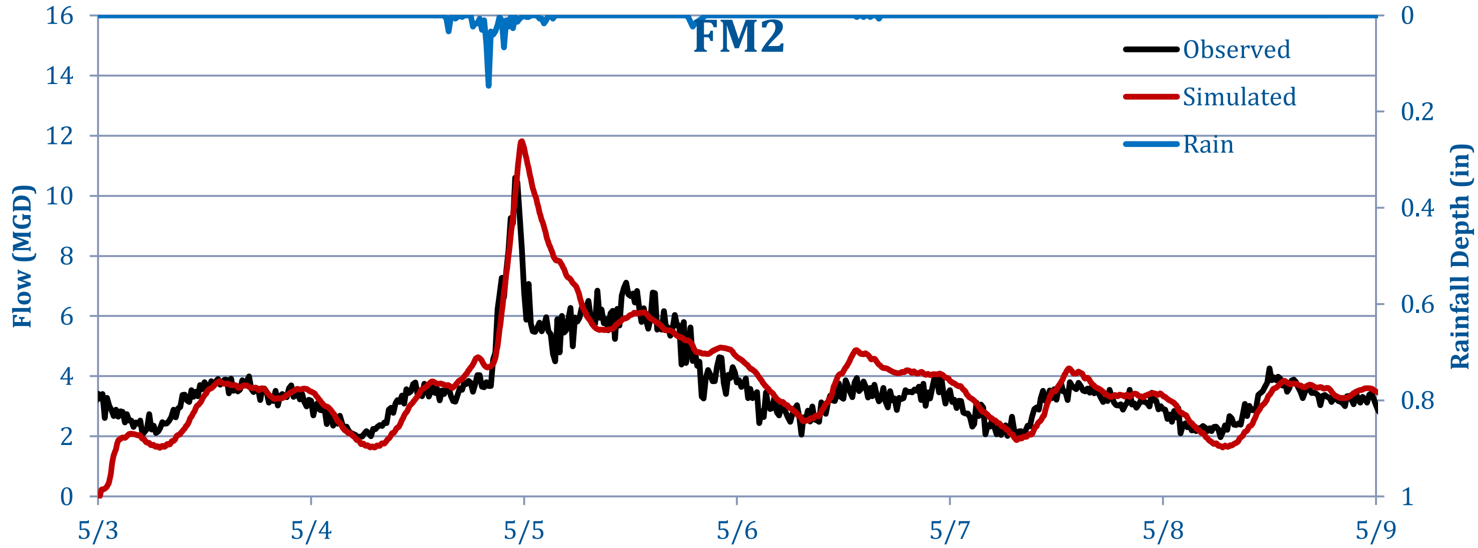

FM1
Observed
Simulated
Rain
Depth (ft)
Rainfall Depth (in)
12
10
8
6
4
2
0
0
0.2
0.4
0.6
0.8
1
5/20
5/21
5/22
5/23
5/24

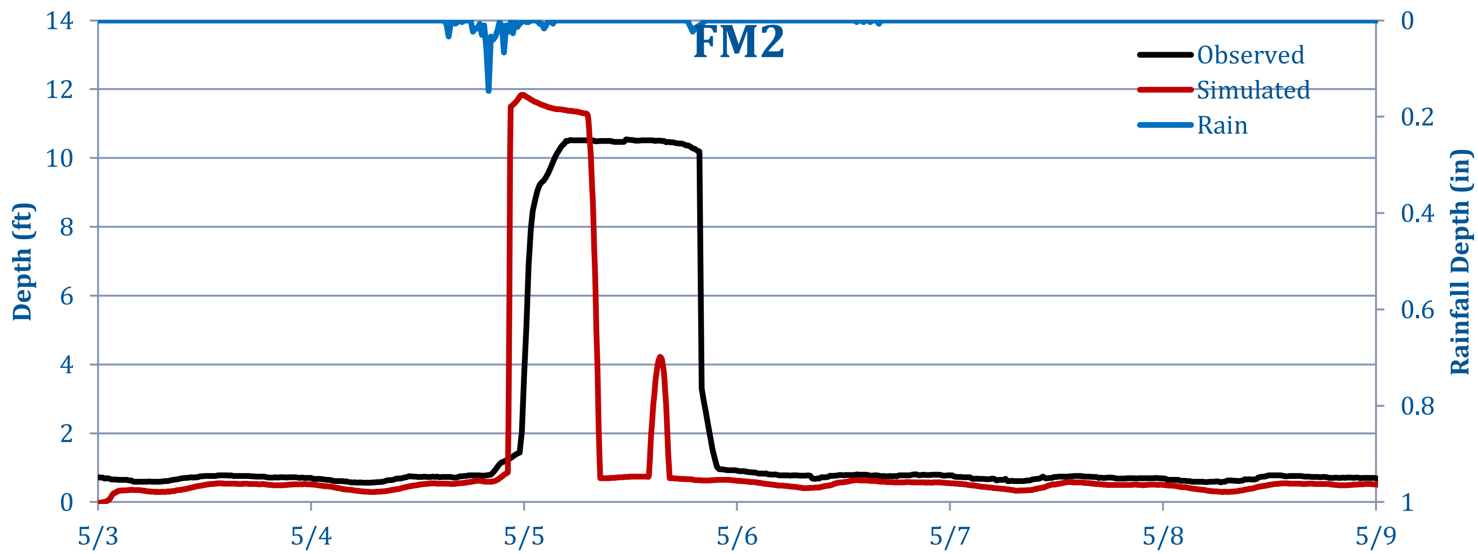


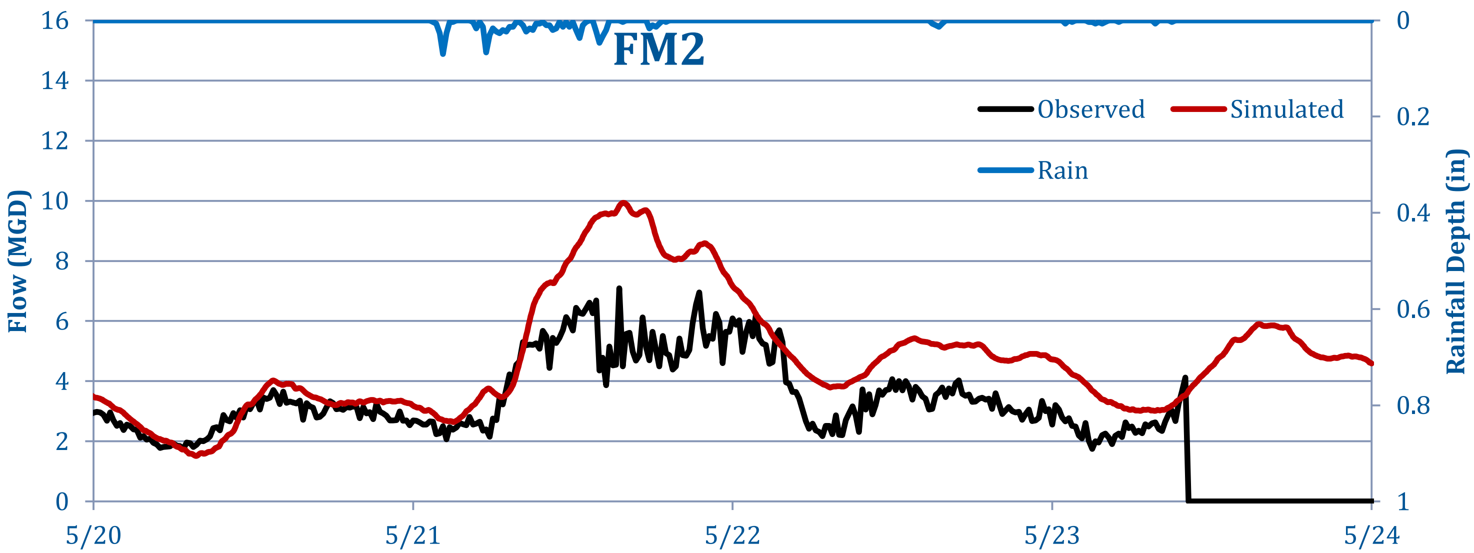
FM2
Flow (MGD)
Rainfall Depth (in)
16
14
12
10
8
6
4
2
0
0
0.2
0.4
0.6
0.8
1
3/30
3/31
4/1
4/2
4/3
4/4
4/5
4/6
4/7
Observed
Simulated
Rain

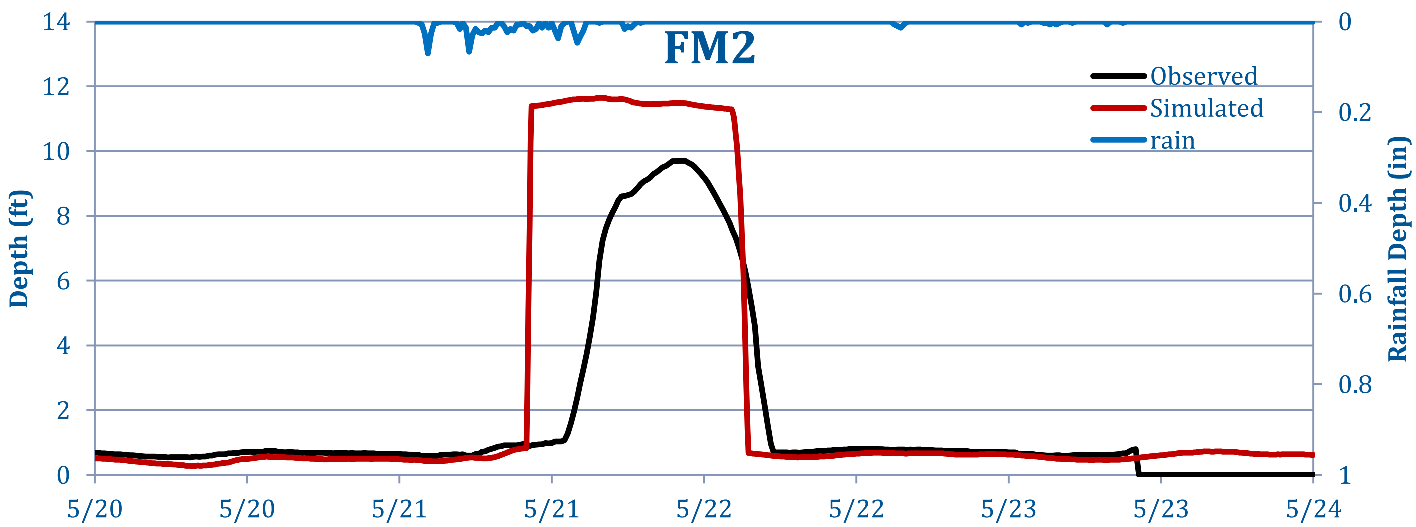


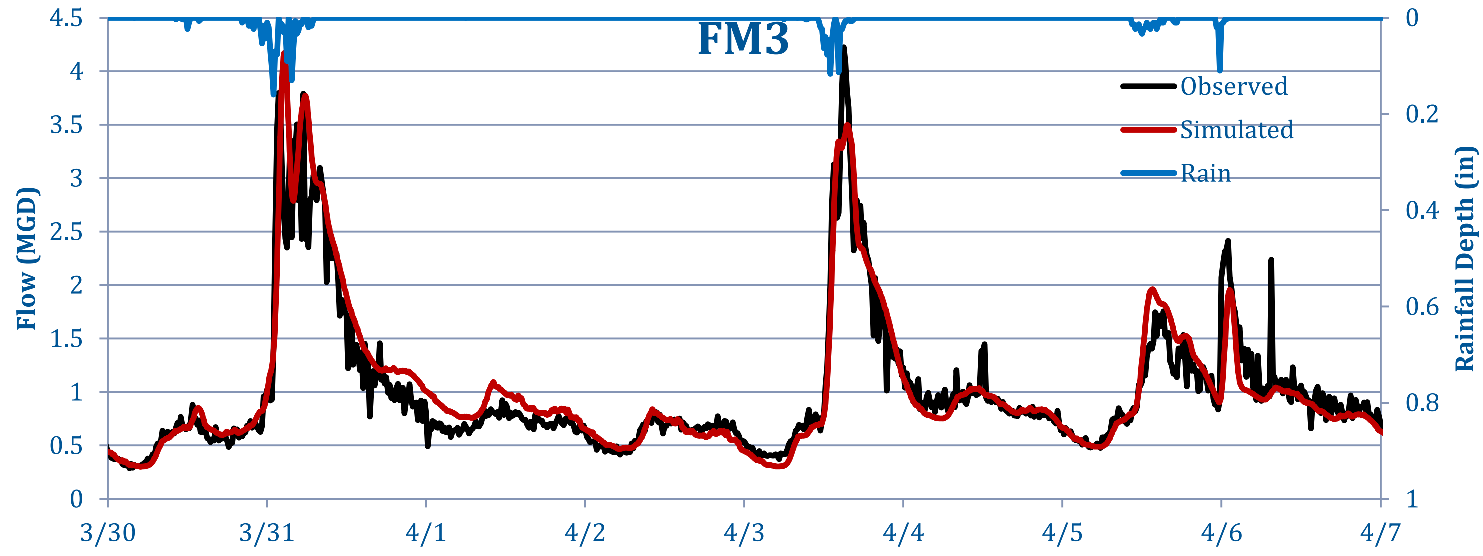
FM2
Depth (ft)
Rainfall Depth (in)
14
12
10
8
6
4
2
0
0
0.2
0.4
0.6
0.8
1
3/29
3/30
3/31
4/1
4/2
4/3
4/4
4/5
4/6
4/7
Observed
Simulated
Rain

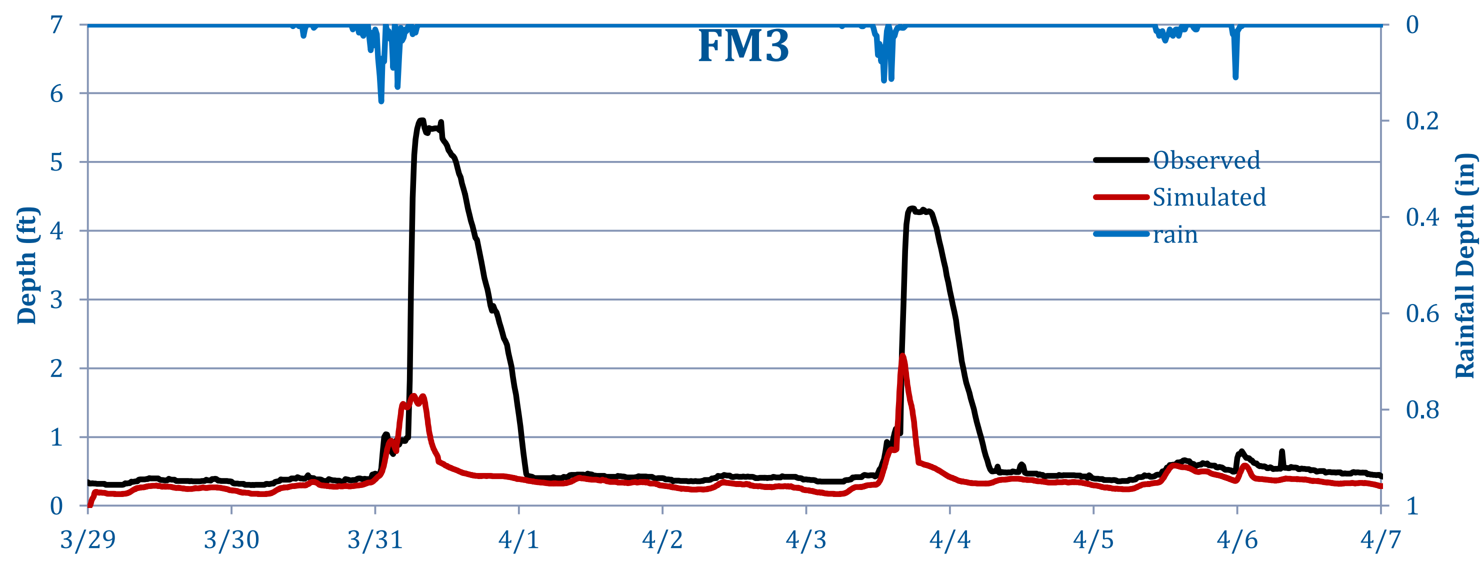

FM2
Observed
Simulated
Rain
Flow (MGD)
Rainfall Depth (in)
16
14
12
10
8
6
4
2
0
0
0.2
0.4
0.6
0.8
1
5/3
5/4
5/5
5/6
5/7
5/8
5/9

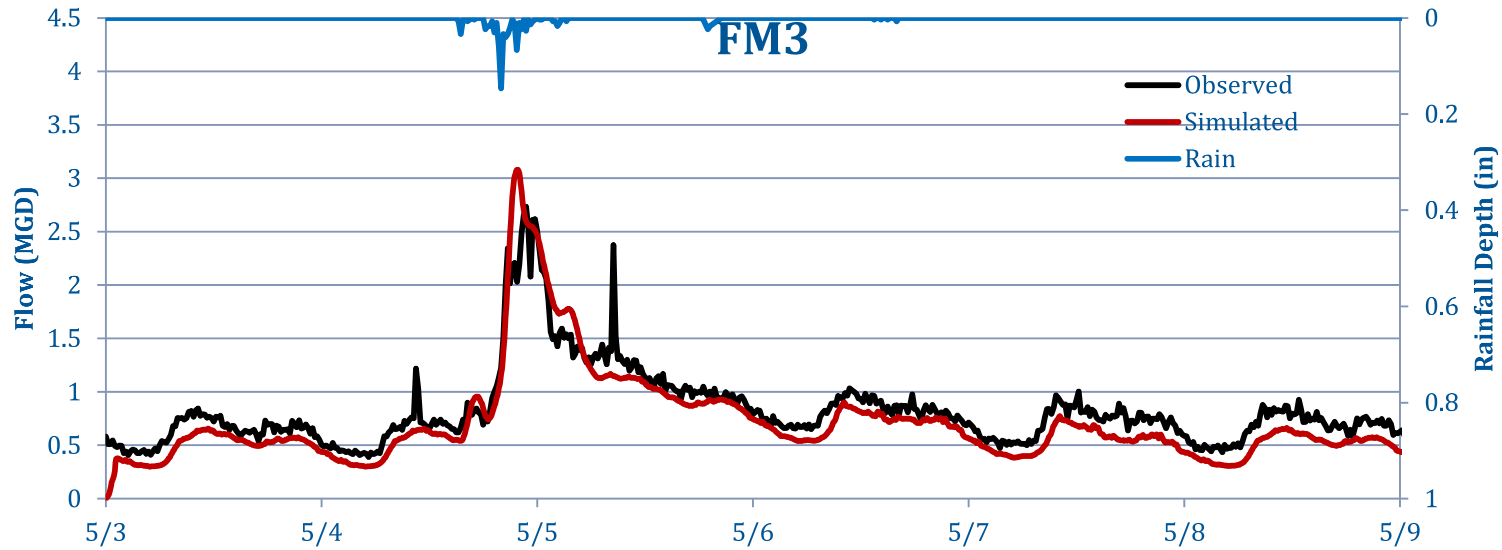

FM2
Observed
Simulated
Rain
Depth (ft)
Rainfall Depth (in)
14
12
10
8
6
4
2
0
0
0.2
0.4
0.6
0.8
1
5/3
5/4
5/5
5/6
5/7
5/8
5/9

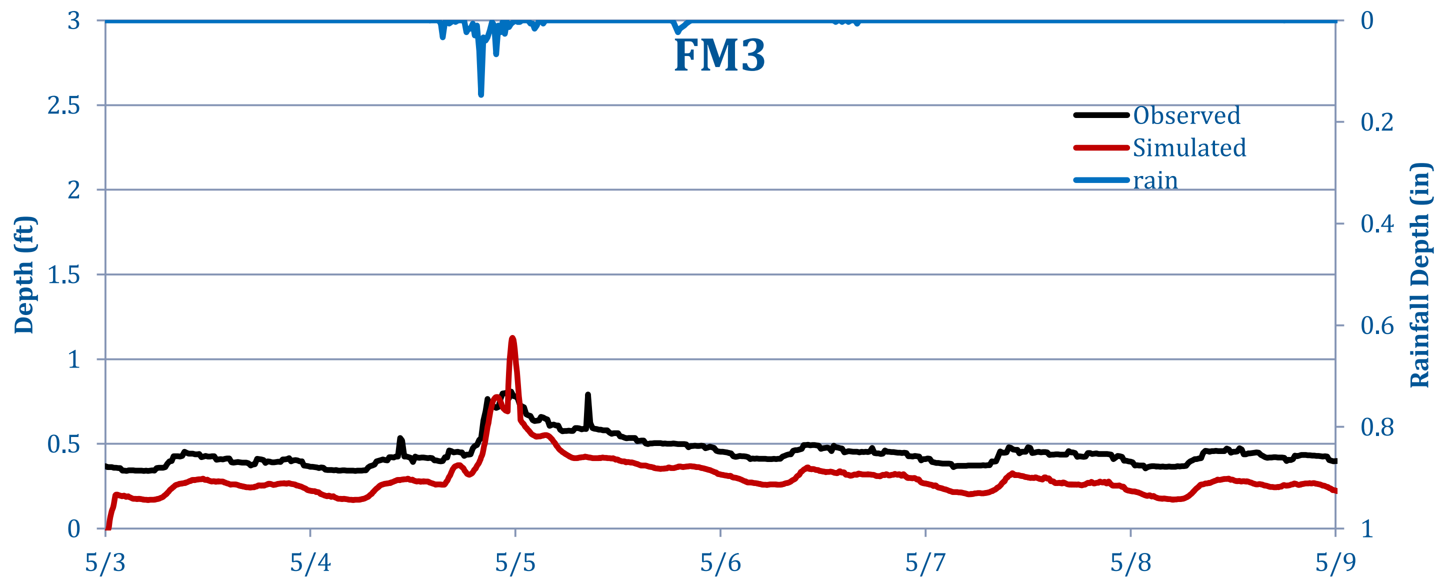

FM2
Observed
Simulated
Rain
Flow (MGD)
Rainfall Depth (in)
16
14
12
10
8
6
4
2
0
0
0.2
0.4
0.6
0.8
1
5/20
5/21
5/22
5/23
5/24

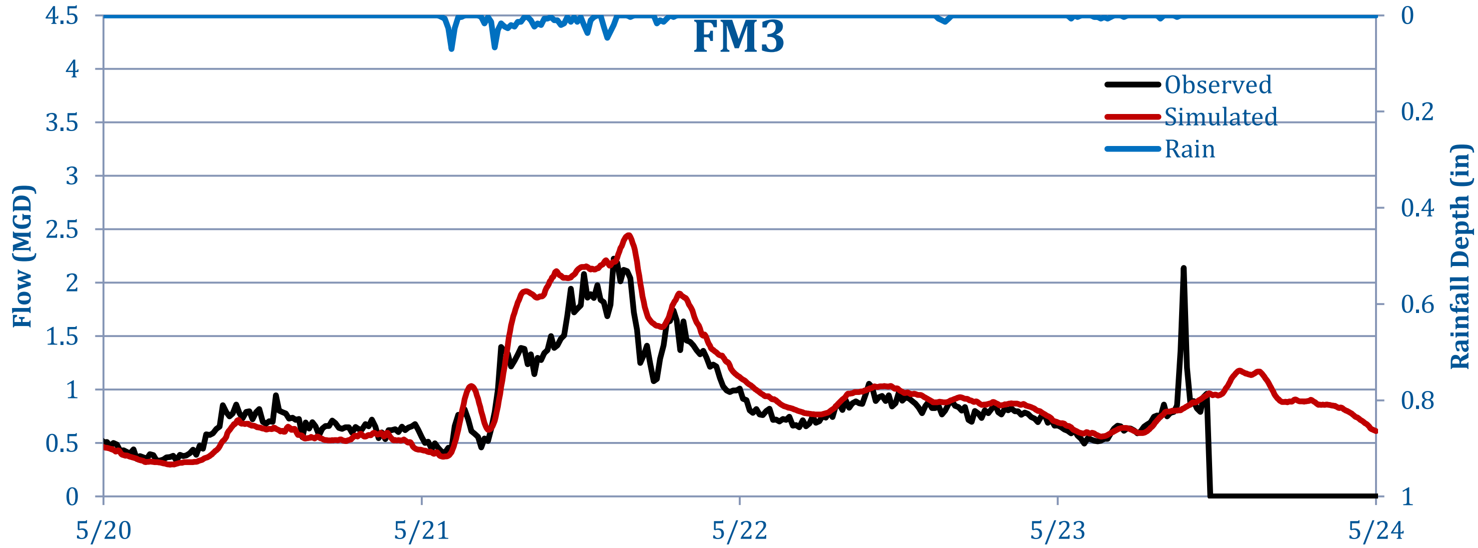

FM2
Observed
Simulated
rain
Depth (ft)
Rainfall Depth (in)
14
12
10
8
6
4
2
0
0
0.2
0.4
0.6
0.8
1
5/20
5/20
5/21
5/21
5/22
5/22
5/23
5/23
5/24

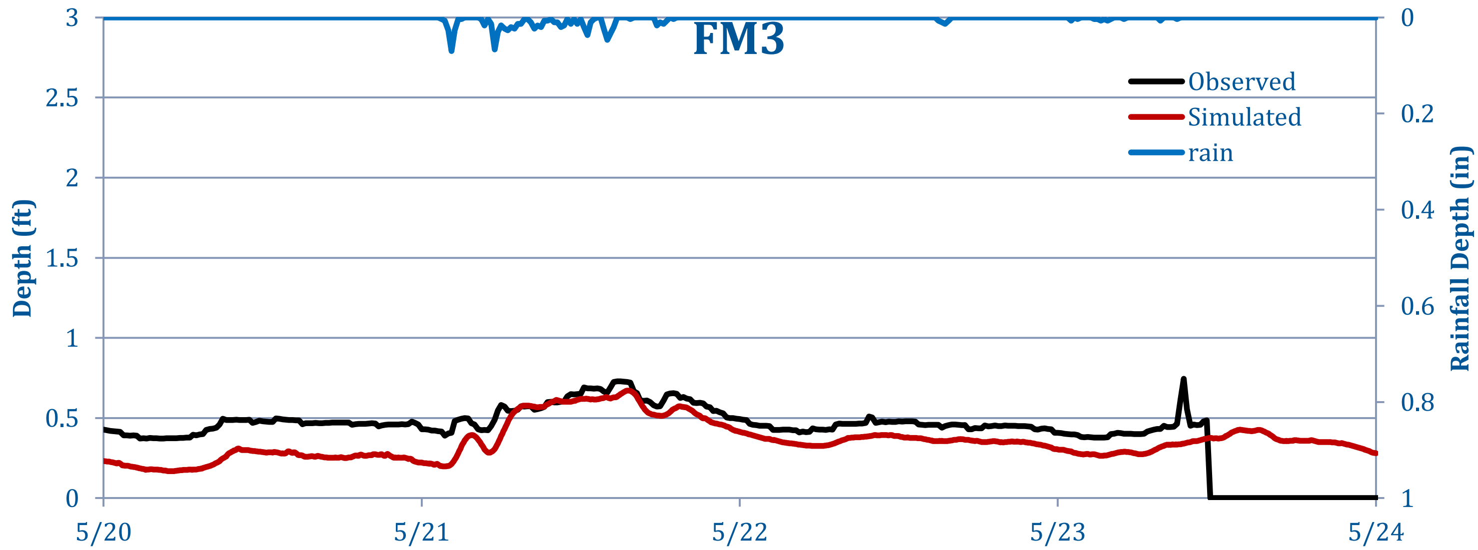

FM3
Flow (MGD)
Rainfall Depth (in)
Observed
Simulated
Rain
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
0
0.2
0.4
0.6
0.8
1
3/30
3/31
4/1
4/2
4/3
4/4
4/5
4/6
4/7


FM3
Depth (ft)
Rainfall Depth (in)
Observed
Simulated
rain
7
6
5
4
3
2
1
0
0
0.2
0.4
0.6
0.8
1
3/29
3/30
3/31
4/1
4/2
4/3
4/4
4/5
4/6
4/7

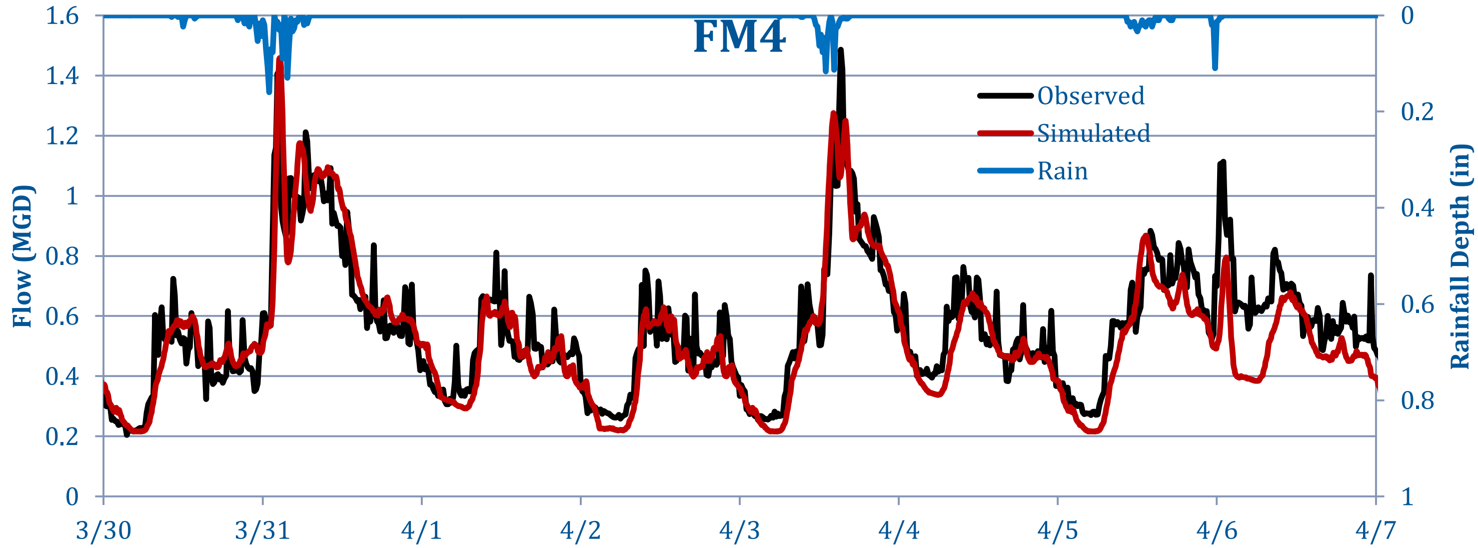

FM3
Observed
Simulated
Rain
Flow (MGD)
Rainfall Depth (in)
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
0
0.2
0.4
0.6
0.8
1
5/3
5/4
5/5
5/6
5/7
5/8
5/9

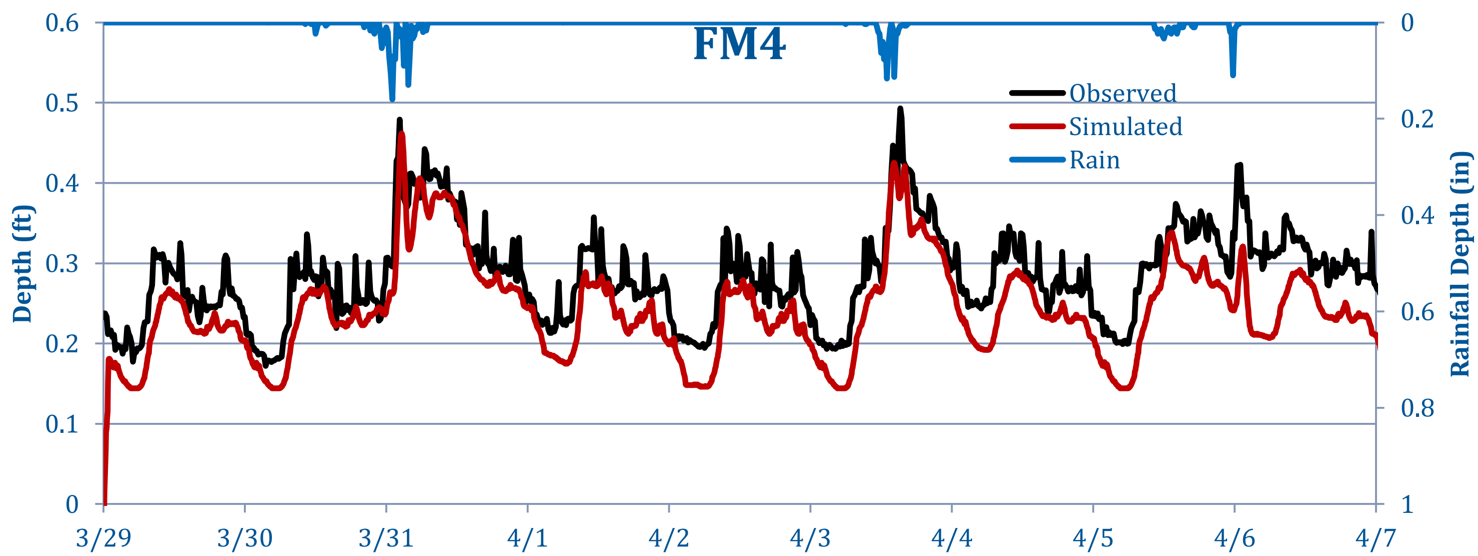

FM3
Observed
Simulated
rain
Depth (ft)
Rainfall Depth (in)
3
2.5
2
1.5
1
0.5
0
0
0.2
0.4
0.6
0.8
1
5/3
5/4
5/5
5/6
5/7
5/8
5/9

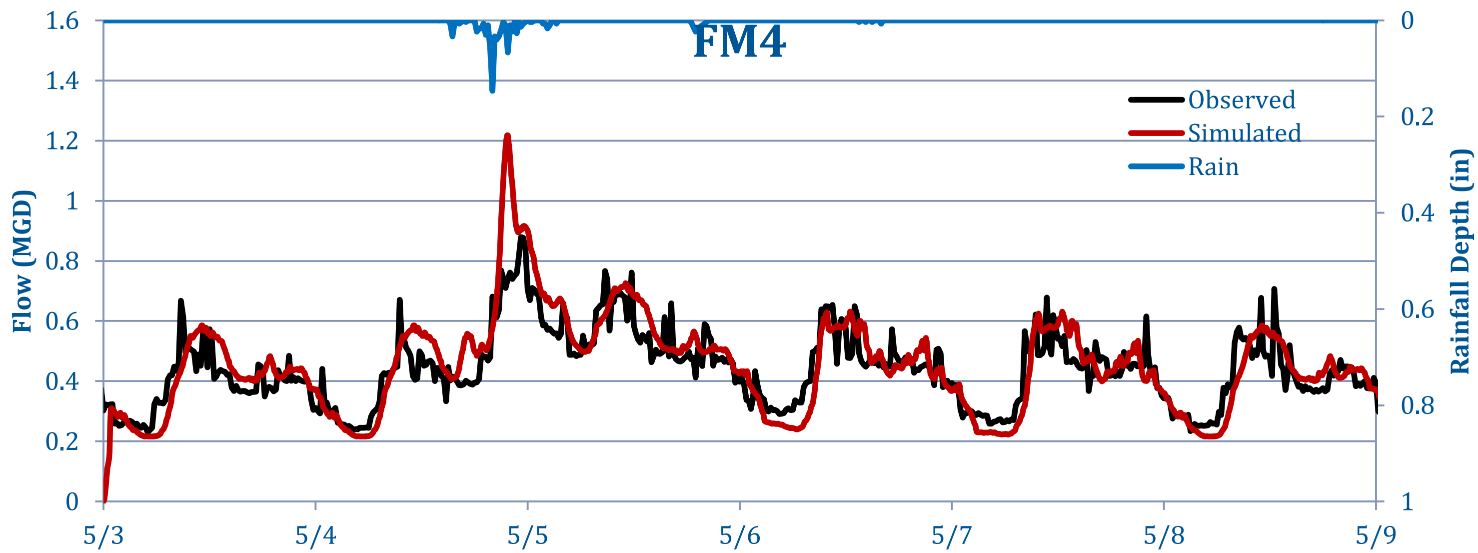

FM3
Observed
Simulated
Rain
Flow (MGD)
Rainfall Depth (in)
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
0
0.2
0.4
0.6
0.8
1
5/20
5/21
5/22
5/23
5/24

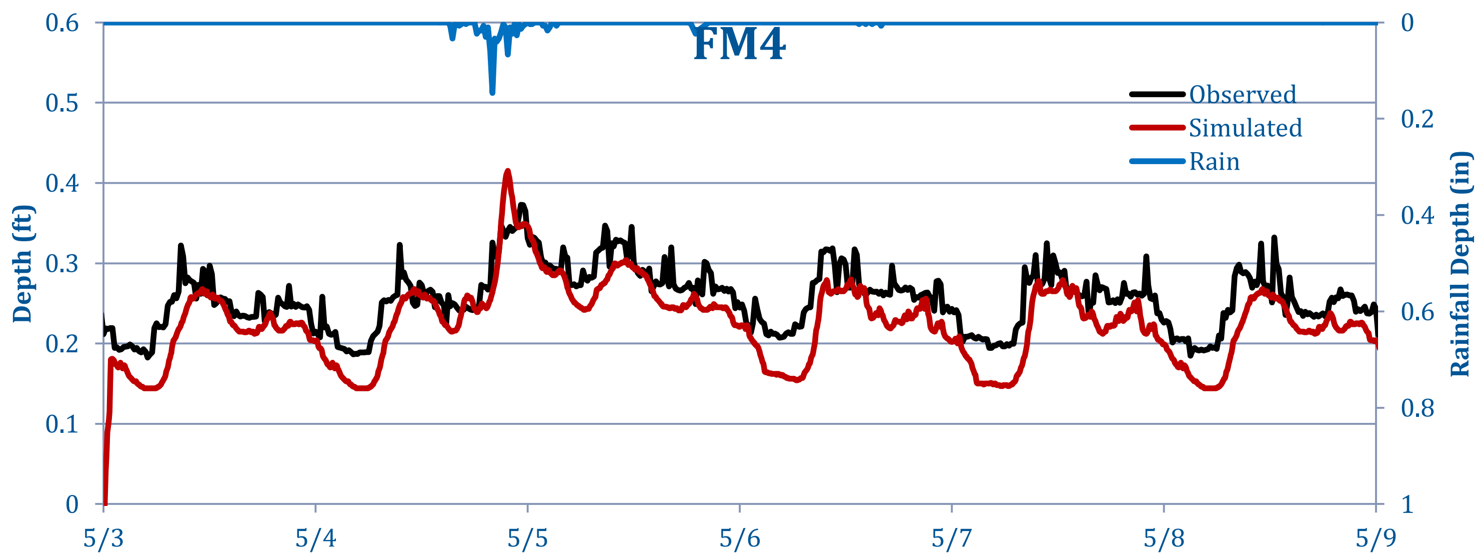

FM3
Observed
Simulated
rain
Depth (ft)
Rainfall Depth (in)
3
2.5
2
1.5
1
0.5
0
0
0.2
0.4
0.6
0.8
1
5/20
5/21
5/22
5/23
5/24

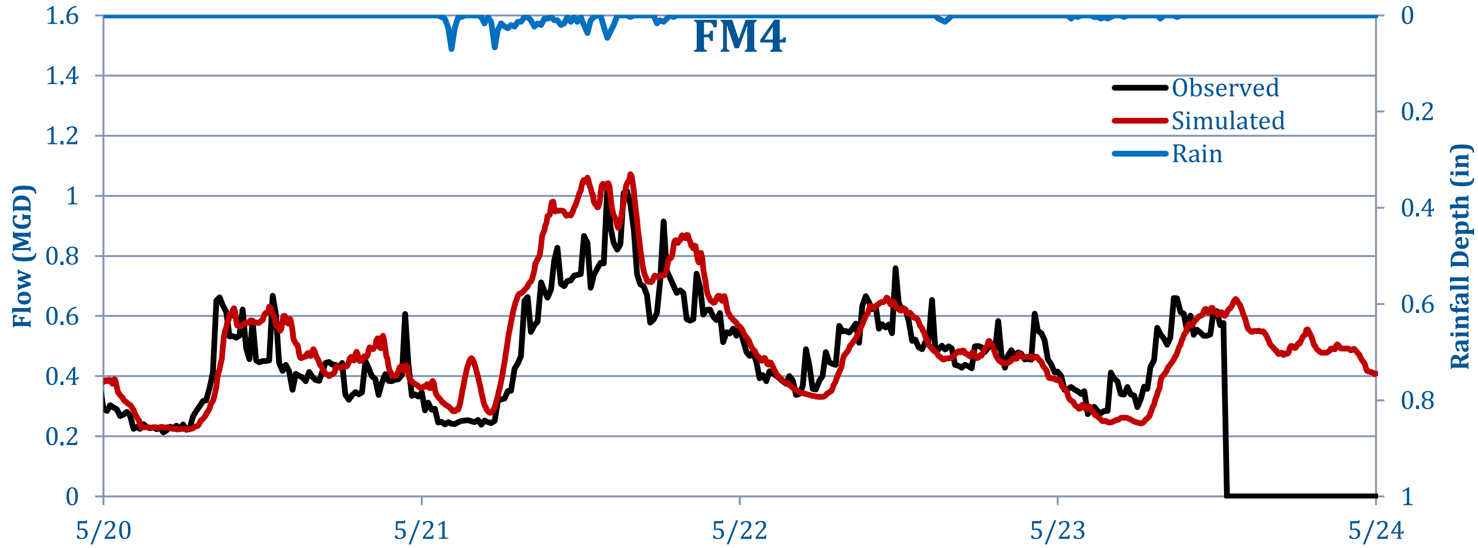
FM-4

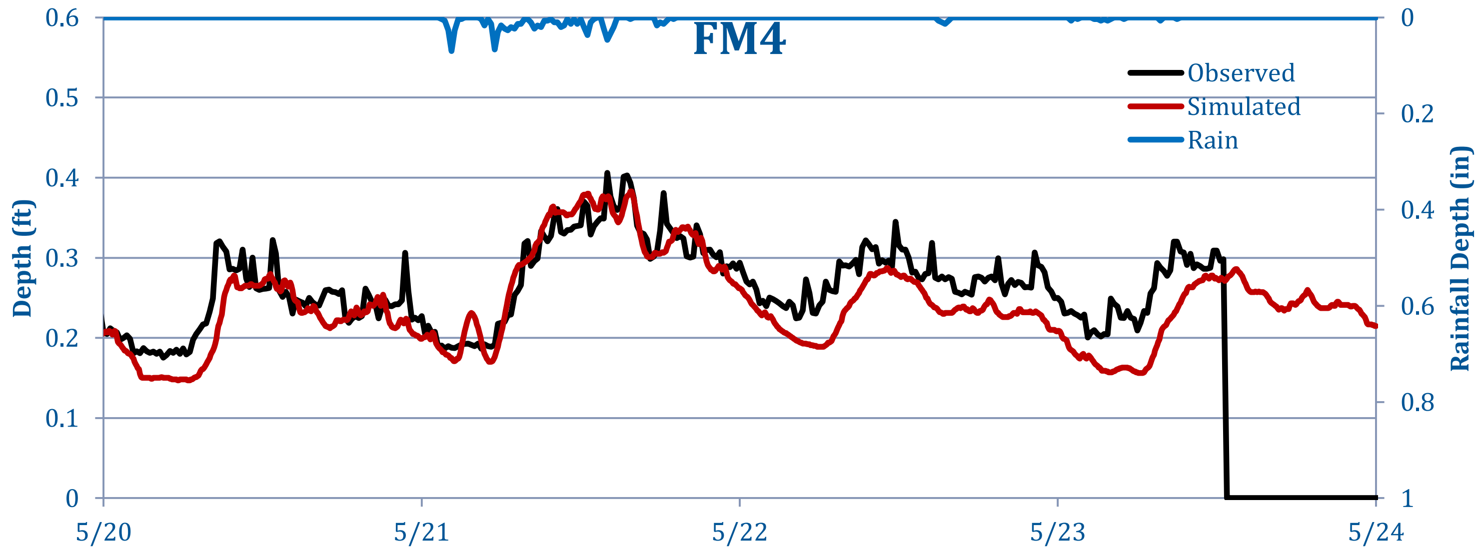

FM4
Observed
Simulated
Rain
Flow (MGD)
Rainfall Depth (in)
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0
0.2
0.4
0.6
0.8
1
3/30
3/31
4/1
4/2
4/3
4/4
4/5
4/6
4/7

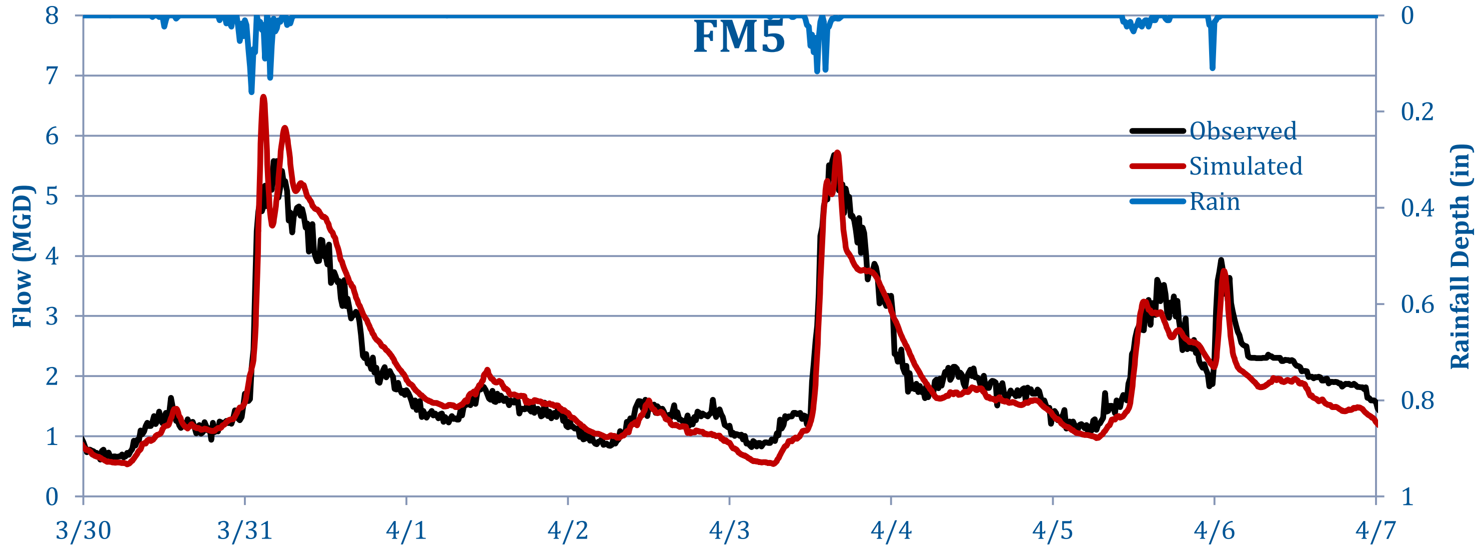

FM4
Observed
Simulated
Rain
Depth (ft)
Rainfall Depth (in)
0.6
0.5
0.4
0.3
0.2
0.1
0
0
0.2
0.4
0.6
0.8
1
3/29
3/30
3/31
4/1
4/2
4/3
4/4
4/5
4/6
4/7

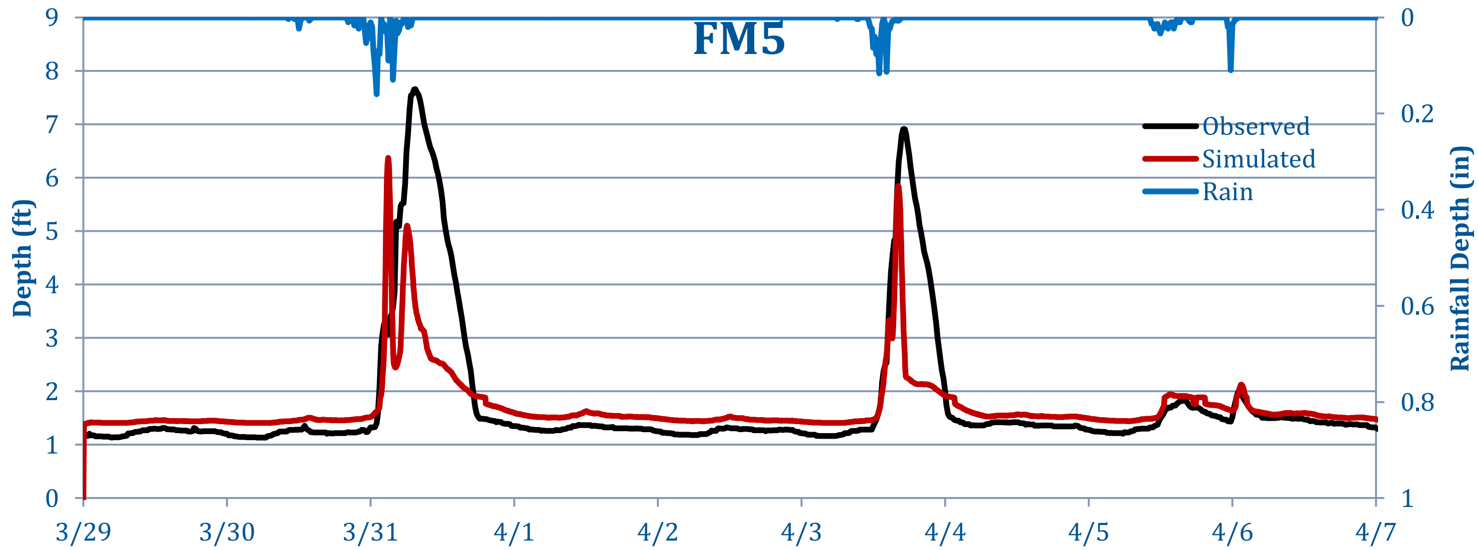

FM4
Observed
Simulated
Rain
Flow (MGD)
Rainfall Depth (in)
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0
0.2
0.4
0.6
0.8
1
5/3
5/4
5/5
5/6
5/7
5/8
5/9

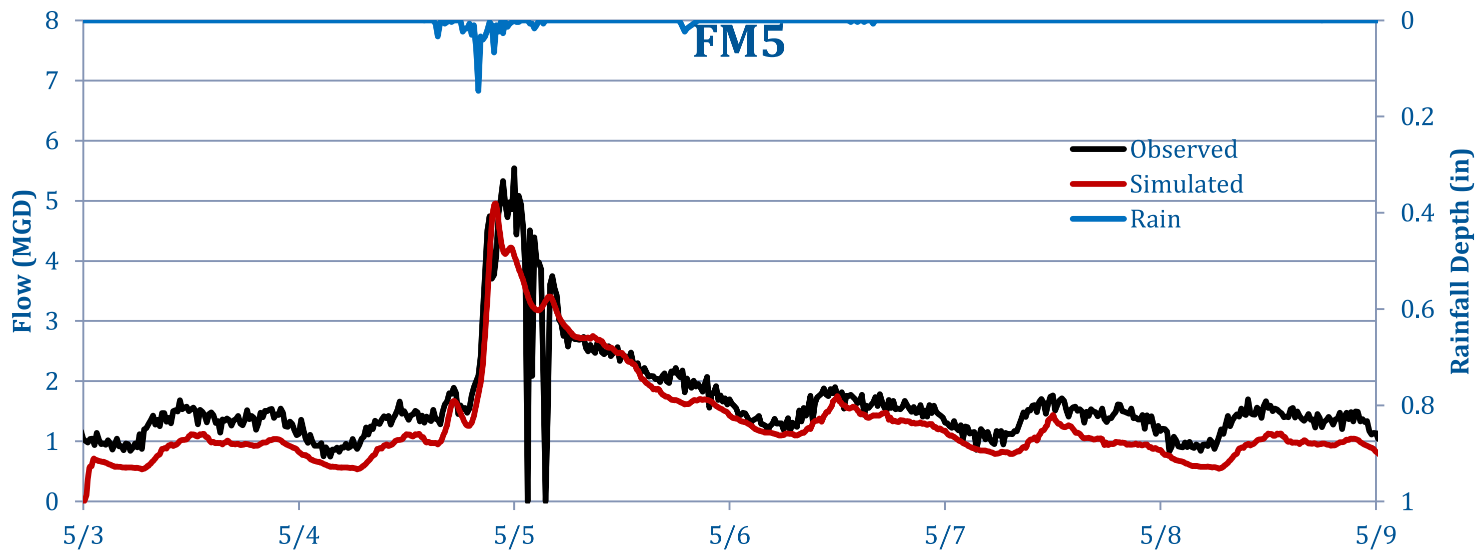

FM4
Observed
Simulated
Rain
Depth (ft)
Rainfall Depth (in)
0.6
0.5
0.4
0.3
0.2
0.1
0
0
0.2
0.4
0.6
0.8
1
5/3
5/4
5/5
5/6
5/7
5/8
5/9

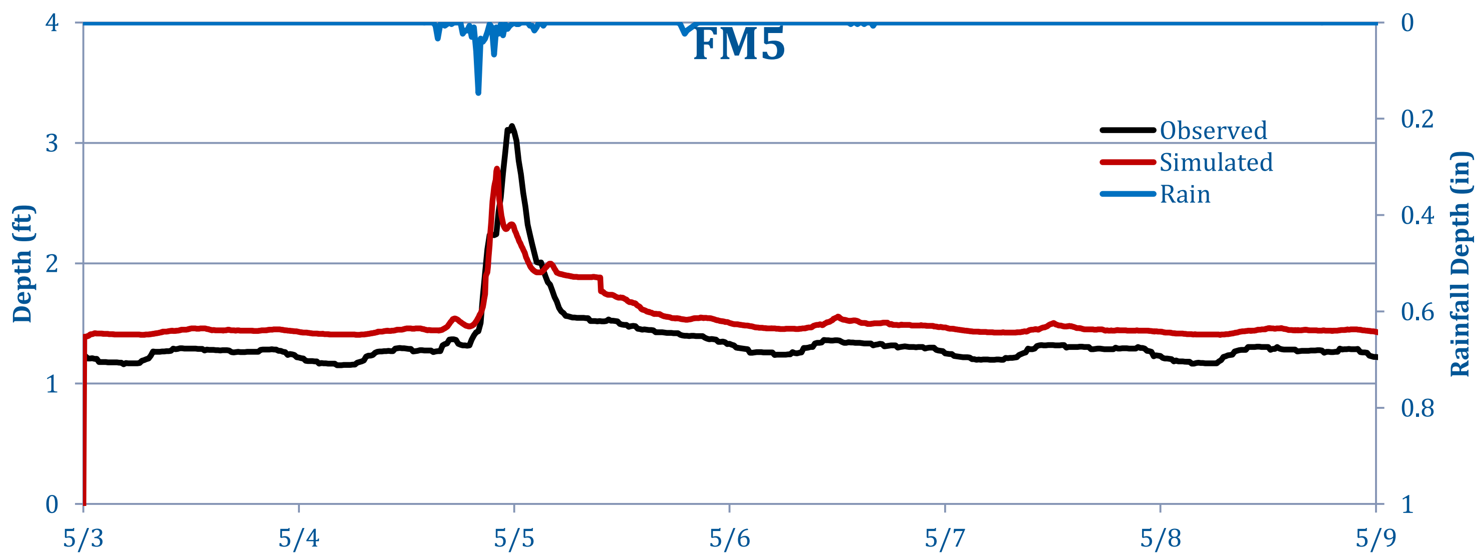

FM4
Observed
Simulated
Rain
Flow (MGD)
Rainfall Depth (in)
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0
0.2
0.4
0.6
0.8
1
5/20
5/21
5/22
5/23
5/24

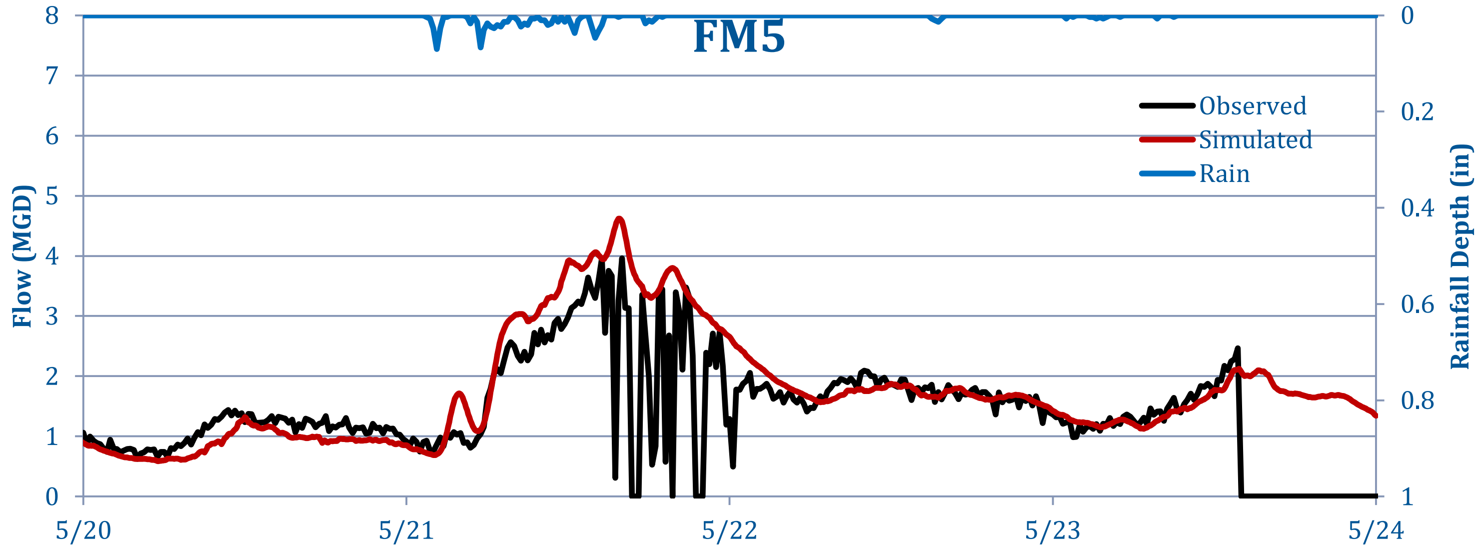

FM4
Observed
Simulated
Rain
Depth (ft)
Rainfall Depth (in)
0.6
0.5
0.4
0.3
0.2
0.1
0
0
0.2
0.4
0.6
0.8
1
5/20
5/21
5/22
5/23
5/24

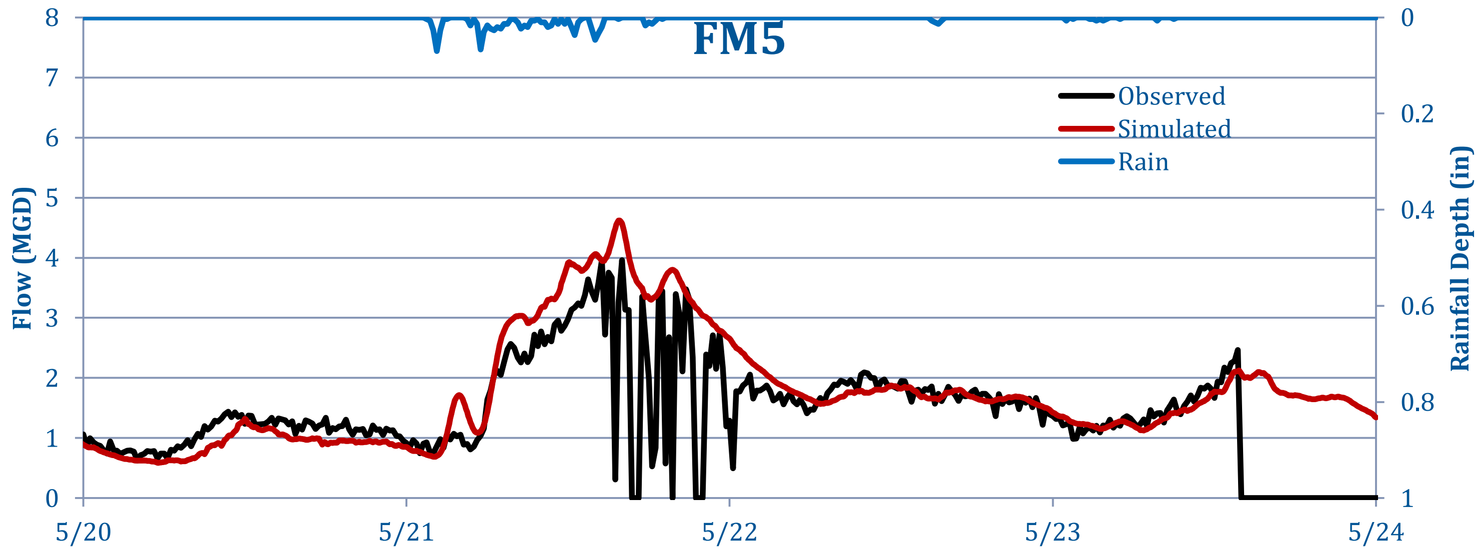

FM5
Flow (MGD)
Rainfall Depth (in)
Observed
Simulated
Rain
0
1
2
3
4
5
6
7
8
0
0.2
0.4
0.6
0.8
1
3/30
3/31
4/1
4/2
4/3
4/4
4/5
4/6
4/7

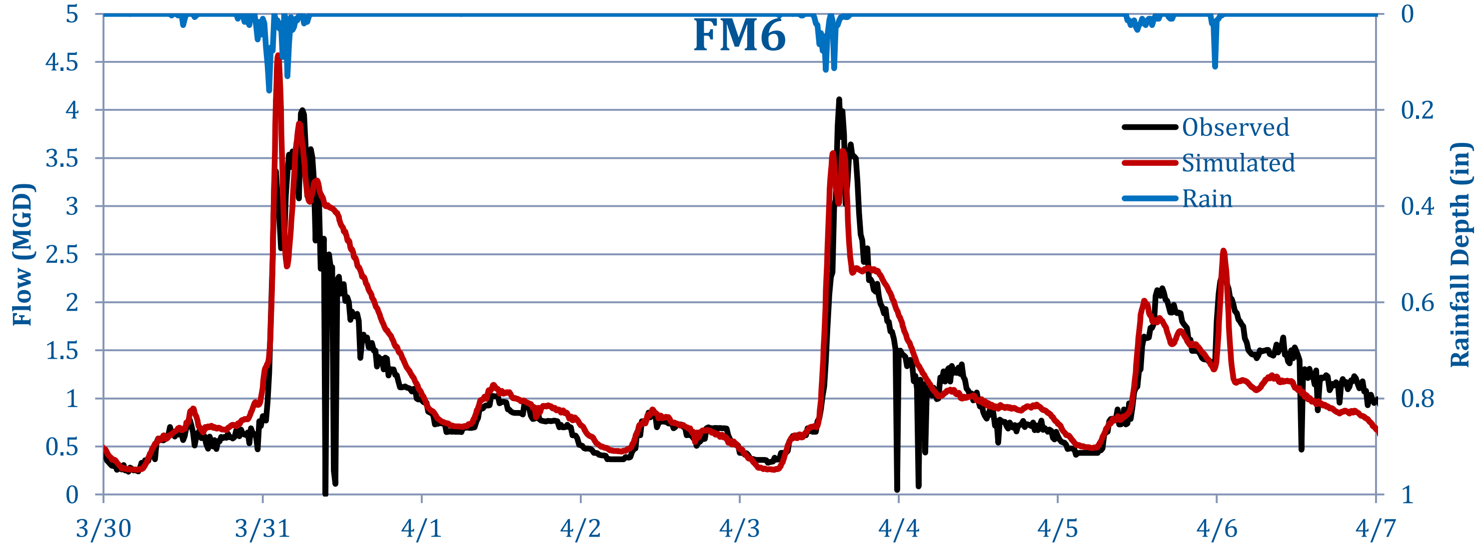

FM5
Depth (ft)
Rainfall Depth (in)
Observed
Simulated
Rain
0
1
2
3
4
5
6
7
8
9
0
0.2
0.4
0.6
0.8
1
3/29
3/30
3/31
4/1
4/2
4/3
4/4
4/5
4/6
4/7

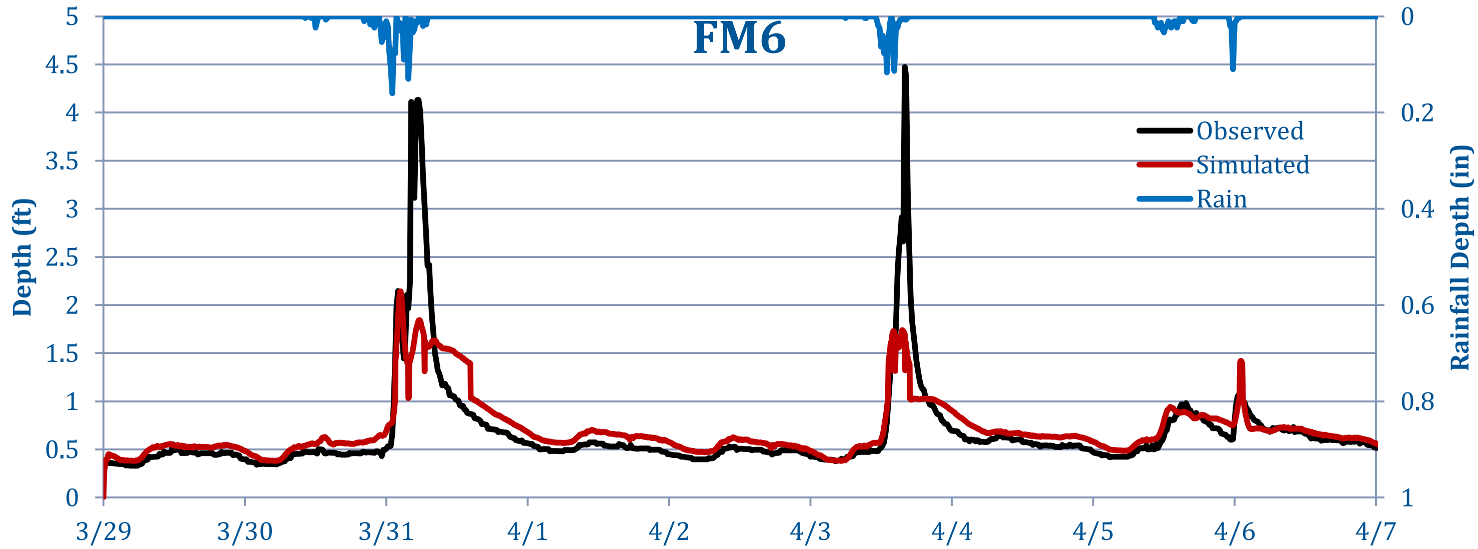

FM5
Flow (MGD)
Rainfall Depth (in)
Observed
Simulated
Rain
8
7
6
5
4
3
2
1
0
0
0.2
0.4
0.6
0.8
1
5/3
5/4
5/5
5/6
5/7
5/8
5/9

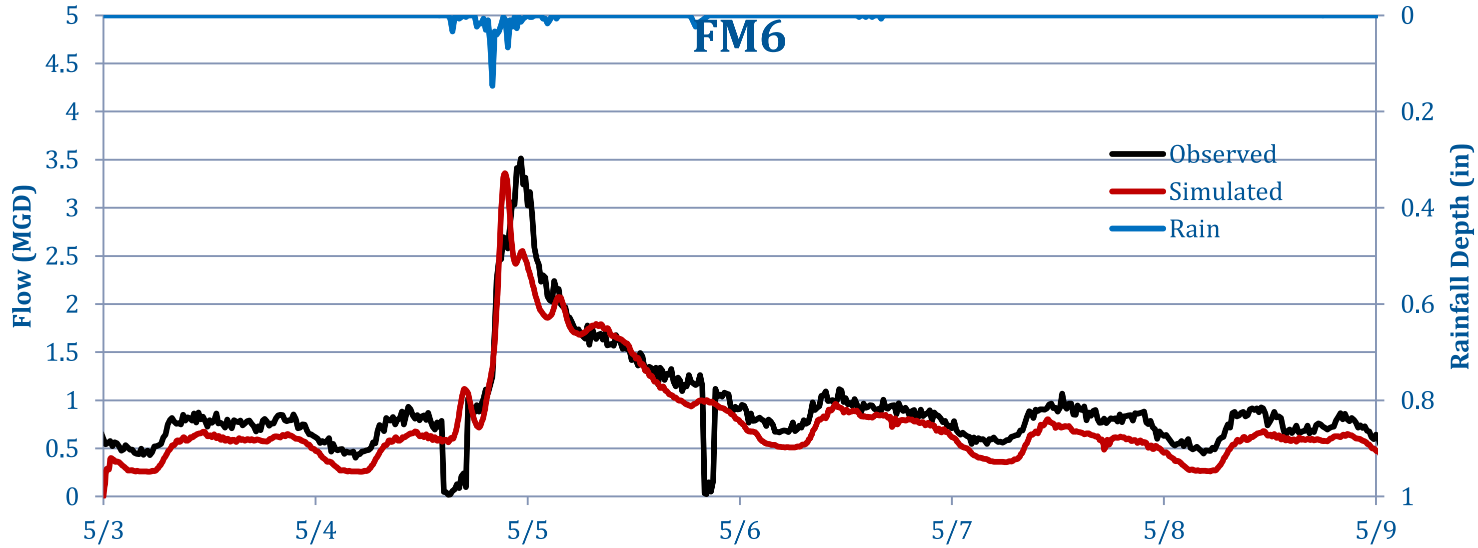

FM5
Depth (ft)
Rainfall Depth (in)
Observed
Simulated
Rain
4
3
2
1
0
0
0.2
0.4
0.6
0.8
1
5/3
5/4
5/5
5/6
5/7
5/8
5/9

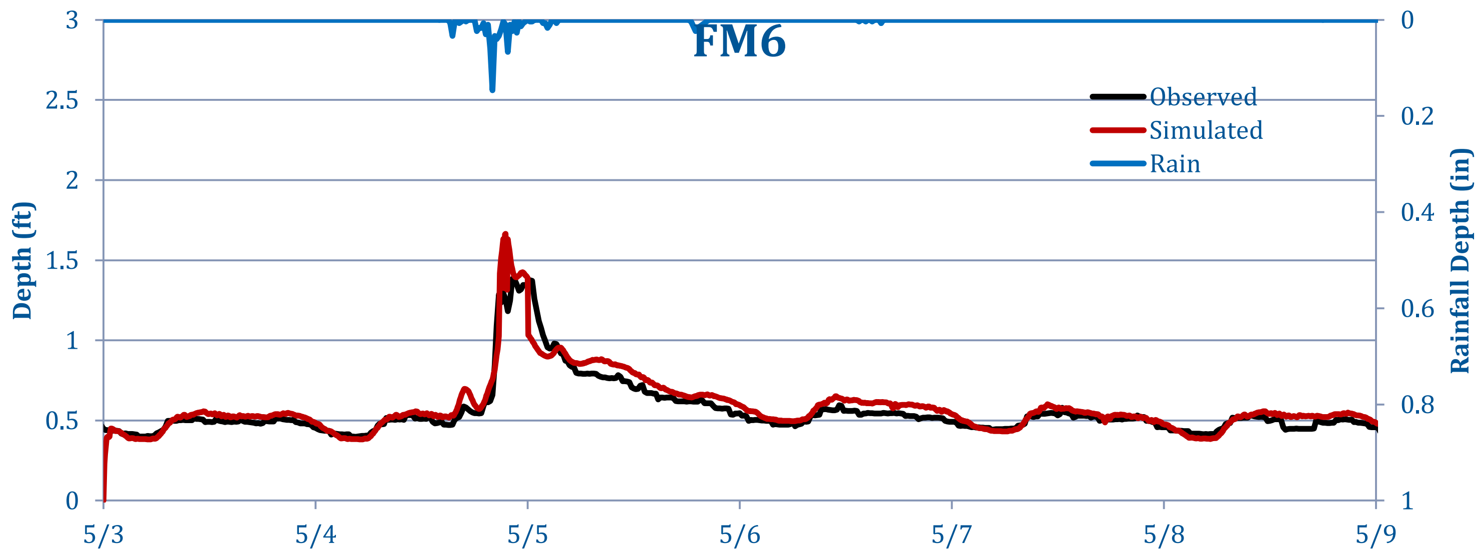

FM5
Observed
Simulated
Rain
Flow (MGD)
Rainfall Depth (in)
8
7
6
5
4
3
2
1
0
0
0.2
0.4
0.6
0.8
1
5/20
5/21
5/22
5/23
5/24

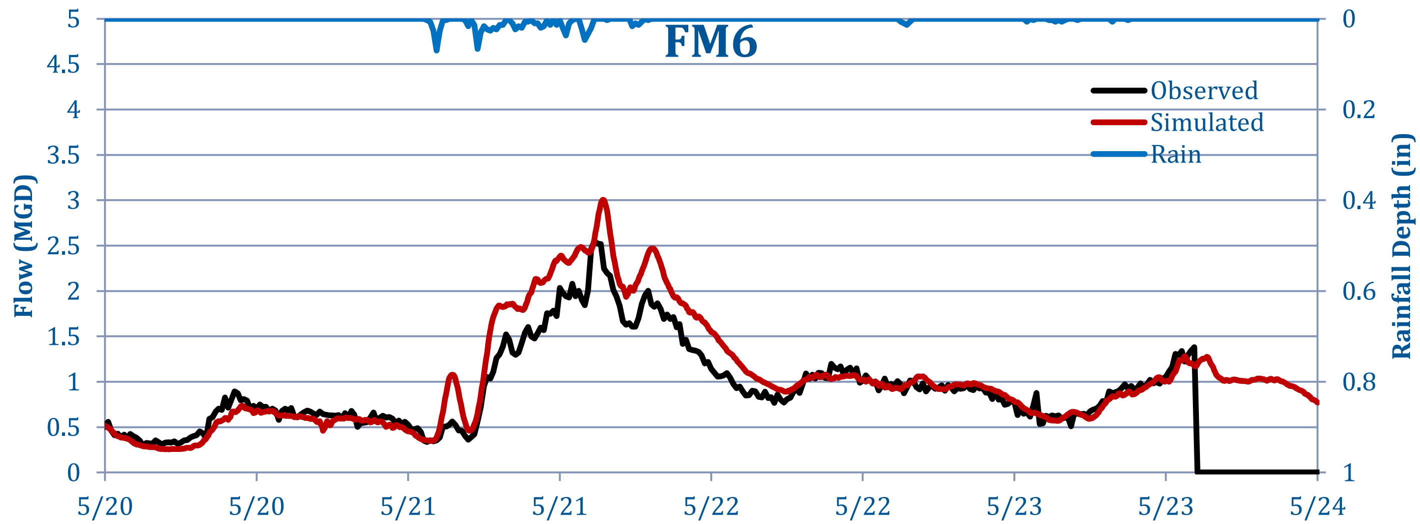

FM5
Observed
Simulated
Rain
Flow (MGD)
Rainfall Depth (in)
8
7
6
5
4
3
2
1
0
0
0.2
0.4
0.6
0.8
1
5/20
5/21
5/22
5/23
5/24

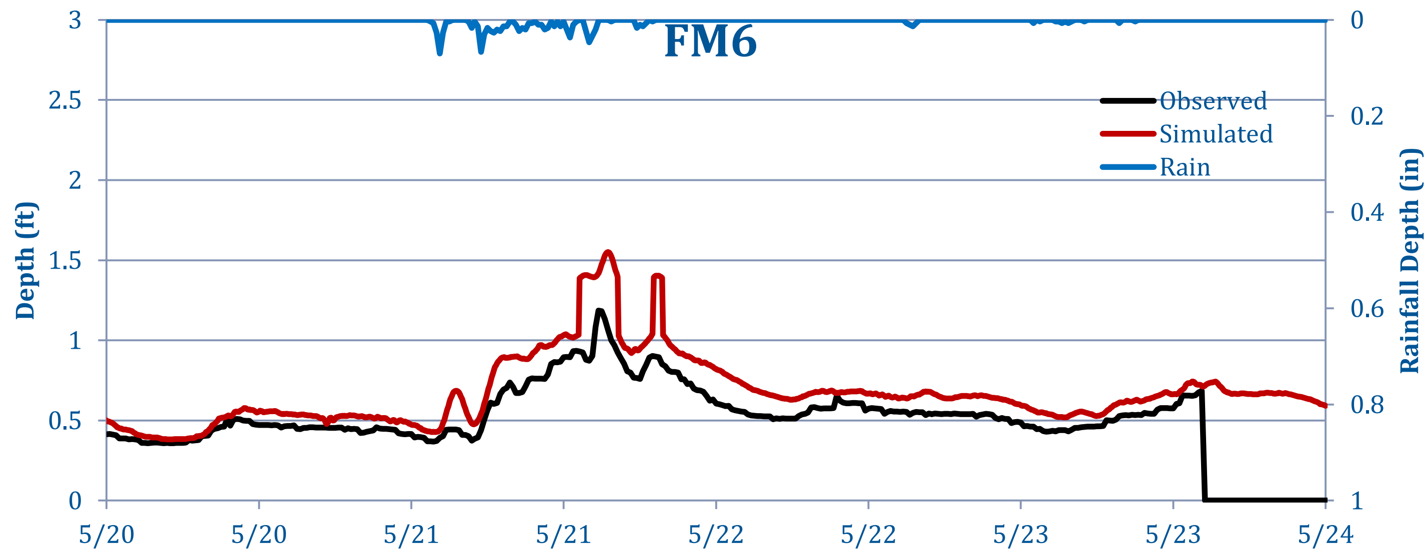

FM6
Flow (MGD)
Rainfall Depth (in)
Observed
Simulated
Rain
5
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
0
0.2
0.4
0.6
0.8
1
3/30
3/31
4/1
4/2
4/3
4/4
4/5
4/6
4/7

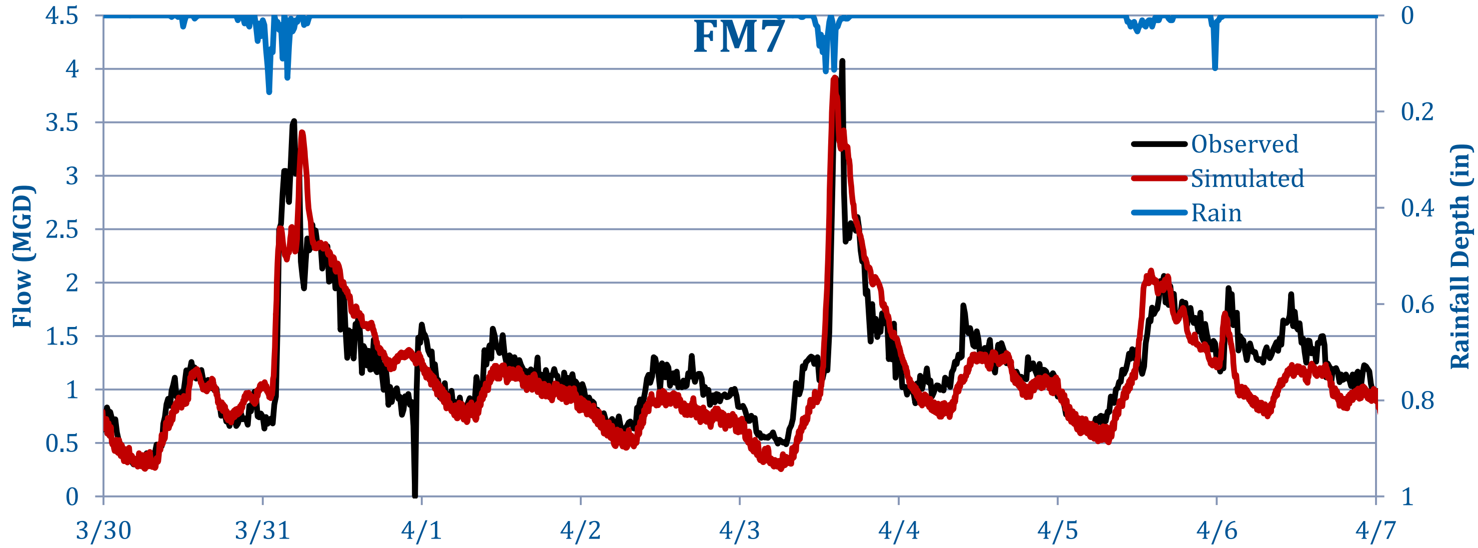

FM6
Depth (ft)
Rainfall Depth (in)
Observed
Simulated
Rain
5
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
0
0.2
0.4
0.6
0.8
1
3/29
3/30
3/31
4/1
4/2
4/3
4/4
4/5
4/6
4/7

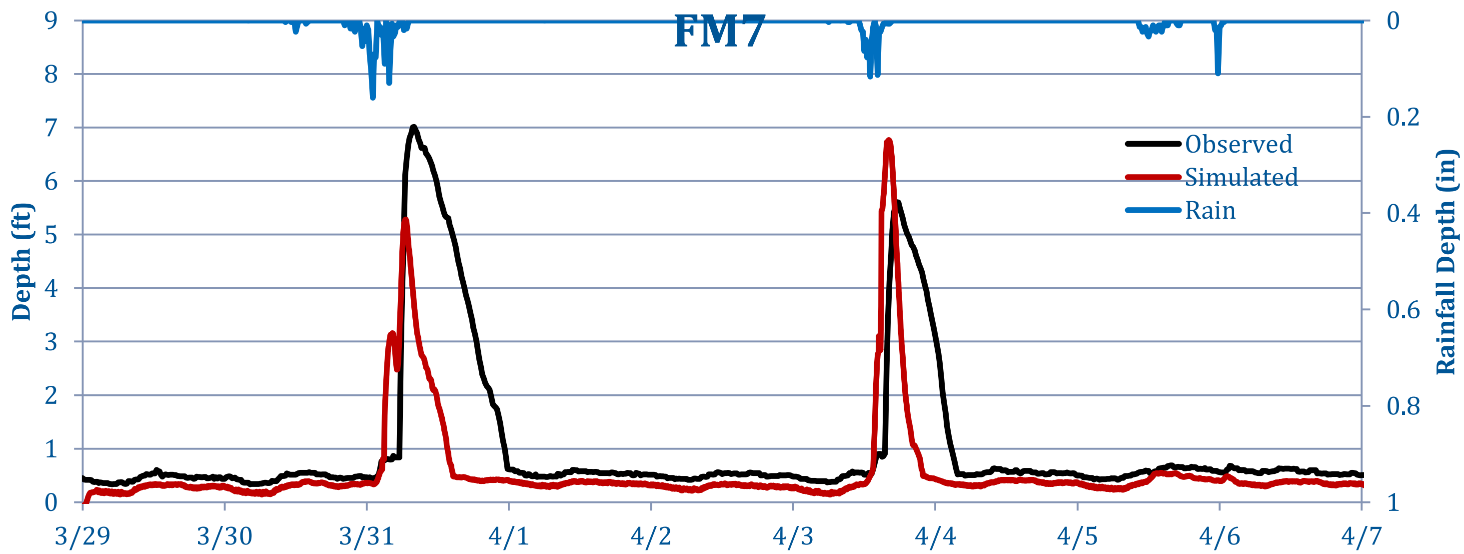

FM6
Flow (MGD)
5
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
Rainfall Depth (in)
0
0.2
0.4
0.6
0.8
1
5/3
5/4
5/5
5/6
5/7
5/8
5/9
Observed
Simulated
Rain

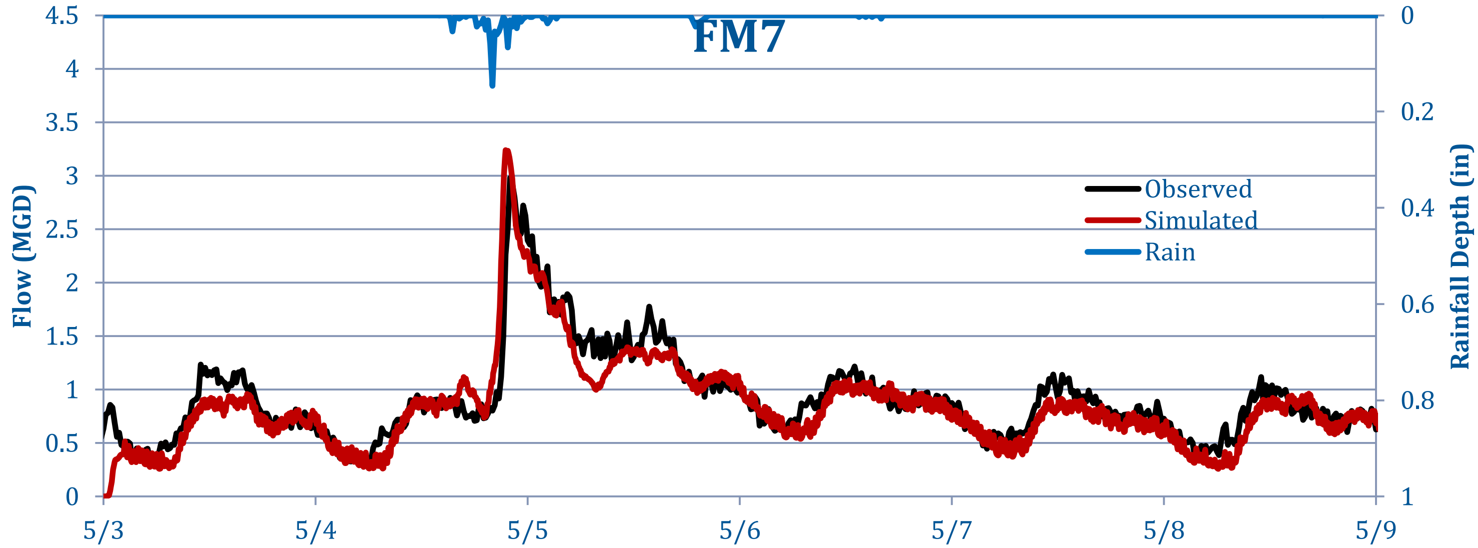

FM6
Depth (ft)
3
2.5
2
1.5
1
0.5
0
Rainfall Depth (in)
0
0.2
0.4
0.6
0.8
1
5/3
5/4
5/5
5/6
5/7
5/8
5/9
Observed
Simulated
Rain

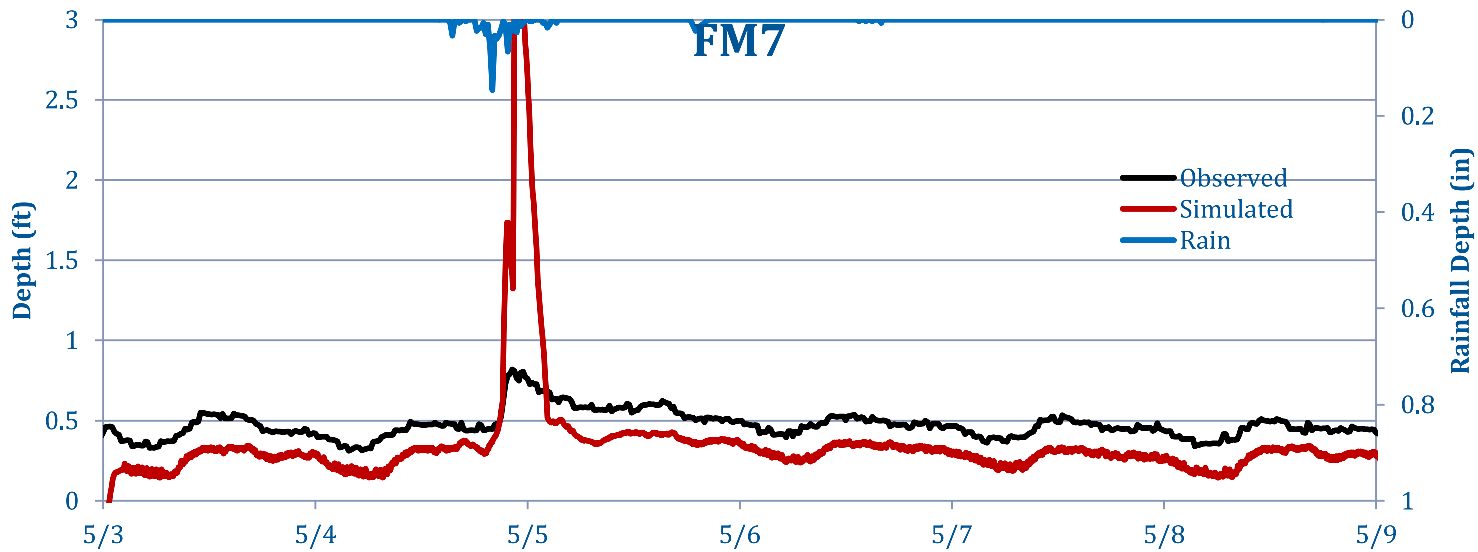

FM6
Flow (MGD)
Rainfall Depth (in)
Observed
Simulated
Rain
5
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
0
0.2
0.4
0.6
0.8
1
5/20
5/20
5/21
5/21
5/22
5/22
5/23
5/23
5/24

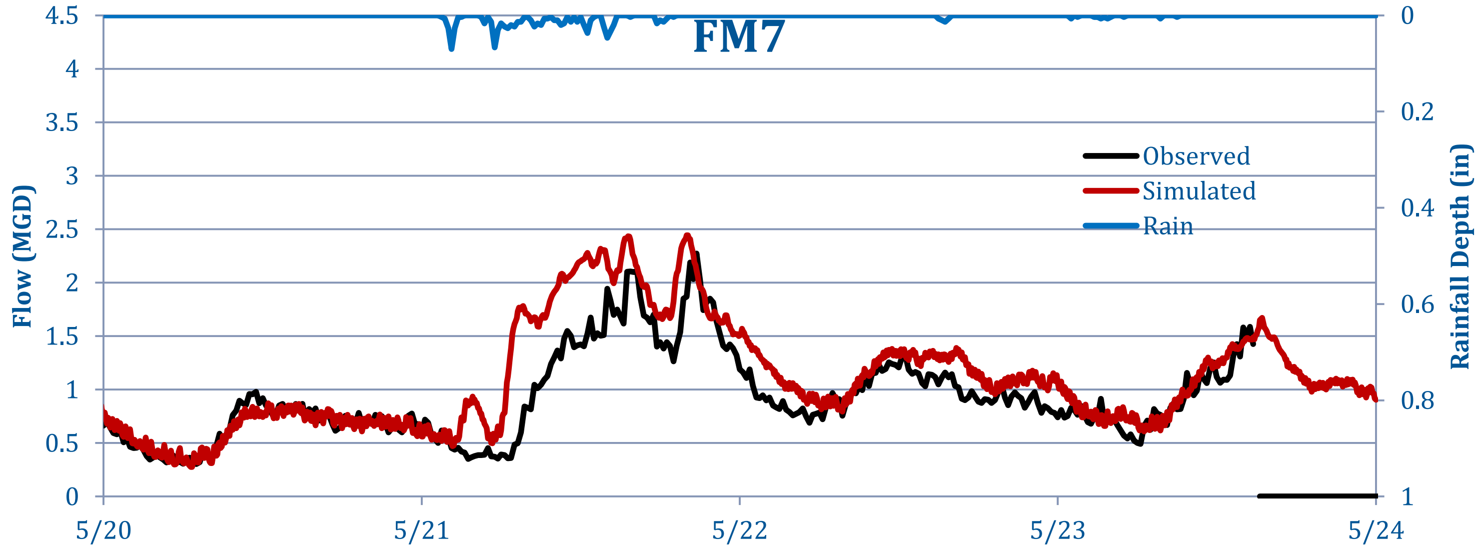

FM6
Depth (ft)
Rainfall Depth (in)
Observed
Simulated
Rain
3
2.5
2
1.5
1
0.5
0
0
0.2
0.4
0.6
0.8
1
5/20
5/20
5/21
5/21
5/22
5/22
5/23
5/23
5/24

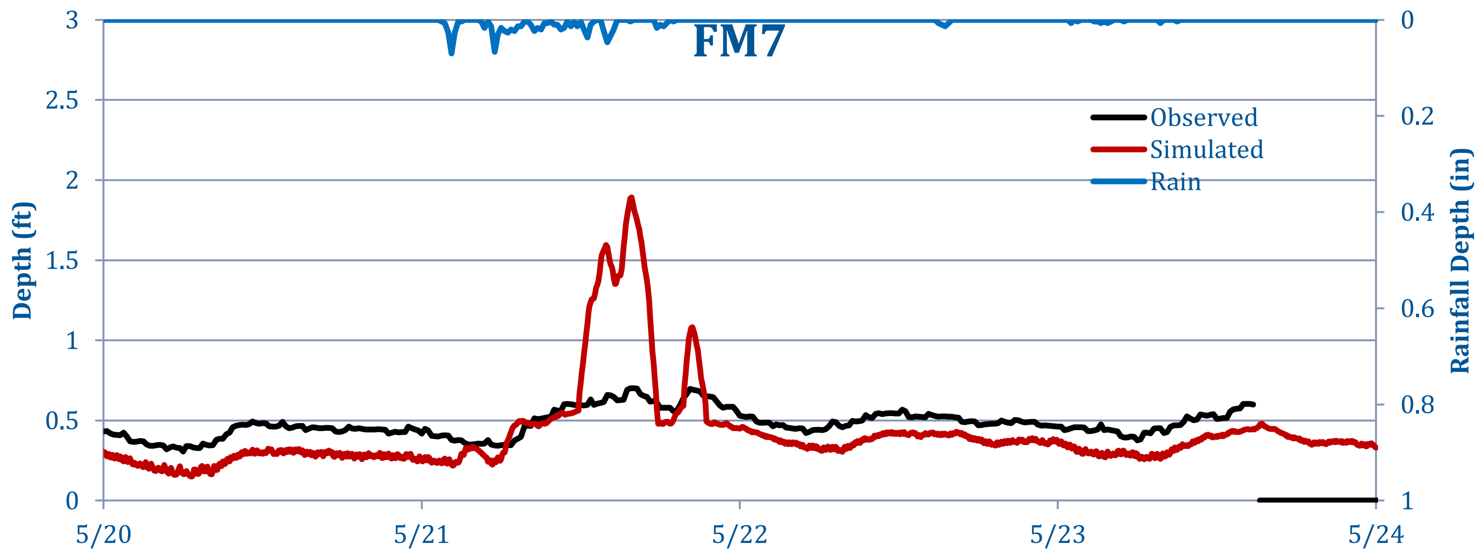

FM7
Flow (MGD)
Rainfall Depth (in)
Observed
Simulated
Rain
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
0
0.2
0.4
0.6
0.8
1
3/30
3/31
4/1
4/2
4/3
4/4
4/5
4/6
4/7


FM7
Depth (ft)
Rainfall Depth (in)
Observed
Simulated
Rain
9
8
7
6
5
4
3
2
1
0
0
0.2
0.4
0.6
0.8
1
3/29
3/30
3/31
4/1
4/2
4/3
4/4
4/5
4/6
4/7

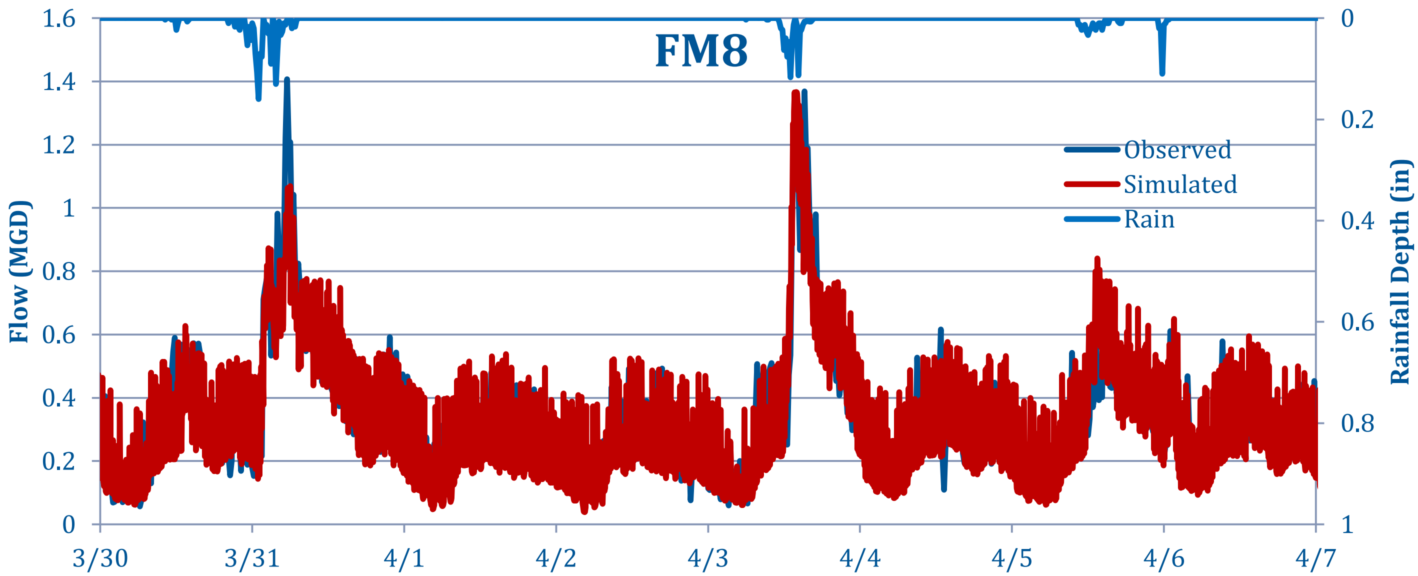

FM7
Flow (MGD)
Rainfall Depth (in)
Observed
Simulated
Rain
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
0
0.2
0.4
0.6
0.8
1
5/3
5/4
5/5
5/6
5/7
5/8
5/9

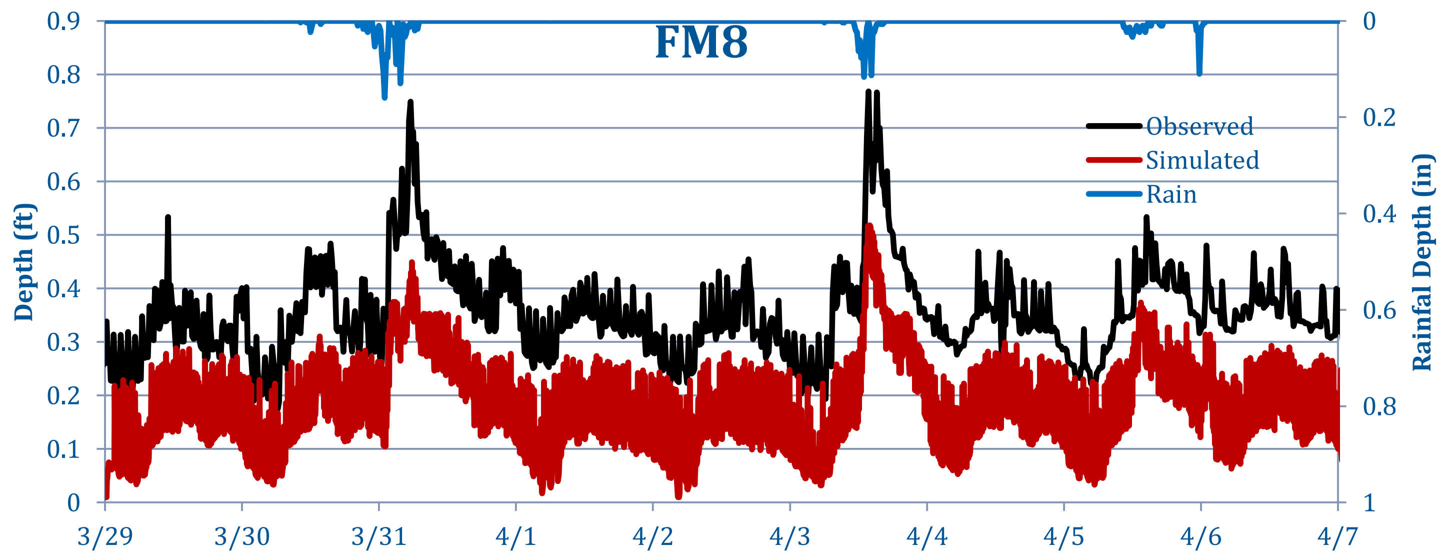

FM7
Depth (ft)
Rainfall Depth (in)
Observed
Simulated
Rain
3
2.5
2
1.5
1
0.5
0
0
0.2
0.4
0.6
0.8
1
5/3
5/4
5/5
5/6
5/7
5/8
5/9

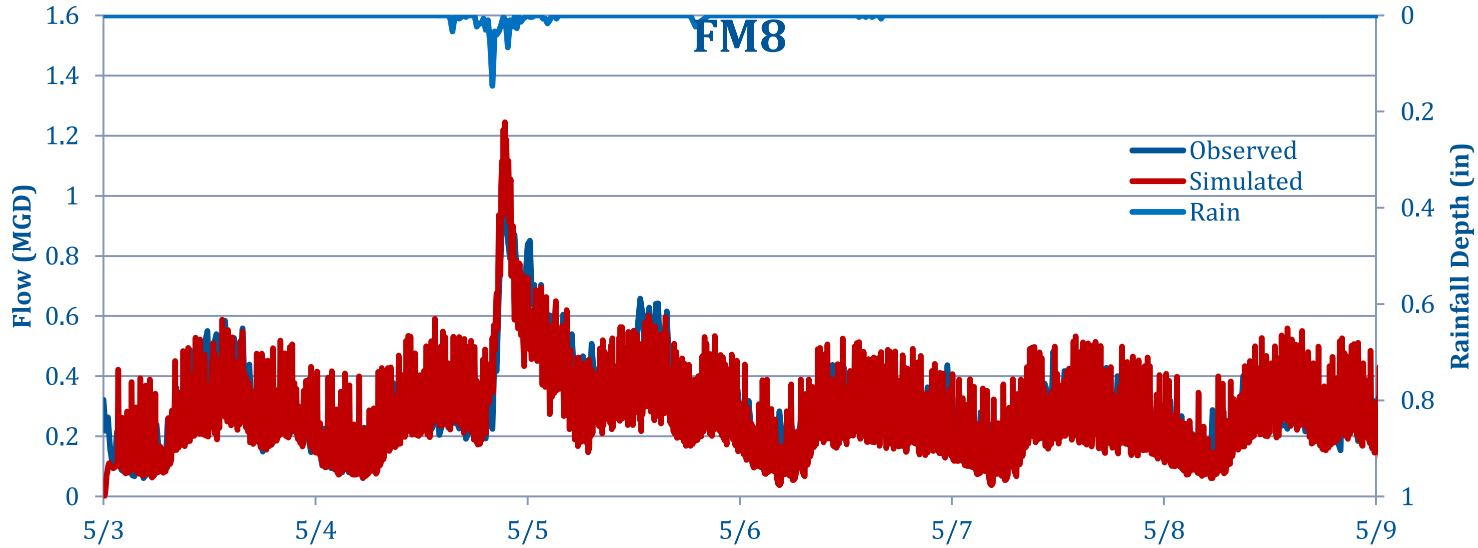

FM7
Flow (MGD)
Rainfall Depth (in)
Observed
Simulated
Rain
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
0
0.2
0.4
0.6
0.8
1
5/20
5/21
5/22
5/23
5/24

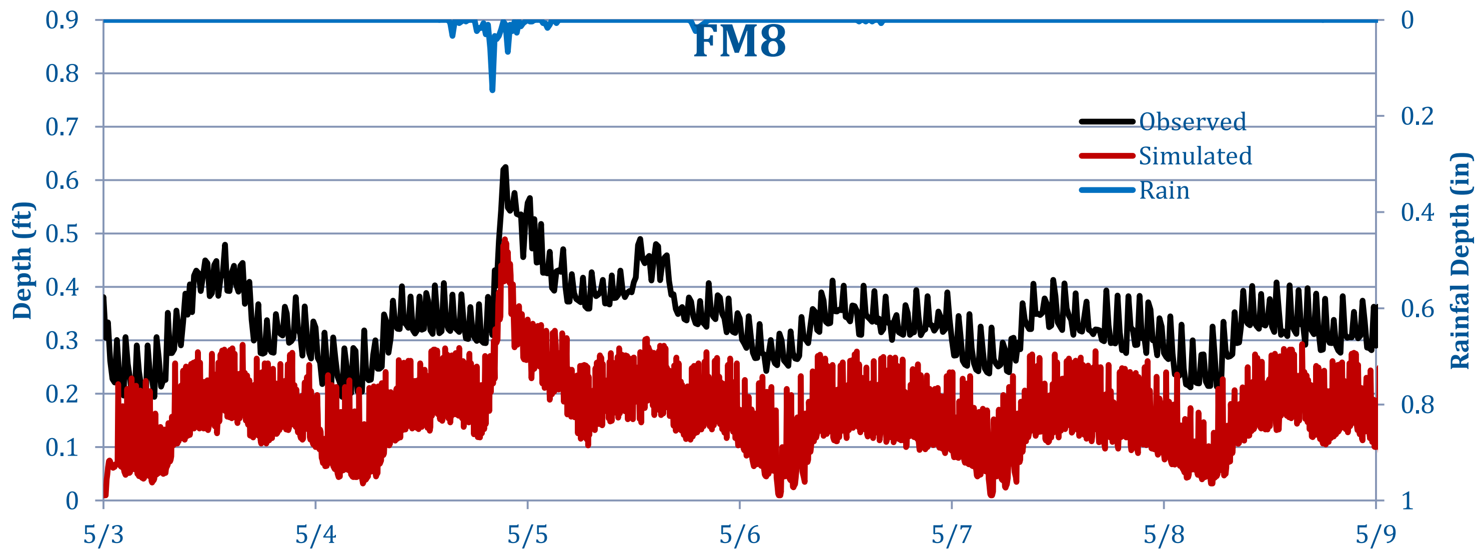

FM7
Depth (ft)
Rainfall Depth (in)
Observed
Simulated
Rain
3
2.5
2
1.5
1
0.5
0
0
0.2
0.4
0.6
0.8
1
5/20
5/21
5/22
5/23
5/24

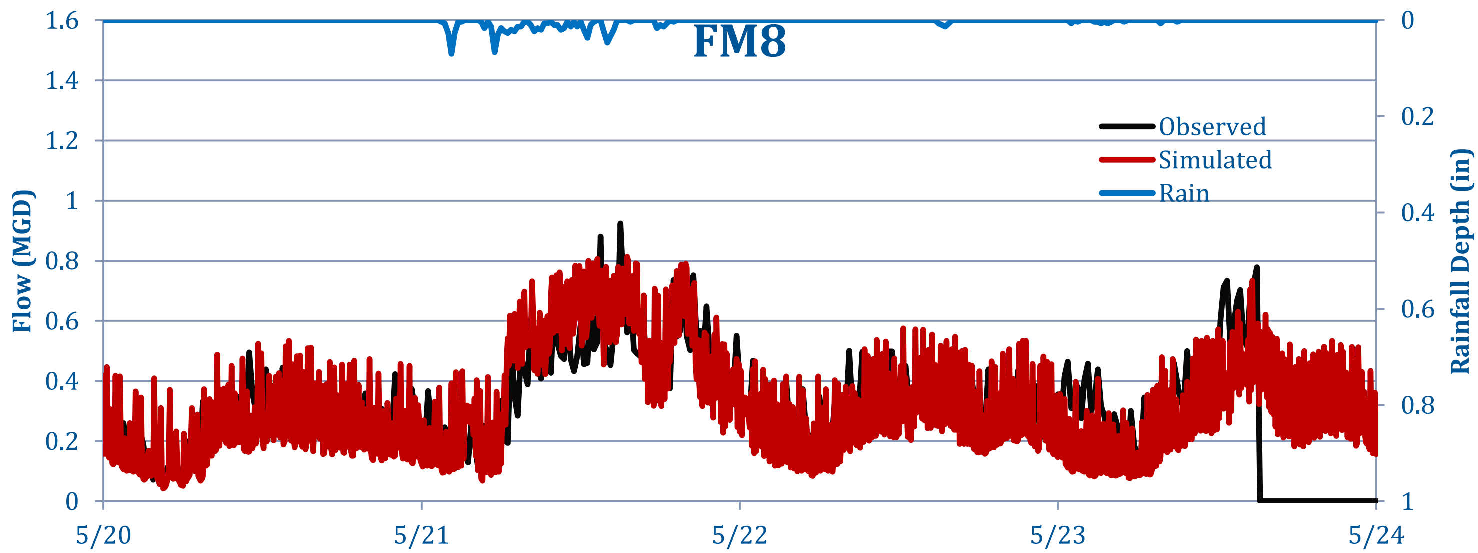
FM8

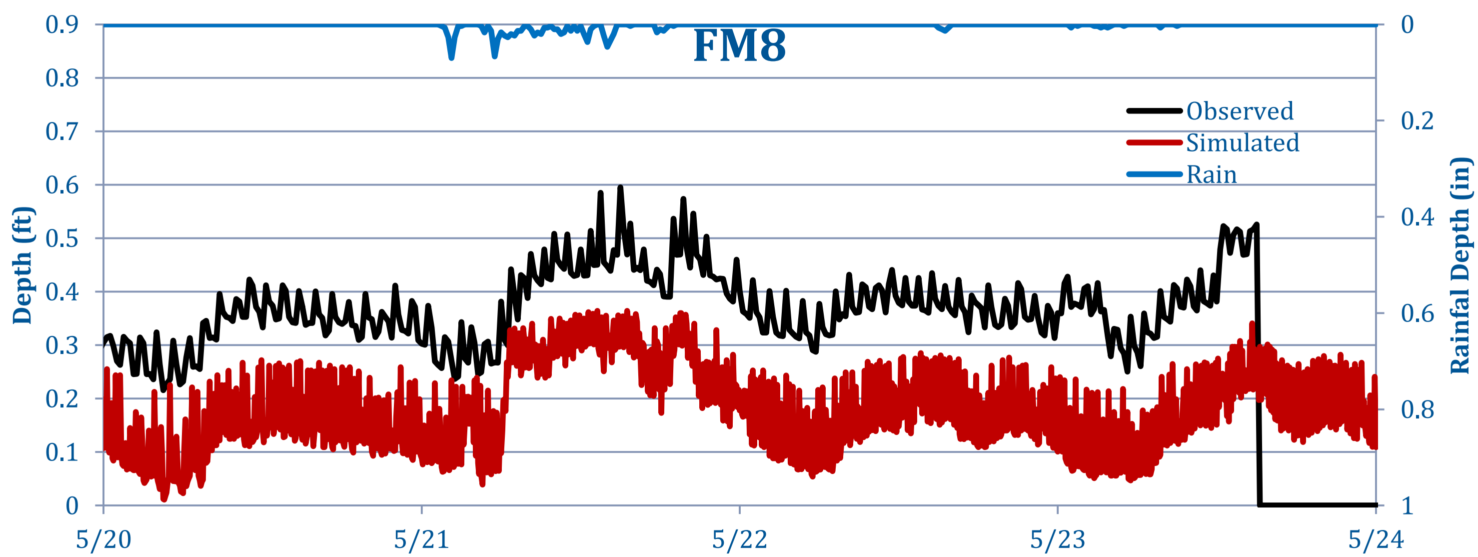

FM8
Flow (MGD)
Rainfall Depth (in)
Observed
Simulated
Rain
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0.2
0.4
0.6
0.8
1
3/30
3/31
4/1
4/2
4/3
4/4
4/5
4/6
4/7


FM8
Depth (ft)
Rainfal Depth (in)
Observed
Simulated
Rain
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0.2
0.4
0.6
0.8
1
3/29
3/30
3/31
4/1
4/2
4/3
4/4
4/5
4/6
4/7


FM8
Flow (MGD)
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
Rainfall Depth (in)
0
0.2
0.4
0.6
0.8
1
Observed
Simulated
Rain
5/3
5/4
5/5
5/6
5/7
5/8
5/9


FM8
Depth (ft)
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
Rainfal Depth (in)
0
0.2
0.4
0.6
0.8
1
Observed
Simulated
Rain
5/3
5/4
5/5
5/6
5/7
5/8
5/9


FM8
Flow (MGD)
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
Rainfall Depth (in)
0
0.2
0.4
0.6
0.8
1
5/20
5/21
5/22
5/23
5/24
Observed
Simulated
Rain


FM8
Depth (ft)
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
Rainfall Depth (in)
0
0.2
0.4
0.6
0.8
1
5/20
5/21
5/22
5/23
5/24
Observed
Simulated
Rain

Appendix E - Smoke Testing Inspection Plan

1.0 Background

The condition assessment work in Phase 1 consists of smoke testing of 20,000 feet of sewer pipe. The Implementation of this inspection plan will provide the basis for the initial assessment of the condition of these pipelines and, based on these results, more detailed inspection will be recommended for Phase 2. This inspection will provide information that can be used to determine the locations and pipe material testing for Phase 2.

The gravity sewer network within the collection system is comprised of various pipe material and sizes as shown in Figure 1. The pipe material includes ductile/cast iron, clay, santite pipe and PVC. A large quantity of the pipe material is unknown which limits the use of pipe material as a factor in selecting locations. The gravity collection system is composed of pipe ranging in diameter from 4-inch to 42-inch and the length of pipe based on the GIS information by diameter and material is shown in Table 1.

The forcemains are not included in this work but should be inspected within the next 5 years to document their condition and determine if repair and replacement is required as part of the capital plan.

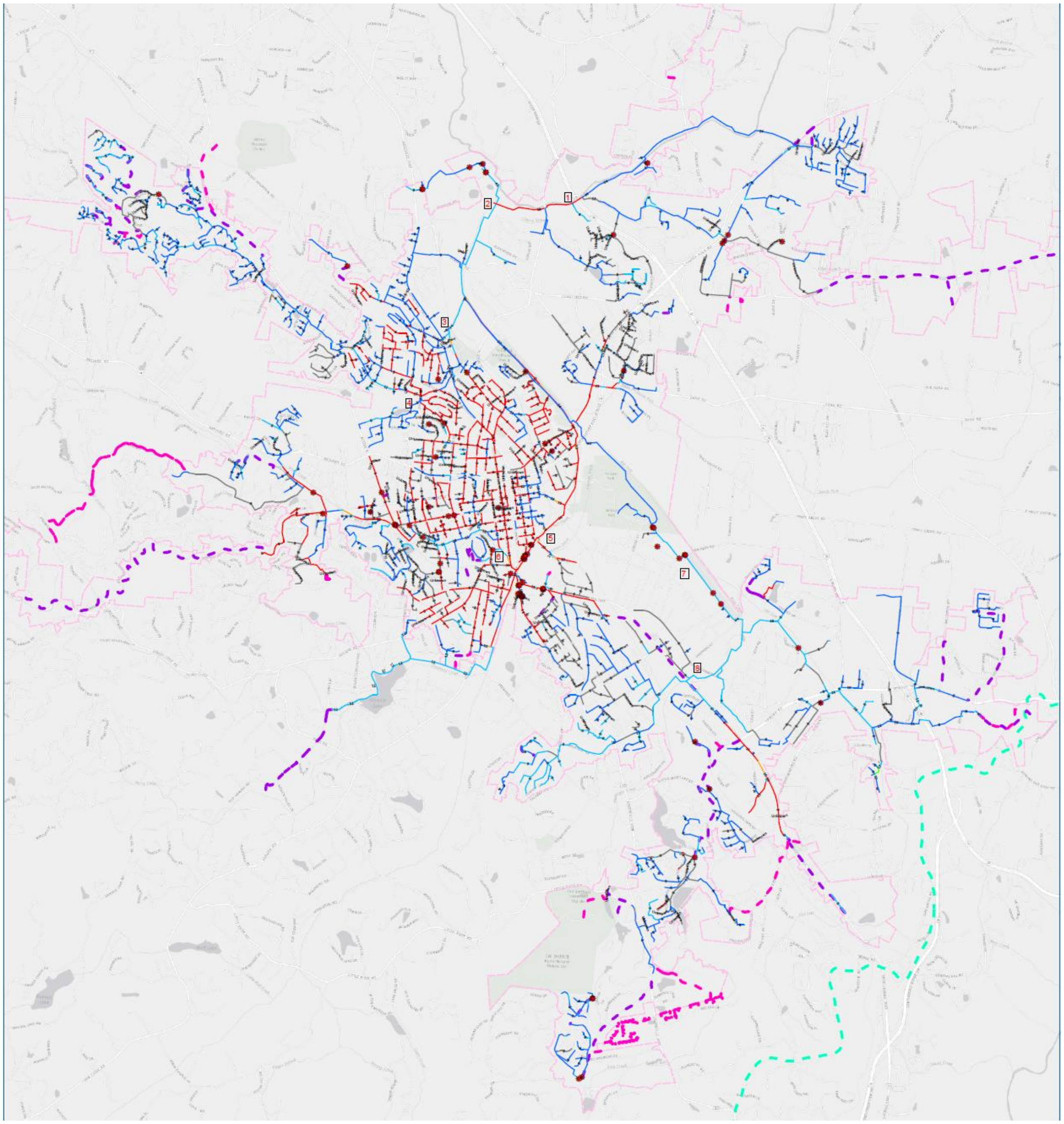


Figure 1 – Hendersonville Collection System

Table 1 – Summary of Collection System Piping

MATERIALS / DIAMETER	4	6	8	10	12	15	18	24	27	30	36	42	UNKNOWN	(BLANK)	TOTAL
Blank	183	524	202											12,419	13,328
Clay		24,773	106,507	13,699	11,969	8,258	5,894	8,131	3,028			2,077	365		184,700
Clay/DIP			469	605	789	817	691		1,132	201					4,704
DIP		657	67,338	3,964	4,769	1,365	16,022	8,146	16	27	28	4,062	1,398		107,793
Other	374														374
PVC	71	5,151	280,176	14,920	13,473	2,958	2,539	10,802					1,661		331,752
PVC/Clay		319	5,168	1,501	2,482	173	224		945						10,812
PVC/DIP			5,312	410	1,746	361									7,829
Sanitite HP					481										481
Unknown	26	4,406	97,906	962	1,229	648	1,140						66,276	188	172,781
Total	**654**	**35,830**	**563,078**	**36,062**	**36,939**	**14,580**	**26,510**	**27,079**	**5,121**	**228**	**28**	**6,140**	**69,700**	**12,606**	**834,555**

2.0 Identification of Inspection Areas

The inspection will be limited to 20,000 feet of smoke testing. These inspections will only cover a small percentage of the total pipelines in the system and do not include pipe from all areas of the City.

The results of these inspections will be used to estimate inflow and infiltration (I/I), and the location of possible blockages and structural defects so the overall condition assessment of the entire basin can be estimated. More detailed inspections will be required to develop specific capital projects but these inspections will provide useful information in development of a plan for future work. The factors used to select the segments to inspect include previous smoke testing, recent flow metering results, locations of SSO events, creek crossing, and experience with the various pipe materials.

3.0 Coordination of the Work

Black & Veatch (B&V) will be responsible for the engineering support required for the inspection and for providing on-site observation of the inspection to convey to the City regular updates on the progress. B&V will coordinate the work of the subcontractor, Frazier Engineering.

During the smoke testing, any preliminary findings of significant defects, blockages or significant cross connection that could impact operations will be reported to the City. During the inspection any manholes identified that are not shown in the current GIS information will be documented and reported. The subcontractor will provide qualified personnel to use the inspection equipment and other tools necessary to perform the work. They will also be responsible for their safety and shall provide the required safety equipment for their workers.

Table 3 provides contact names and numbers for those working on the project. The names for the subcontractor are also provided.

Table 2 – Contact Information for Personnel on Project

Company	Name	Position	Telephone No.	Email Address
Hendersonville	Tim Sexton	Collections Supervisor	(828) 243-3740	tsexton@hvlnc.gov
Hendersonville	Kenneth Page	City Contact	(828) 450-4315	Kpage@hvlnc.gov
Fire Department	D. James Miller	Deputy Fire Chief	(828)697-3024 (O) 828) 674-6339 (C)	
Non-Emergency Dispatch		Fire & Police	(828) 697-4911	
Black & Veatch	Mike Osborne	Project Manager	(704) 510-8451 (704) 575-5558	osbornejm@bv.com
Black & Veatch	Bryon Livingston	Assessment Lead	(913) 458-3368 (816) 729-3546	livingstonb@bv.com
Frazier Engineering	Dan Anderson	Project Manager	(704) 822-8444 (704) 877-3003	danderson@frazier-engineering.com
Frazier Engineering	John Guidone	Field Manager	(704) 202-0178	

3.1 PUBLIC NOTIFICATION AND PERMIT

In the areas affected by the smoke testing the public will be notified through the use of door hangers distributed by Frazier Engineering a couple of days prior to the smoke testing work. Black & Veatch will provide a list of property owners' names and addresses from the GIS data in the area of the inspection for Frazier Engineering to use in contacting the property owners and informing them of the work in the area.

Frazier Engineering will coordinate contact with the local fire and police departments through the non-emergency dispatch to inform them of the work on a daily basis. They will also contact the Deputy Fire Chief directly each day of the inspection as needed.

An example of the door hanger is shown in Figure 2.

Hendersonville Water Sewer

Date of Notice___________________________

ATTENTION CITY OF HENDERSONVILLE CUSTOMERS

Smoke testing of the sanitary sewers in your neighborhood is scheduled to begin in the next few days. The smoke used for these tests is non-toxic, non-staining and not harmful to people or pets. Testing will occur over the next few days and should be completed within a couple of weeks. If additional testing needs to occur after this date, another notice will be provided.

Upon receipt of this notice, please run about a gallon of water from all faucets in your home into all sinks and showers and let it drain out; this will ensure that all plumbing traps have water in them. Also, pour about a gallon of water into any floor drains and if you have any dry toilets, please fill them with water.

Hendersonville Water and Sewer is evaluating the sewer system in your area by performing smoke testing. Smoke testing is a technique where non-toxic smoke is blown into the sewer system, and the smoke escapes through leaks in the sewers. This evaluation work locates leaks in the sewers that allow rainwater, creek water and other storm water to enter the system. Frazier Engineering has been hired to smoke test the sewer system. Over the next few weeks, you will see uniformed crews from Frazier Engineering working along the sewer lines and at manholes.

IMPORTANT INFORMATION:

- **Smoke should not enter your home or business unless you have defective plumbing or dry drain traps. Please follow the directions above so that water is in all drain traps within your home. Filling your plumbing traps with water now will block smoke from entering your home at these locations.**
- **If smoke enters your home, you should open your windows to help dissipate the smoke, call a plumber about the probable issue with your plumbing and notify Frazier Engineering at the number below.**
- **Crews from Frazier Engineering will not need to enter into your home or business.**
- You can expect to see smoke escape through vent stacks on the roof of your building – this is a sign that the building's plumbing is properly installed.
- The smoke is white to gray in color, nearly odorless, and is non-toxic and non-staining. The smoke will not leave residue or create a fire hazard. The smoke **IS NOT** harmful to people or pets.

If you have any concerns or questions regarding the work, please call the following Frazier Engineering personnel:
Field Manager: John Guidone – 704.202.0178
Frazier Engineering Office: 704.822.8444

If you need additional information, you may contact Hendersonville Water and Sewer at .

Figure 2 – Example Door Hanger

3.2 CITY OF HENDERSONVILLE RESPONSIBILITIES

The City will provide the necessary access for the inspection of each of the pipelines. The location of the planned inspections is shown in Figures 3 through 8 and the City should review these areas in advance of the inspection to allow for any clearing of the easements or public notification that may be required. The work will be facilitated with Frazier Engineering.

The police and fire departments will be included in the notification by the subcontractor but will be coordinated with the City. The contact for the Police and Fire Departments will be through the City's non-emergency dispatcher.

The inspection crews will provide equipment for access to the manholes required for the testing but may require assistance from City staff in locating some manholes.

The City will need to provide the results of the recent smoke testing to use in comparison with the data collected during this testing.

3.3 OVERVIEW OF INSPECTION LOCATIONS

The representative locations selected for inspection with smoke testing are shown on the following Figures 3 through 8. The smoke testing (SMK) locations are highlighted in a dashed orange. Previous smoke testing has been conducted and those areas are shown in yellow if they are in the same area as this planned testing.

The planned inspection sequence will be coordinated with the contractor and the fire department.

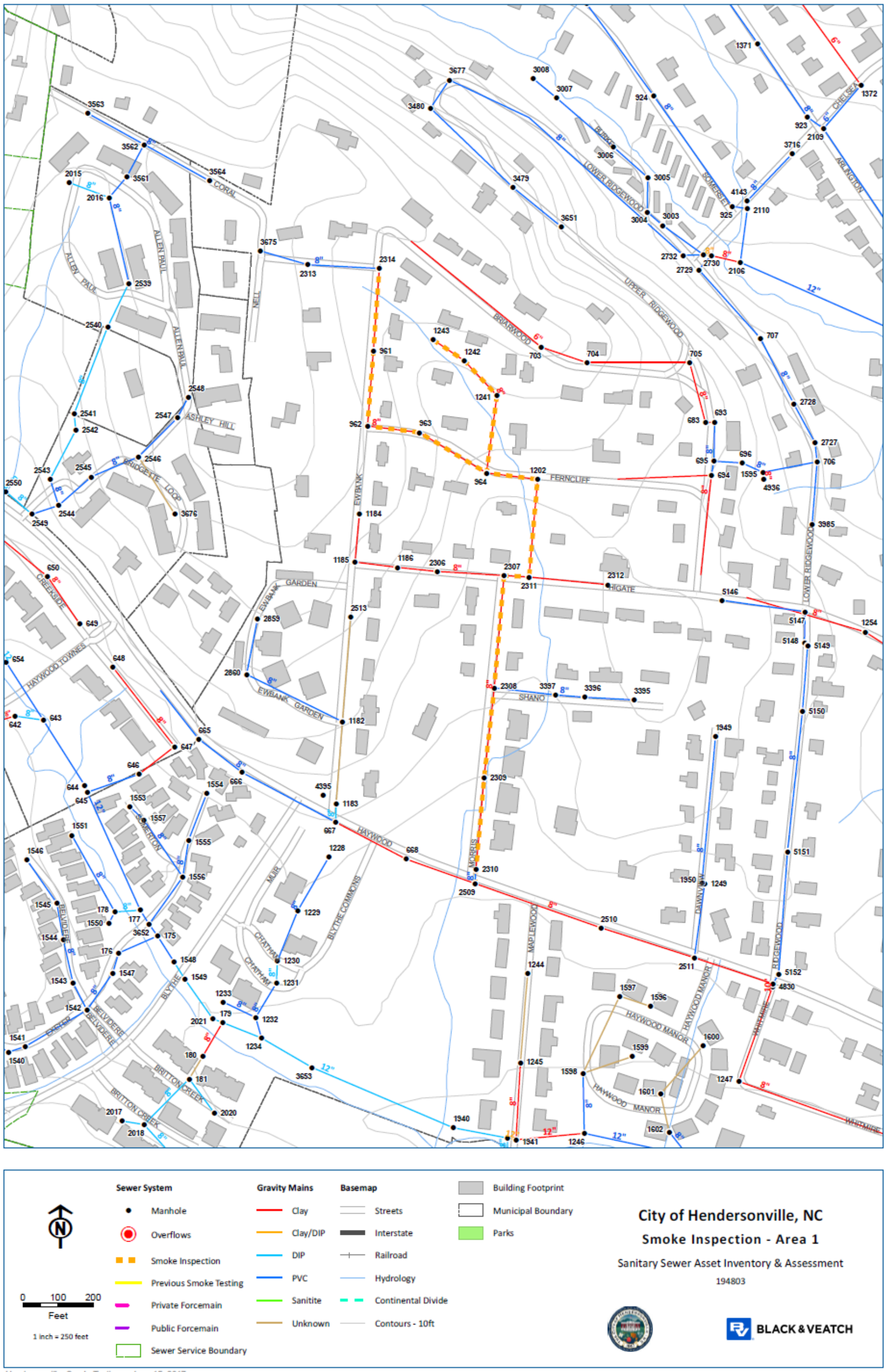


Figure 3 – Inspection Area 1

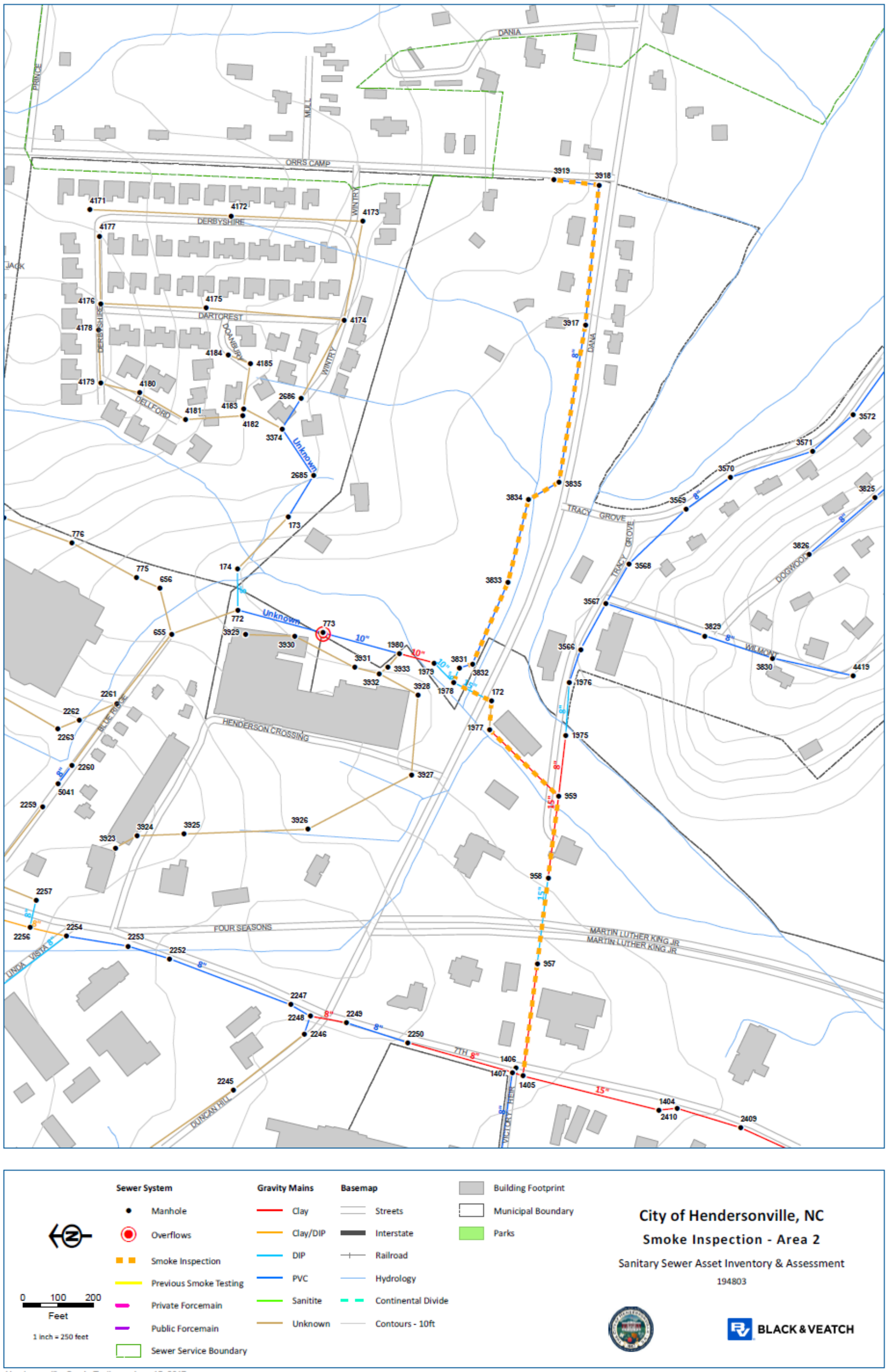


Figure 4 – Inspection Area 2

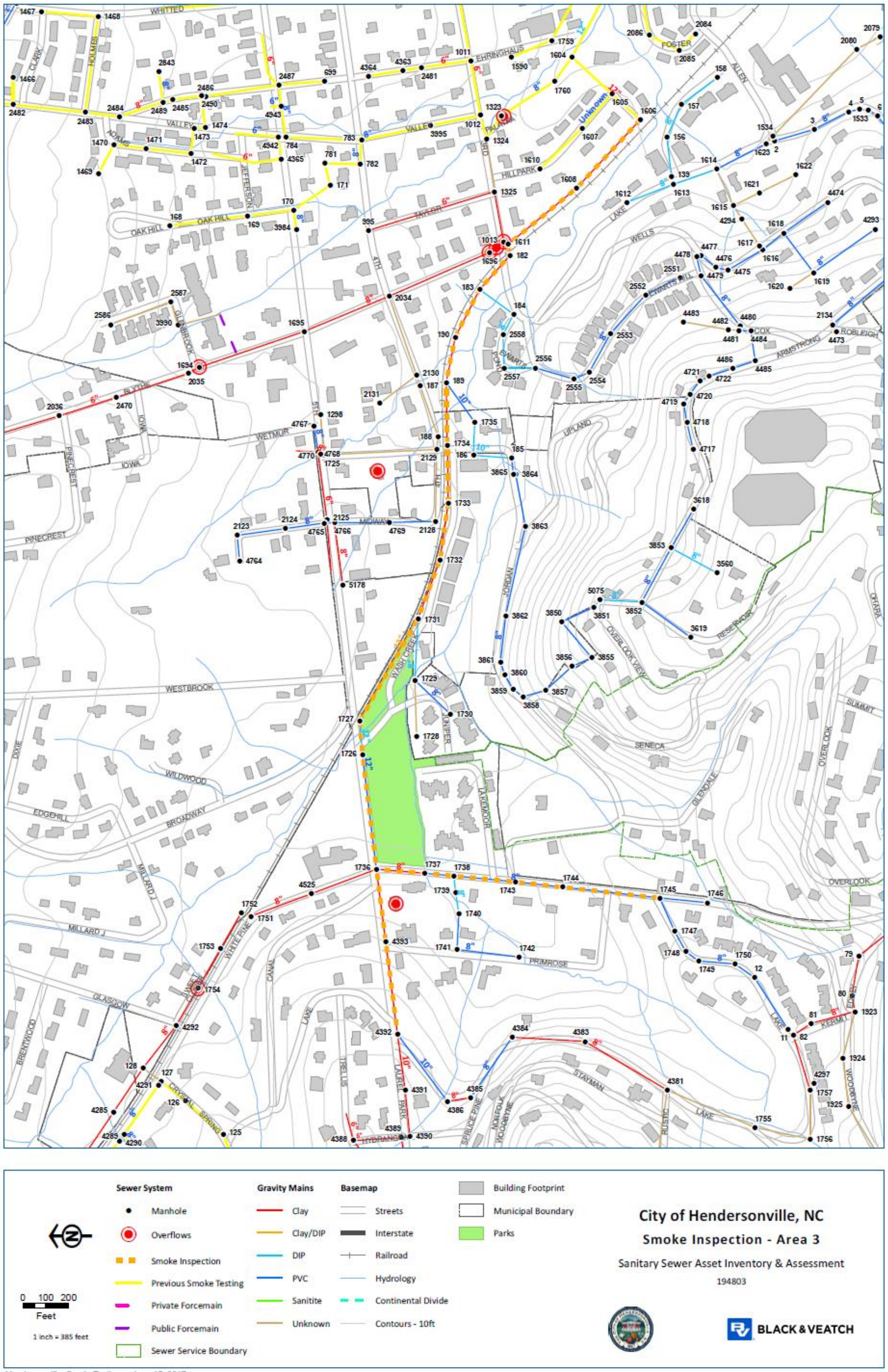


Figure 5 – Inspection Area 3

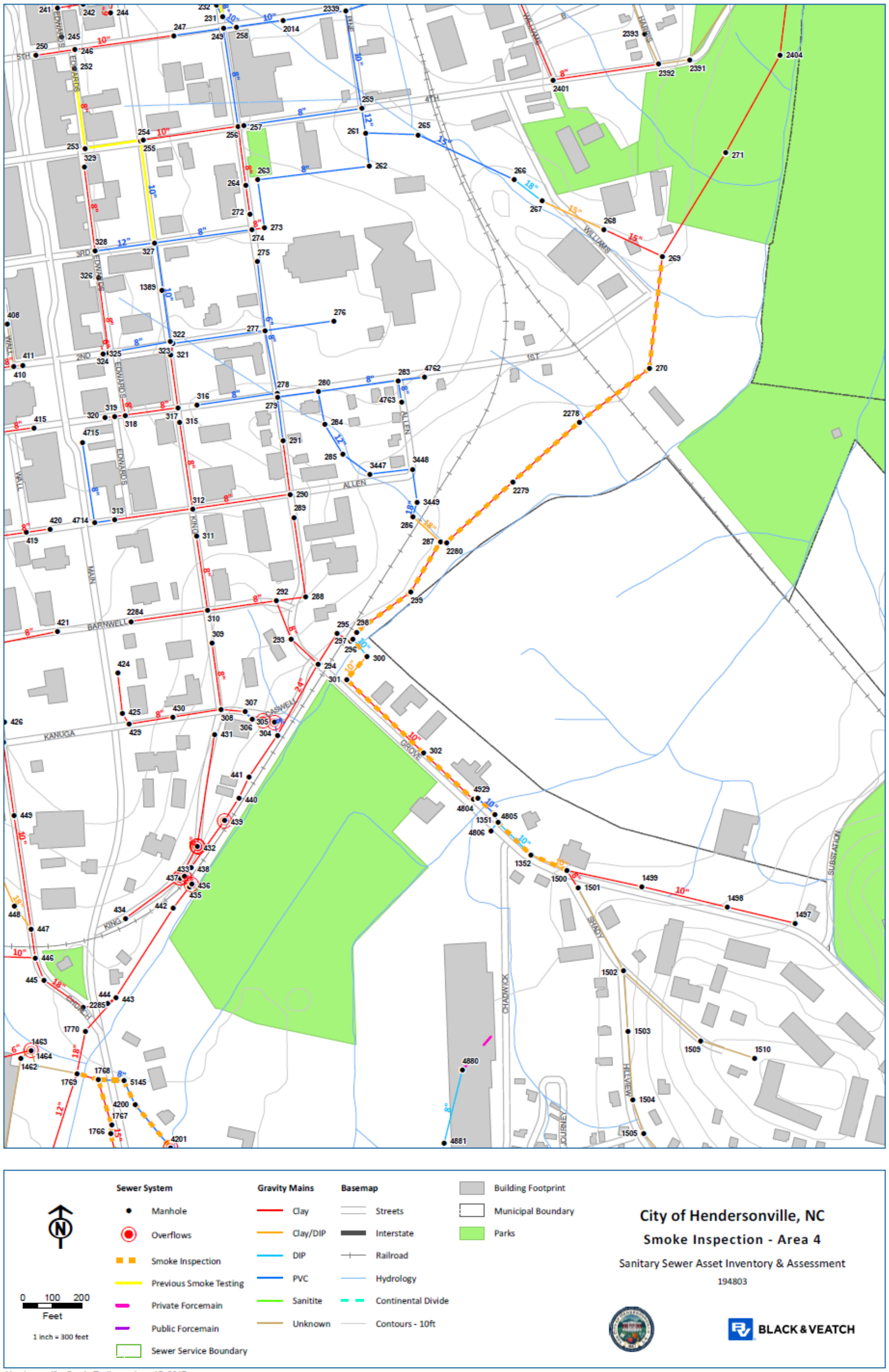


Figure 6 – Inspection Area 4

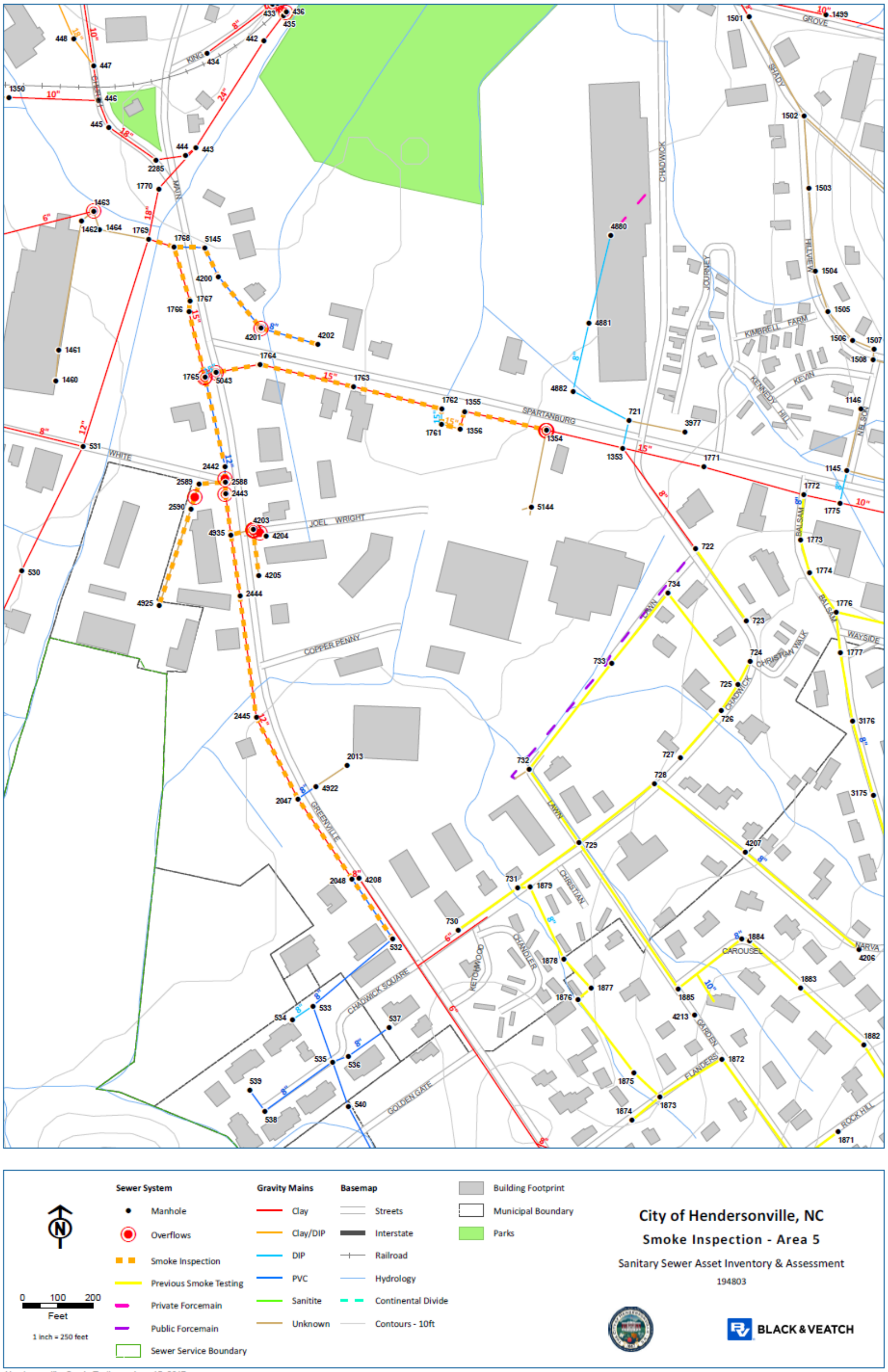


Figure 7 – Inspection Area 5

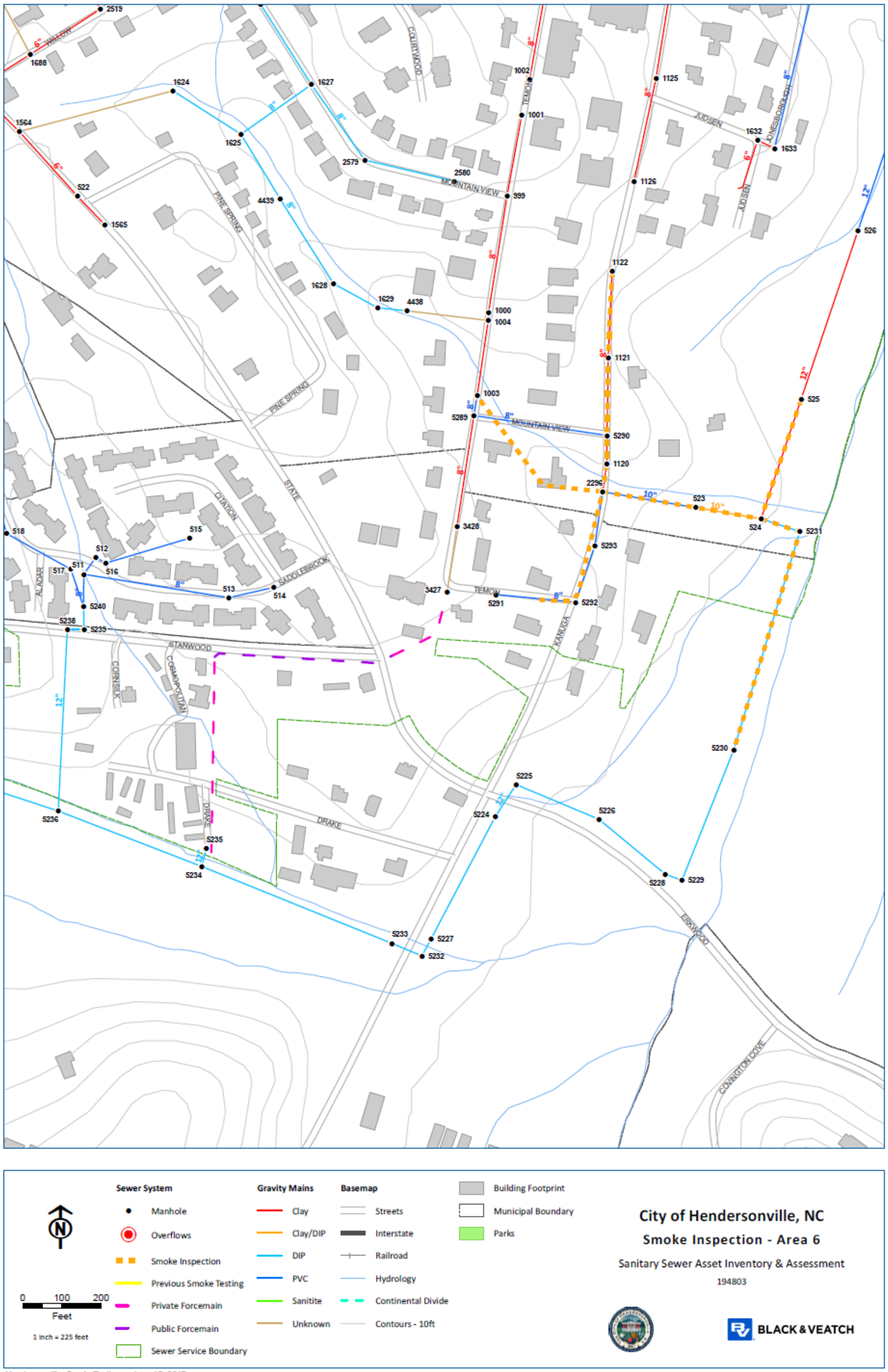


Figure 8 – Inspection Area 6

4.0 Overview of Inspection Methodology

4.1 SMOKE TESTING

The purpose of smoke testing is to identify defects that allow I/I or cross connections in the sewer pipe by forcing smoke into the pipe. The smoke is introduced into the pipe with a blower that seals a manhole and forces the smoke into the pipe, shown in Figure 9. The smoke will be forced out of the pipe at cross connections with storm drains or cracks in the pipe joints or wall shown in Figure 10. The best results are obtained when the soil surrounding the pipe is dry since it will allow the smoke to surface through the voids or cracks in the ground.

Figure 9 – Smoke Testing Blower System

Figure 10 – Smoke Identifies Potential Inflow Locations

5.0 Inspection Schedule

The plan is to have the smoke testing the week of July 11 and is estimated to go for 3 days. This schedule is necessary because the testing provides best results when the ground is dry and the ground water is lowest. Black & Veatch will have an engineer on site during the majority of this work to coordinate with the City and to ensure the data collected is what is needed for the condition assessment.

5.1 SMOKE TESTING

The smoke testing requires notification to the public and fire department in the affected areas prior to the actual testing. The addresses for notification to the property owners will be collected from the GIS maps. The door hangers will be distributed approximately one or two days prior to the testing by Frazier Engineering to the properties along the streets that have been selected for the smoke testing shown on the Figures in Section 3.3.

With the several areas identified it will require some additional time between areas to set up the equipment. It is planned for the smoke testing to be completed in three days.

6.0 Contingency Plan

The coordination of the work with the subcontractors and the City will improve the success of the work. We recognize the potential for delays from access or in collecting the data. The areas identified for inspection can easily be adjusted if there are restrictions in access or if the flows are not suitable for data collection. The lengths and locations of the inspection can be adjusted to meet conditions in the field and remain within the agreed distances.

If there are rain events, the work will be discontinued and re-scheduled because of the impact from the weather on the accuracy of the data collected. In the event of a heavy rain the smoke testing will be pushed back until the ground saturation is low enough to not impact the data collection.

7.0 Data Collection and Reporting

The data shall be collected by the various technologies using the acceptable industry standards to ensure accurate and complete information is gathered. The smoke testing will provide indications of the condition of the pipeline regarding potential for I/I through cracks or cross connections.

It is vital that crewmembers keep complete and accurate field notes documenting each inflow source detected during smoke testing. The following information should be recorded for each inflow source detected:

- Description of defect.
- Street Address and GPS coordinates.
- Document whether the source is located on the city-maintained portion of the sewer system or on a private service line or private property.
- Estimate area (square feet) drained by the inflow source.
- Photograph of the inflow source.

In addition to the above, the following general information should be kept for each smoke test:

- Date.
- Inspectors.
- Setup number.
- Weather conditions.
- Antecedent moisture conditions.
- Time of starting and completion of test.
- Position of smoke blower.
- Manhole Identification Number.

All of the above information shall be kept in a smoke-testing log.

The results of the inspections shall be submitted in a written report by the subcontractor conducting the work. The report shall contain a description of the technology used and the field data gathered during the inspection.

The field data will be used in preparation of the condition assessment report for the pipe and provide recommendations for additional inspections.

8.0 Safety Plan

This work will be covered by the B&V safety guidelines and will be coordinated with Hendersonville Water and Sewer. The subcontractor is responsible for developing and implementing a safety plan for their work.

The primary hazards for this work will be slips, trips, or falls around access sites, exposure to traffic and weather related concerns. The workers shall wear reflective vests and hard hats when working in the roadway and when on-site. The workers shall stay within the cones and use caution when crossing the roads.

Manned entry into the pipe is not anticipated for the inspection. However, if entry into the manholes is required for any reason, the work will require confined space entry compliance. The personnel entering the pipe will be confined space trained.

Appendix F: Frazier Testing Results

Frazier Engineering Acoustic Testing Results

Date	Area	Crew	Manhole #	Downstream Manhole	SLRAT Score	Comments
24-Sep-18	A	MB-BC	MH-1982	MH-1983	8	
24-Sep-18	A	MB-BC	MH-1983	MH-1984	9	
24-Sep-18	A	MB-BC	MH-1984	MH-3742	8	
24-Sep-18	A	MB-BC	MH-1985	MH-1984	8	
24-Sep-18	A	MB-BC	MH-1986	MH-1985	8	
24-Sep-18	A	MB-BC	MH-1987	MH-1988	7	
24-Sep-18	A	MB-BC	MH-1988	MH-3764	9	
24-Sep-18	A	MB-BC	MH-1989	MH-467	7	
24-Sep-18	A	MB-BC	MH-3740	MH-3741	8	
24-Sep-18	A	MB-BC	MH-3741	MH-3742	8	
24-Sep-18	A	MB-BC	MH-3742	MH-3750.1	7	
24-Sep-18	A	MB-BC	MH-3743	MH-3744	8	
24-Sep-18	A	MB-BC	MH-3744	MH-3746	8	
24-Sep-18	A	MB-BC	MH-3745	MH-3746	8	Slight roots at frame
24-Sep-18	A	MB-BC	MH-3746	MH-3747	8	
24-Sep-18	A	MB-BC	MH-3747	MH-3748	8	
24-Sep-18	A	MB-BC	MH-3748	MH-3749	8	
24-Sep-18	A	MB-BC	MH-3749	MH-700	6	Combined Score. Added manhole found 15 feet upstream of MH-700
24-Sep-18	A	MB-BC	MH-3750	MH-3751	9	
24-Sep-18	A	MB-BC	MH-3750.1	MH-3750	9	
24-Sep-18	A	MB-BC	MH-3751	MH-700	8	
24-Sep-18	A	MB-BC	MH-3752	MH-1986	7	
24-Sep-18	A	MB-BC	MH-3753	MH-3754	8	
24-Sep-18	A	MB-BC	MH-3754	MH-3755	8	

24-Sep-18	A	MB-BC	MH-3755	MH-3756	7	
24-Sep-18	A	MB-BC	MH-3756	MH-3757	8	
24-Sep-18	A	MB-BC	MH-3757	MH-3758.1	7	
24-Sep-18	A	MB-BC	MH-3758	MH-3759	8	
24-Sep-18	A	MB-BC	MH-3758.1	MH-3758	8	
24-Sep-18	A	MB-BC	MH-3759	MH-3760	7	
24-Sep-18	A	MB-BC	MH-3760	MH-3761	8	
24-Sep-18	A	MB-BC	MH-3761	MH-3762	8	
24-Sep-18	A	MB-BC	MH-3762	MH-3763	8	
24-Sep-18	A	MB-BC	MH-3763	MH-702	9	
24-Sep-18	A	MB-BC	MH-3764	MH-3765	9	
24-Sep-18	A	MB-BC	MH-3765	MH-3766	8	
24-Sep-18	A	MB-BC	MH-3766	MH-467	8	
24-Sep-18	A	MB-BC	MH-700	MH-701	8	
24-Sep-18	A	MB-BC	MH-701	MH-702	8	
24-Sep-18	A	MB-BC	MH-702	MH-1987	8	
25-Sep-18	B	MB-BC	MH-1011	MH-1012	9	
25-Sep-18		MB-BC	MH-1012	MH-1324	7	
25-Sep-18		MB-BC	MH-1013	MH-1325	1	
25-Sep-18		MB-BC	MH-1018	MH-5296	0	Blockage
25-Sep-18		MB-BC	MH-1119	MH-1286	0	Blockage
25-Sep-18		MB-BC	MH-1283	MH-1282	6	
25-Sep-18		MB-BC	MH-1285	MH-1283	6	
25-Sep-18		MB-BC	MH-1287	MH-1288	5	
25-Sep-18		MB-BC	MH-1325	MH-1324	1	
25-Sep-18		MB-BC	MH-1466	MH-2482	5	
25-Sep-18		MB-BC	MH-1581	MH-1580	0	Blockage
25-Sep-18		MB-BC	MH-1585	MH-887	9	
25-Sep-18		MB-BC	MH-1674	MH-1675	3	Full of debris

25-Sep-18		MB-BC	MH-1675	MH-977	8	
25-Sep-18		MB-BC	MH-1676	MH-1675	2	
25-Sep-18		MB-BC	MH-1677	MH-1676	2	
25-Sep-18		MB-BC	MH-1678	MH-1018	0	Blockage
25-Sep-18		MB-BC	MH-1760	MH-1604	1	
25-Sep-18		MB-BC	MH-1991	MH-1990	7	
25-Sep-18		MB-BC	MH-1993	MH-1991	7	
25-Sep-18		MB-BC	MH-1994	MH-1993	8	
25-Sep-18		MB-BC	MH-1995	MH-1994	9	
25-Sep-18		MB-BC	MH-1996	MH-459	8	
25-Sep-18		MB-BC	MH-2482	MH-2483	8	
25-Sep-18		MB-BC	MH-2483	MH-2484	5	
25-Sep-18		MB-BC	MH-4412	MH-990	5	
25-Sep-18		MB-BC	MH-459	MH-1996	7	
25-Sep-18		MB-BC	MH-5296	MH-4412	7	
25-Sep-18		MB-BC	MH-659	MH-661	3	MH-660 appears to have been paved over
25-Sep-18		MB-BC	MH-662	MH-663	5	
25-Sep-18		MB-BC	MH-663	MH-884	2	
25-Sep-18		MB-BC	MH-885.1	MH-1287	6	
25-Sep-18		MB-BC	MH-886	MH-884	3	
25-Sep-18		MB-BC	MH-887	MH-886	2	
25-Sep-18		MB-BC	MH-978	MH-977	9	
25-Sep-18		MB-BC	MH-979	MH-978	9	
25-Sep-18		MB-BC	MH-980	MH-979	7	
25-Sep-18		MB-BC	MH-981	MH-980	8	
25-Sep-18		MB-BC	MH-986	MH-884	1	
25-Sep-18		MB-BC	MH-987	MH-988	3	
25-Sep-18		MB-BC	MH-988	MH-989	5	
25-Sep-18		MB-BC	MH-989	MH-990	5	
25-Sep-18		MB-BC	MH-990	MH-931	5	

26-Sep-18		MB-BC	MH-1014	MH-1015	0	Blockage
26-Sep-18		MB-BC	MH-1015	MH-2517	1	
26-Sep-18		MB-BC	MH-1388	MH-1424	10	
26-Sep-18		MB-BC	MH-1467	MH-1468	3	
26-Sep-18		MB-BC	MH-1468	MH-2483	0	Blockage
26-Sep-18		MB-BC	MH-1694	MH-1695.1	7	
26-Sep-18		MB-BC	MH-1695	MH-2034.1	7	Combined score with MH-2034.2
26-Sep-18		MB-BC	MH-1695.1	MH-1695	7	
26-Sep-18		MB-BC	MH-1696	MH-1013	8	
26-Sep-18		MB-BC	MH-1712	MH-1015	0	Blockage
26-Sep-18		MB-BC	MH-1713	MH-1015	2	
26-Sep-18		MB-BC	MH-1714	MH-2479	0	Blockage
26-Sep-18		MB-BC	MH-2034	MH-1696	0	Blockage
26-Sep-18		MB-BC	MH-2034.1	MH-2034	7	
26-Sep-18		MB-BC	MH-2035	MH-1694	2	
26-Sep-18		MB-BC	MH-2036	MH-2470	2	
26-Sep-18		MB-BC	MH-2037	MH-2436	0	Blockage
26-Sep-18		MB-BC	MH-2069	MH-2037	0	Blockage
26-Sep-18		MB-BC	MH-2470	MH-2035	5	
26-Sep-18		MB-BC	MH-2478	MH-2479	0	Blockage
26-Sep-18		MB-BC	MH-2479	MH-2480	2	
26-Sep-18		MB-BC	MH-2480	MH-2517	1	
26-Sep-18		MB-BC	MH-2516	MH-531	0	Blockage
26-Sep-18		MB-BC	MH-2854	MH-2069	9	
26-Sep-18		MB-BC	MH-2855	MH-2854	9	
26-Sep-18		MB-BC	MH-2856	MH-2855	9	
26-Sep-18		MB-BC	MH-2857	MH-2856	10	
26-Sep-18		MB-BC	MH-306	MH-305	7	
26-Sep-18		MB-BC	MH-307	MH-306	8	

26-Sep-18		MB-BC	MH-308	MH-307	5	
26-Sep-18		MB-BC	MH-309	MH-308	1	
26-Sep-18		MB-BC	MH-424	MH-425	9	
26-Sep-18		MB-BC	MH-425	MH-429	8	
26-Sep-18		MB-BC	MH-429	MH-430	4	
26-Sep-18		MB-BC	MH-430	MH-308	5	
26-Sep-18		MB-BC	MH-431	MH-432	0	Blockage
26-Sep-18		MB-BC	MH-432	MH-433	0	Blockage
26-Sep-18		MB-BC	MH-434	MH-433	0	Blockage
27-Sep-18		MB-BC	MH-1323	MH-1760	9	
27-Sep-18		MB-BC	MH-1324	MH-1323	6	
27-Sep-18		MB-BC	MH-1423	MH-878	5	
27-Sep-18		MB-BC	MH-1424	MH-1423	8	
27-Sep-18		MB-BC	MH-1425	MH-1426	2	
27-Sep-18		MB-BC	MH-1426	MH-1161	0	Blockage
27-Sep-18		MB-BC	MH-220	MH-5070	0	Blockage
27-Sep-18		MB-BC	MH-5072	MH-5071	6	
27-Sep-18		MB-BC	MH-5974	MH-5073	8	
27-Sep-18		MB-BC	MH-876	MH-1425	5	Manhole lid cracked
27-Sep-18		MB-BC	MH-877	MH-876	0	Blockage
27-Sep-18		MB-BC	MH-878	MH-877	0	Blockage
27-Sep-18		MB-BC	MH-880	MH-879	0	Blockage

Frazier Engineering Smoke Testing Results

2017 SEGMENTS SMOKED						
Date	**Crew**	**Weather**	**Manhole**	**Downstream Manhole**	**Setup Number**	**Comments**
12-Jul-17	JD/KA	Wet-Low Groundwater	MH-1120	MH-2296	11	
12-Jul-17	JD/KA	Wet-Low Groundwater	MH-1121	MH-5290	11	
12-Jul-17	JD/KA	Wet-Low Groundwater	MH-1122	MH-1121	11	
11-Jul-17	JD/KA	Wet-Low Groundwater	MH-1202	MH-2311	2	
11-Jul-17	JD/KA	Wet-Low Groundwater	MH-1241	MH-964	2	
11-Jul-17	JD/KA	Wet-Low Groundwater	MH-1242	MH-1241	2	
11-Jul-17	JD/KA	Wet-Low Groundwater	MH-1243	MH-1242	2	
12-Jul-17	JD/KA	Wet-Low Groundwater	MH-1351	MH-4929	10	
12-Jul-17	JD/KA	Wet-Low Groundwater	MH-1352	MH-4805	10	
13-Jul-17	JD/KA	Wet-Low Groundwater	MH-1354	MH-1355	13	
13-Jul-17	JD/KA	Wet-Low Groundwater	MH-1355	MH-1356	13	
13-Jul-17	JD/KA	Wet-Low Groundwater	MH-1356	MH-1761	13	
12-Jul-17	JD/KA	Wet-Low Groundwater	MH-1500	MH-1352	10	
12-Jul-17	JD/KA	Wet-Low Groundwater	MH-1501	MH-1500	10	
12-Jul-17	JD/KA	Wet-Low Groundwater	MH-1608	MH-1606	9	
12-Jul-17	JD/KA	Wet-Low Groundwater	MH-1611	MH-1608	9	
11-Jul-17	JD/KA	Wet-Low Groundwater	MH-172	MH-1977	5	
11-Jul-17	JD/KA	Wet-Low Groundwater	MH-1726	MH-1727	7	
12-Jul-17	JD/KA	Wet-Low Groundwater	MH-1727	MH-1731	8	
12-Jul-17	JD/KA	Wet-Low Groundwater	MH-1731	MH-1732	8	
12-Jul-17	JD/KA	Wet-Low Groundwater	MH-1732	MH-1733	8	
12-Jul-17	JD/KA	Wet-Low Groundwater	MH-1733	MH-1734	8	

12-Jul-17	JD/KA	Wet-Low Groundwater	MH-1734	MH-189	9	
11-Jul-17	JD/KA	Wet-Low Groundwater	MH-1736	MH-1726	7	
11-Jul-17	JD/KA	Wet-Low Groundwater	MH-1737	MH-1736	7	
11-Jul-17	JD/KA	Wet-Low Groundwater	MH-1738	MH-1737	6	
11-Jul-17	JD/KA	Wet-Low Groundwater	MH-1743	MH-1738	6	
11-Jul-17	JD/KA	Wet-Low Groundwater	MH-1744	MH-1743	6	
11-Jul-17	JD/KA	Wet-Low Groundwater	MH-1745	MH-1744	6	
13-Jul-17	JD/KA	Wet-Low Groundwater	MH-1761	MH-1762	13	
13-Jul-17	JD/KA	Wet-Low Groundwater	MH-1762	MH-1763	13	
13-Jul-17	JD/KA	Wet-Low Groundwater	MH-1763	MH-1764	13	
13-Jul-17	JD/KA	Wet-Low Groundwater	MH-1764	MH-5043	13	
13-Jul-17	JD/KA	Wet-Low Groundwater	MH-1765	MH-1766	13	
13-Jul-17	JD/KA	Wet-Low Groundwater	MH-1766	MH-1767	13	
13-Jul-17	JD/KA	Wet-Low Groundwater	MH-1767	MH-1768	13	
13-Jul-17	JD/KA	Wet-Low Groundwater	MH-1768	MH-1769	13	
12-Jul-17	JD/KA	Wet-Low Groundwater	MH-182	MH-1611	9	
12-Jul-17	JD/KA	Wet-Low Groundwater	MH-183	MH-182	9	
12-Jul-17	JD/KA	Wet-Low Groundwater	MH-189	MH-190	9	
12-Jul-17	JD/KA	Wet-Low Groundwater	MH-190	MH-183	9	
11-Jul-17	JD/KA	Wet-Low Groundwater	MH-1977	MH-959	5	
11-Jul-17	JD/KA	Wet-Low Groundwater	MH-1978	MH-172	4	
13-Jul-17	JD/KA	Wet-Low Groundwater	MH-2047	MH-2445	14	
13-Jul-17	JD/KA	Wet-Low Groundwater	MH-2048	MH-2047	14	
12-Jul-17	JD/KA	Wet-Low Groundwater	MH-2278	MH-270	10	
12-Jul-17	JD/KA	Wet-Low Groundwater	MH-2279	MH-2278	10	
12-Jul-17	JD/KA	Wet-Low Groundwater	MH-2280	MH-2279	10	
12-Jul-17	JD/KA	Wet-Low Groundwater	MH-2296	MH-523	12	
11-Jul-17	JD/KA	Wet-Low Groundwater	MH-2307	MH-2308	3	
11-Jul-17	JD/KA	Wet-Low Groundwater	MH-2308	MH-2309	3	
11-Jul-17	JD/KA	Wet-Low Groundwater	MH-2309	MH-2310	3	

11-Jul-17	JD/KA	Wet-Low Groundwater	MH-2310	MH-2509	3	
11-Jul-17	JD/KA	Wet-Low Groundwater	MH-2311	MH-2307	2	
11-Jul-17	JD/KA	Wet-Low Groundwater	MH-2314	MH-961	1	
13-Jul-17	JD/KA	Wet-Low Groundwater	MH-2442	MH-1765	13	
13-Jul-17	JD/KA	Wet-Low Groundwater	MH-2443	MH-2588	13	
13-Jul-17	JD/KA	Wet-Low Groundwater	MH-2444	MH-4935	13	
13-Jul-17	JD/KA	Wet-Low Groundwater	MH-2445	MH-2444	13	
13-Jul-17	JD/KA	Wet-Low Groundwater	MH-2588	MH-2442	13	
13-Jul-17	JD/KA	Wet-Low Groundwater	MH-2589	MH-2588	13	
13-Jul-17	JD/KA	Wet-Low Groundwater	MH-2590	MH-2589	13	
12-Jul-17	JD/KA	Wet-Low Groundwater	MH-270	MH-269	10	
12-Jul-17	JD/KA	Wet-Low Groundwater	MH-287	MH-2280	10	
12-Jul-17	JD/KA	Wet-Low Groundwater	MH-296	MH-297	10	
12-Jul-17	JD/KA	Wet-Low Groundwater	MH-297	MH-298	10	
12-Jul-17	JD/KA	Wet-Low Groundwater	MH-298	MH-299	10	
12-Jul-17	JD/KA	Wet-Low Groundwater	MH-299	MH-287	10	
12-Jul-17	JD/KA	Wet-Low Groundwater	MH-300	MH-296	10	
12-Jul-17	JD/KA	Wet-Low Groundwater	MH-301	MH-300	10	
12-Jul-17	JD/KA	Wet-Low Groundwater	MH-302	MH-301	10	
11-Jul-17	JD/KA	Wet-Low Groundwater	MH-3831	MH-1978	4	
11-Jul-17	JD/KA	Wet-Low Groundwater	MH-3832	MH-3831	4	
11-Jul-17	JD/KA	Wet-Low Groundwater	MH-3833	MH-3832	4	
11-Jul-17	JD/KA	Wet-Low Groundwater	MH-3834	MH-3833	4	
11-Jul-17	JD/KA	Wet-Low Groundwater	MH-3835	MH-3834	4	
11-Jul-17	JD/KA	Wet-Low Groundwater	MH-3917	MH-3835	4	
11-Jul-17	JD/KA	Wet-Low Groundwater	MH-3918	MH-3917	4	
11-Jul-17	JD/KA	Wet-Low Groundwater	MH-3919	MH-3918	4	
13-Jul-17	JD/KA	Wet-Low Groundwater	MH-4200	MH-5145	13	
13-Jul-17	JD/KA	Wet-Low Groundwater	MH-4201	MH-4200	13	
13-Jul-17	JD/KA	Wet-Low Groundwater	MH-4202	MH-4201	13	

13-Jul-17	JD/KA	Wet-Low Groundwater	MH-4203	MH-4935	13	
13-Jul-17	JD/KA	Wet-Low Groundwater	MH-4205	MH-4203	13	
11-Jul-17	JD/KA	Wet-Low Groundwater	MH-4392	MH-4393	7	
11-Jul-17	JD/KA	Wet-Low Groundwater	MH-4393	MH-1736	7	
12-Jul-17	JD/KA	Wet-Low Groundwater	MH-4804	MH-302	10	
12-Jul-17	JD/KA	Wet-Low Groundwater	MH-4805	MH-1351	10	
12-Jul-17	JD/KA	Wet-Low Groundwater	MH-4806	MH-4805	10	
13-Jul-17	JD/KA	Wet-Low Groundwater	MH-4925	MH-2590	13	
12-Jul-17	JD/KA	Wet-Low Groundwater	MH-4929	MH-4804	10	
13-Jul-17	JD/KA	Wet-Low Groundwater	MH-4935	MH-2443	13	
13-Jul-17	JD/KA	Wet-Low Groundwater	MH-5043	MH-1765	13	
13-Jul-17	JD/KA	Wet-Low Groundwater	MH-5145	MH-1768	13	
12-Jul-17	JD/KA	Wet-Low Groundwater	MH-523	MH-524	12	
12-Jul-17	JD/KA	Wet-Low Groundwater	MH-5231	MH-524	12	
12-Jul-17	JD/KA	Wet-Low Groundwater	MH-524	MH-525	12	
12-Jul-17	JD/KA	Wet-Low Groundwater	MH-5290	MH-1120	11	
12-Jul-17	JD/KA	Wet-Low Groundwater	MH-5292	MH-5293	11	
12-Jul-17	JD/KA	Wet-Low Groundwater	MH-5293	MH-2296	11	
13-Jul-17	JD/KA	Wet-Low Groundwater	MH-532	MH-2048	14	
11-Jul-17	JD/KA	Wet-Low Groundwater	MH-957	MH-1405	5	
11-Jul-17	JD/KA	Wet-Low Groundwater	MH-958	MH-957	5	
11-Jul-17	JD/KA	Wet-Low Groundwater	MH-959	MH-958	5	
11-Jul-17	JD/KA	Wet-Low Groundwater	MH-961	MH-962	1	
11-Jul-17	JD/KA	Wet-Low Groundwater	MH-962	MH-963	1	
11-Jul-17	JD/KA	Wet-Low Groundwater	MH-963	MH-964	1	
11-Jul-17	JD/KA	Wet-Low Groundwater	MH-964	MH-1202	1	

Smoke Testing Results

Date	Crew	Smoke Sketch Number	MH	Downstream Manhole	Leak Number	Address	Defect Type	Potential	Photo Number	Comments
12-Jul-17	JD/KA	1	MH-183	MH-182	1	175' Downstream of Manhole 183	Storm Drain	Severe	111-1068/ 111-1069	Heavy smoke from storm drain catch basin and storm pipe. Severe inflow potential
13-Jul-17	JD/KA	2	MH-532	MH-2048	1	110A Greenville Highway	SVC Cleanout	Light	111-1072	4" PVC cleanout cap and insert missing at grade. Light inflow potential
12-Jul-17	JD/KA	3	MH-1726	MH-1727	1	120' Downstrean of Manhole 1726	Mainline Quick Entry	Severe	111-1065	Heavy smoke from mainline sewer at creek. VCP Aerial. Severe inflow potential
12-Jul-17	JD/KA	4	MH-1727	MH-1731	1	66' Downstrean of Manhole 1727	Mainline Multiple	Severe	111-1066	Heavy smoke from multiple sinkholes over mainline sewer. Line being crushed by railroad tracks. Severe inflow potential
13-Jul-17	JD/KA	5	MH-1763	MH-1764	1	610 Spartanburg Highway	SVC Cleanout	Moderate	111-1071	Two 4" PVC services are open and exposed 2' below grade. Moderate inflow potential
11-Jul-17	JD/KA	6	MH-2309	MH-2310	1	204A Morris Lane	SVC Cleanout	Light	111-1061	4" PVC cleanout standpipe broken 12" below grade in vault. Light inflow potential
11-Jul-17	JD/KA	7	MH-2311	MH-2307	1	220 Morris Lane	SVC Cleanout	Moderate	111-1060	4" PVC cleanout cap missing 4" below grade near storm ditch.

										Moderate inflow potential
11-Jul-17	JD/KA	8	MH-3917	MH-3835	1	216 Dana Road	SVC Cleanout	Moderate	111-1062	4" PVC cleanout cap and insert missing 1" above grade in low lying area. Moderate inflow potential

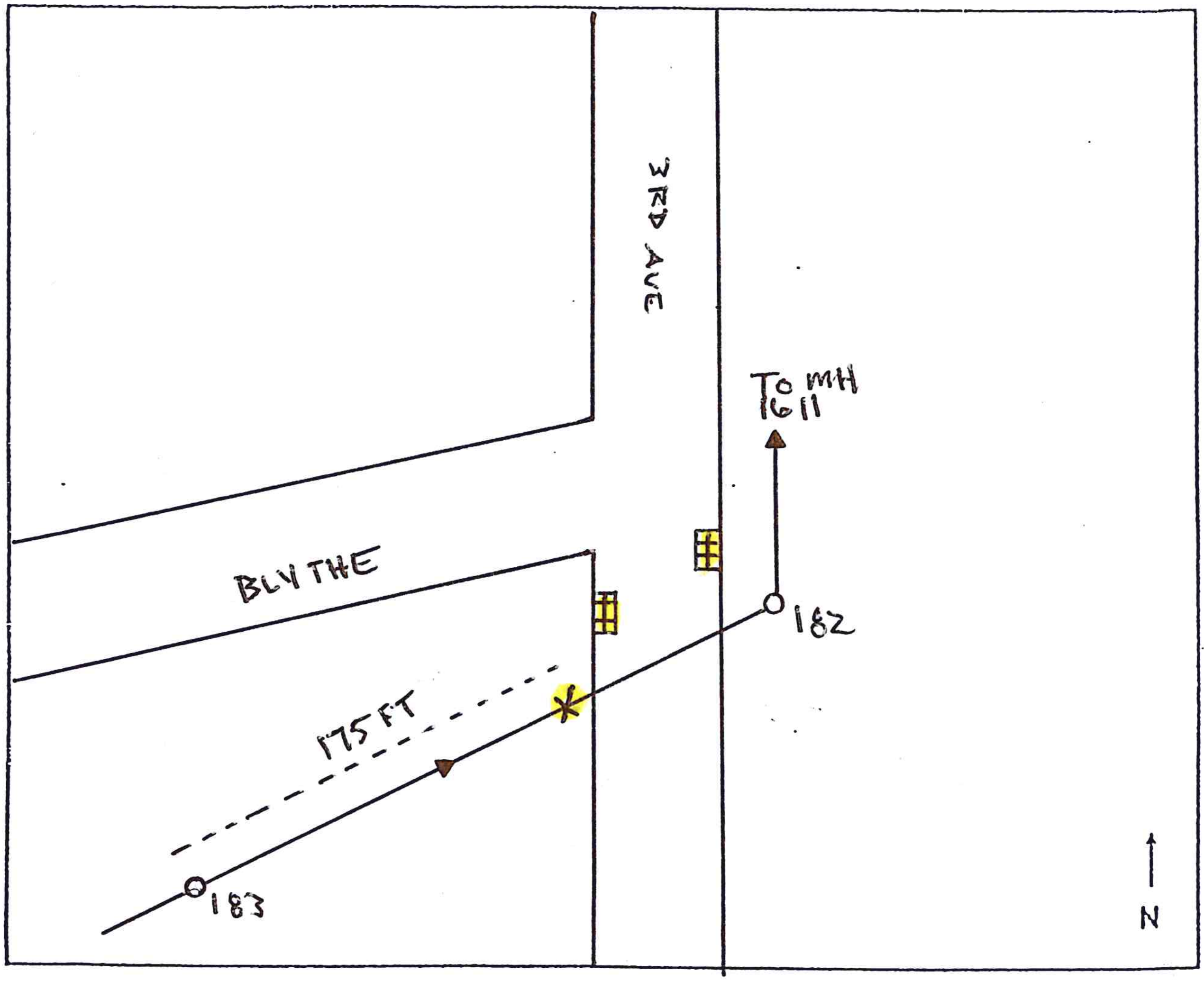
1

SMOKE TESTING FORM

Basin HENDERSONVILLE Inspector JD | KA Date 7/12/17

Photo Number(s) 111-1068 111-1069 Upstream MH 183 Downstream MH 182

Location 175 FT DOWNSTREAM OF MH #183.



1. Source Description / Location HEAVY SMOKE FROM STORM DRAIN CATCH BASINS. SEVERE INFLOW POTENTIAL.

2. Source Description / Location

3. Source Description / Location

4. Source Description / Location

Smoke Sketch 1

Photo: 111-1068

Photo: 111-1069

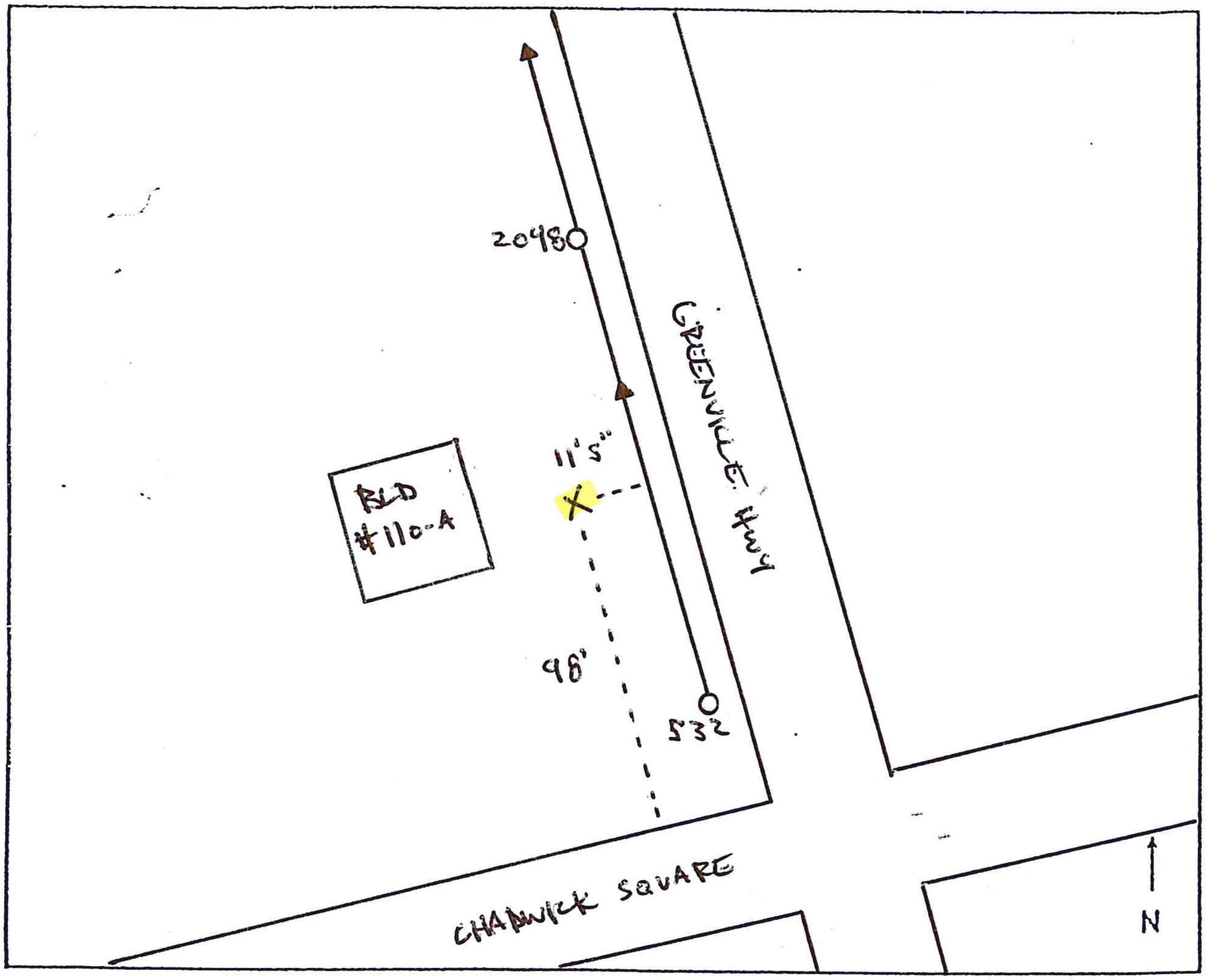
2

SMOKE TESTING FORM

Basin HENDERSONVILLE Inspector JD/KA Date 7/13/17

Photo Number(s) 111-1072 Upstream MH 532 Downstream MH 2048

Location #110-A GREENVILLE HWY



1. Source Description / Location 4" PVC CLEANOUT CAP AND INSERT ARE MISSING AT GRADE. LOW INFLOW POTENTIAL.

2. Source Description / Location

3 Source Description / Location

4 Source Description / Location

Smoke Sketch 2 Photo: 111-1072

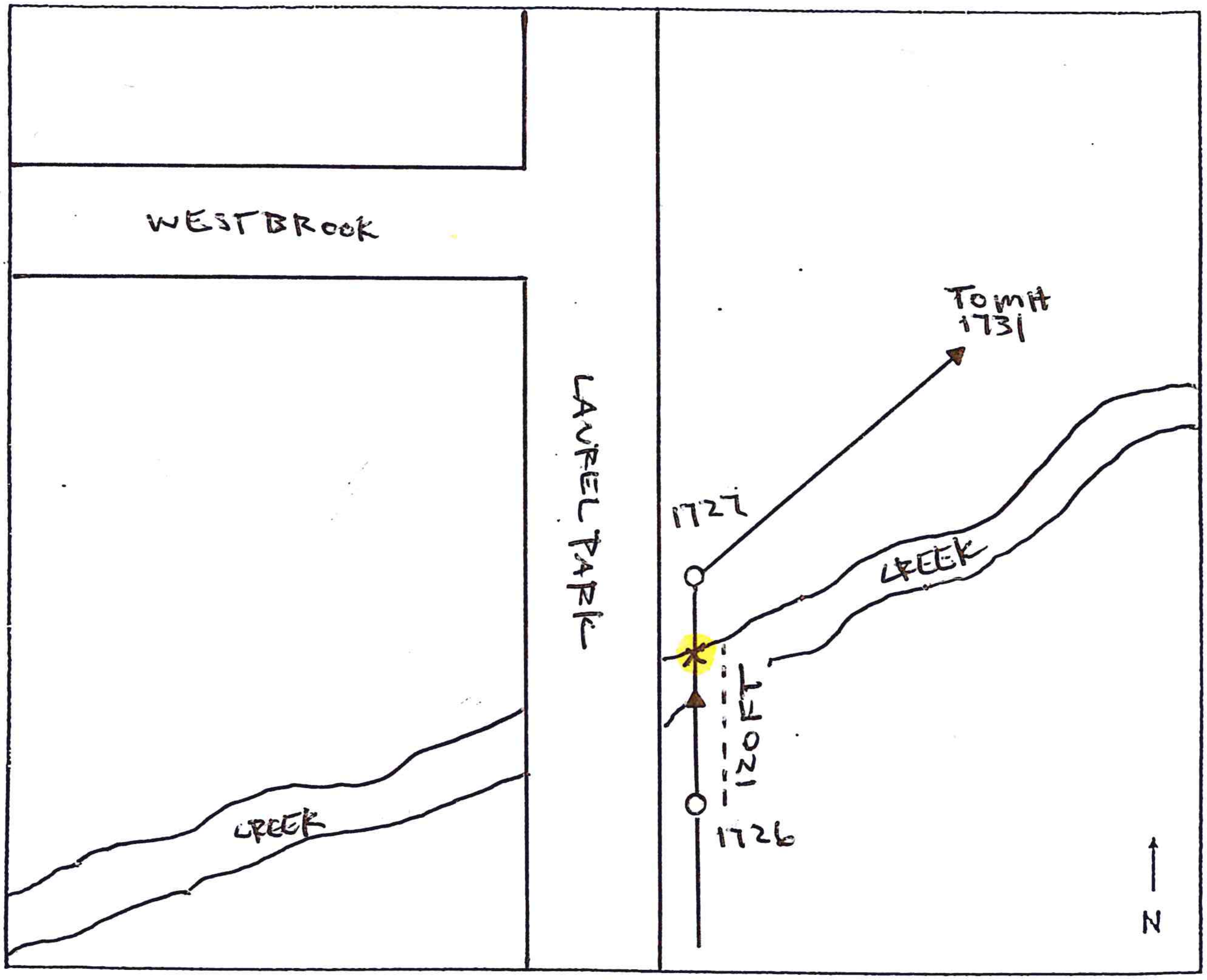
3

SMOKE TESTING FORM

Basin HENDERSONVILLE Inspector JD/KA Date 7/12/17

Photo Number(s) 111-1065 Upstream MH 1726 Downstream MH 1727

Location 120 FT DOWNSTREAM OF MH#1726



1. Source Description / Location VERY HEAVY SMOKE FROM MAINLINE QUICK ENTRY CROSSING CREEK. VCP MATERIAL. SEVERE INFLOW

2. Source Description / Location POTENTIAL.

3 Source Description / Location

4 Source Description / Location

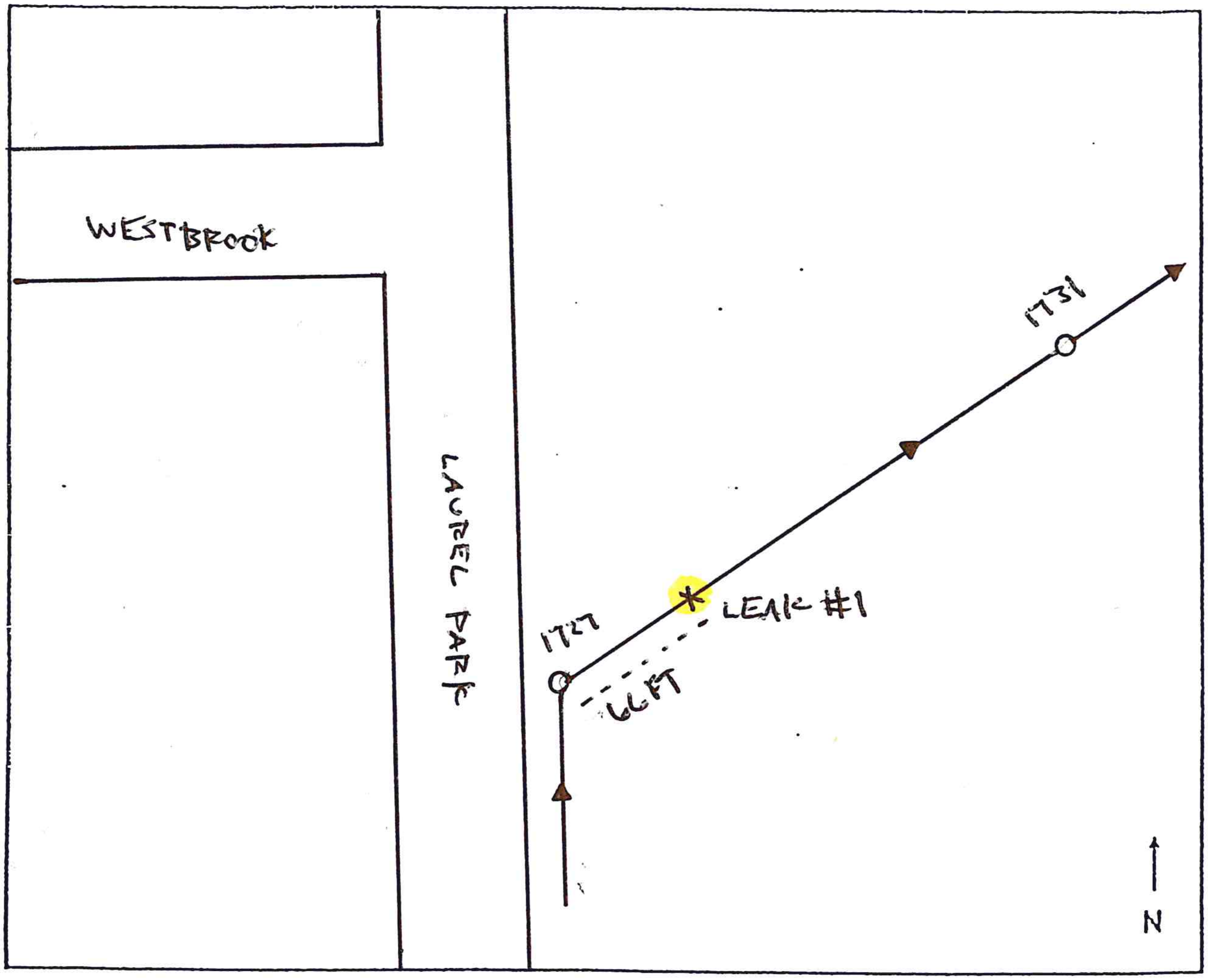
Smoke Sketch 3 Photo: 111-1065

SMOKE TESTING FORM

Basin HENDERSONVILLE Inspector JD/KA Date 7/12/17

Photo Number(s) 111-1066 Upstream MH 1727 Downstream MH 1731

Location 66 FT DOWNSTREAM OF MH #1727



1. Source Description / Location HEAVY SMOKE FROM MULTIPLE SINKHOLES OVER MAINLINE SEWER. SEVERE INFLOW POTENTIAL. RIGHT

2. Source Description / Location BESIDE TRAIN TRACKS.

3. Source Description / Location

4. Source Description / Location

Smoke Sketch 4

Photo: 111-1066

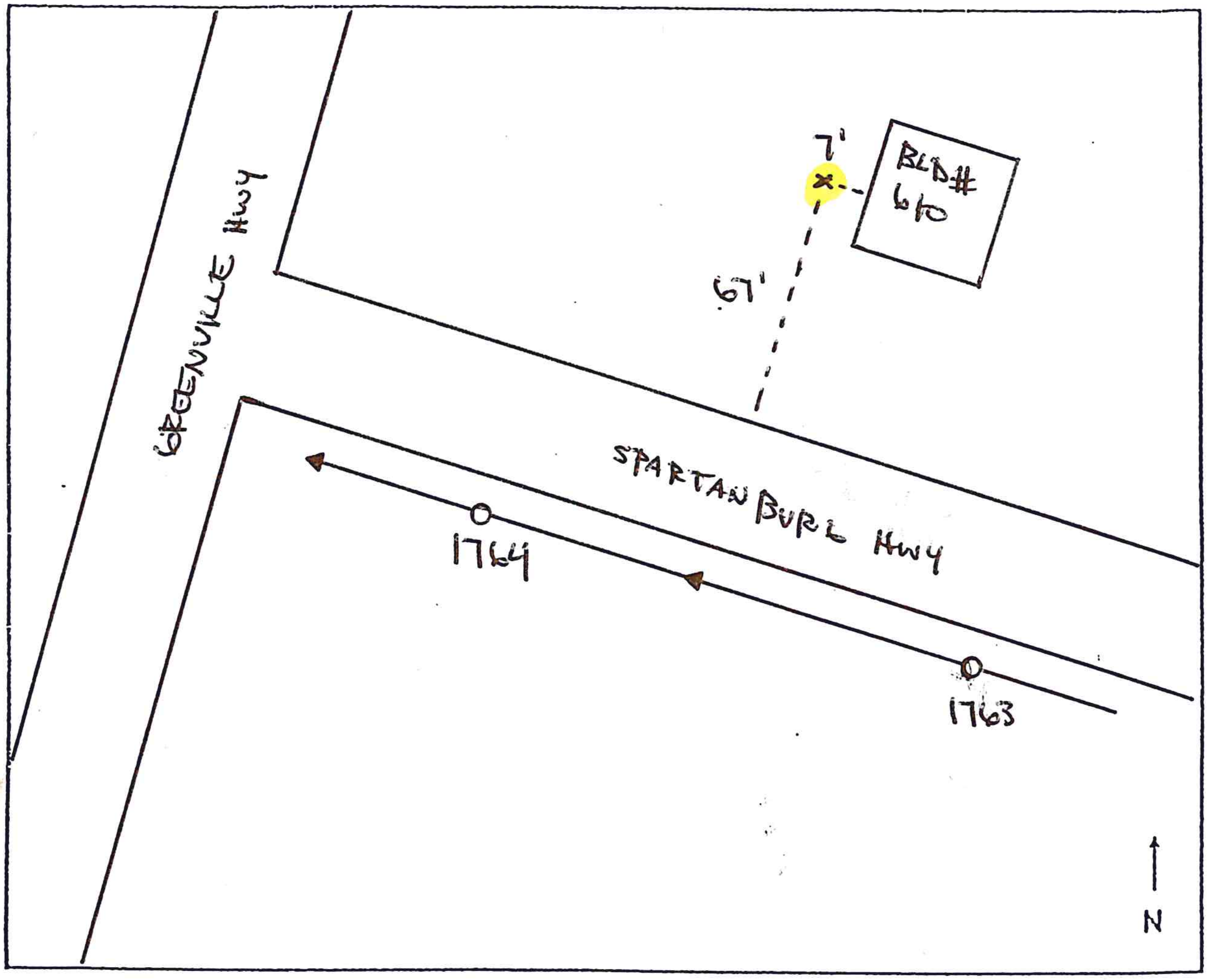
5

SMOKE TESTING FORM

Basin HENDERSONVILLE Inspector JD/KA Date 7/13/17

Photo Number(s) 111-1071 Upstream MH 1763 Downstream MH 1764

Location #610 SPARTANBURG HWY (CHURCH BUILDING)



1. Source Description / Location TWO SERVICE LINES ARE OPEN AND EXPOSED 2 FT BELOW GRADE, HAS NO TOPSIDE CAP. NEEDS PERMANENT

2. Source Description / Location CAP INSTALLED.

3 Source Description / Location

4 Source Description / Location

Smoke Sketch 5 Photo: 111-1071

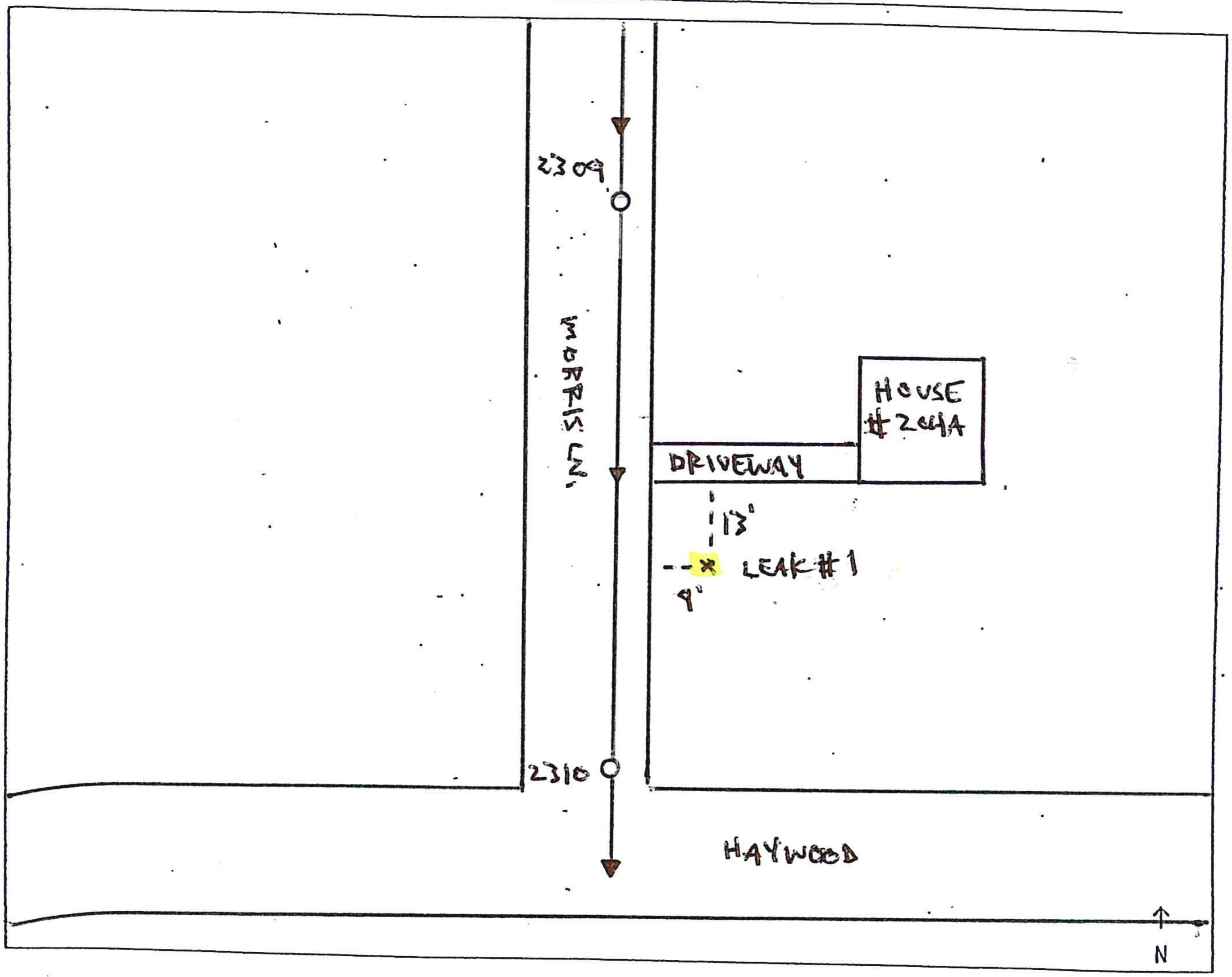
6

Smoke Testing Sketch

Basin HENDERSONVILLE Inspector JD/KA Date 7/11/17

Photo #(s) 111-1061 Upstream MH 2309 Downstream MH 2310

Location #204 A MORRIS LN.



1 Source Description 4" PVC STANDPIPE IS BROKEN 12" BELOW GRADE IN VAULT. LOW INFLOW POTENTIAL.

2 Source Description

3 Source Description

4 Source Description

Smoke Sketch 6 Photo: 111-1061

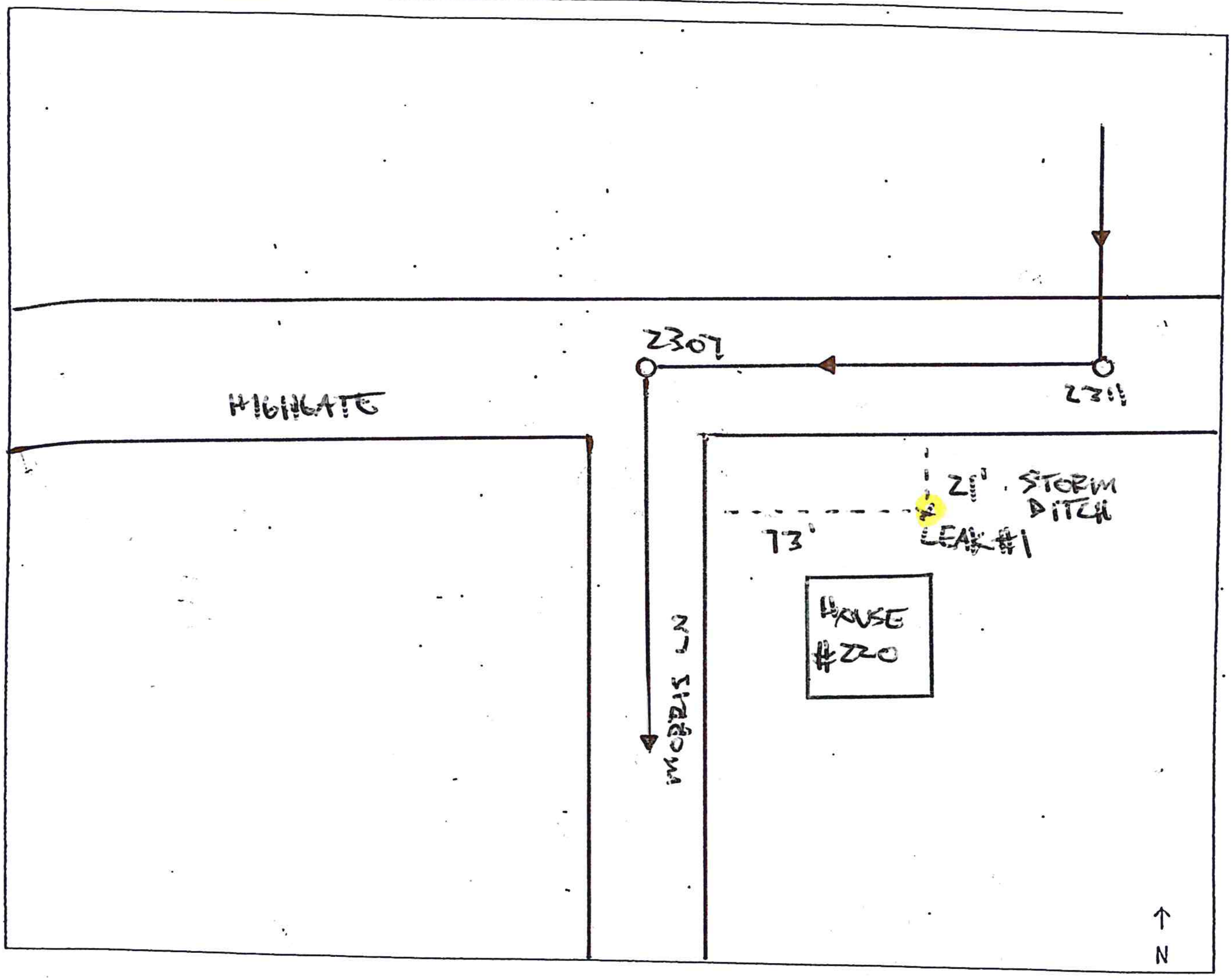
7

Smoke Testing Sketch

Basin HENDERSONVILLE Inspector JD/KA Date 7/11/17

Photo #(s) 111-1060 Upstream MH 2311 Downstream MH 2367

Location #220 MORRIS LN.



1 Source Description 4" PVC CLEANOUT IS MISSING CAP 4" BELOW GRADE, NEAR STORM PIPE. MODERATE INFLOW POTENTIAL.

2 Source Description

3 Source Description

4 Source Description

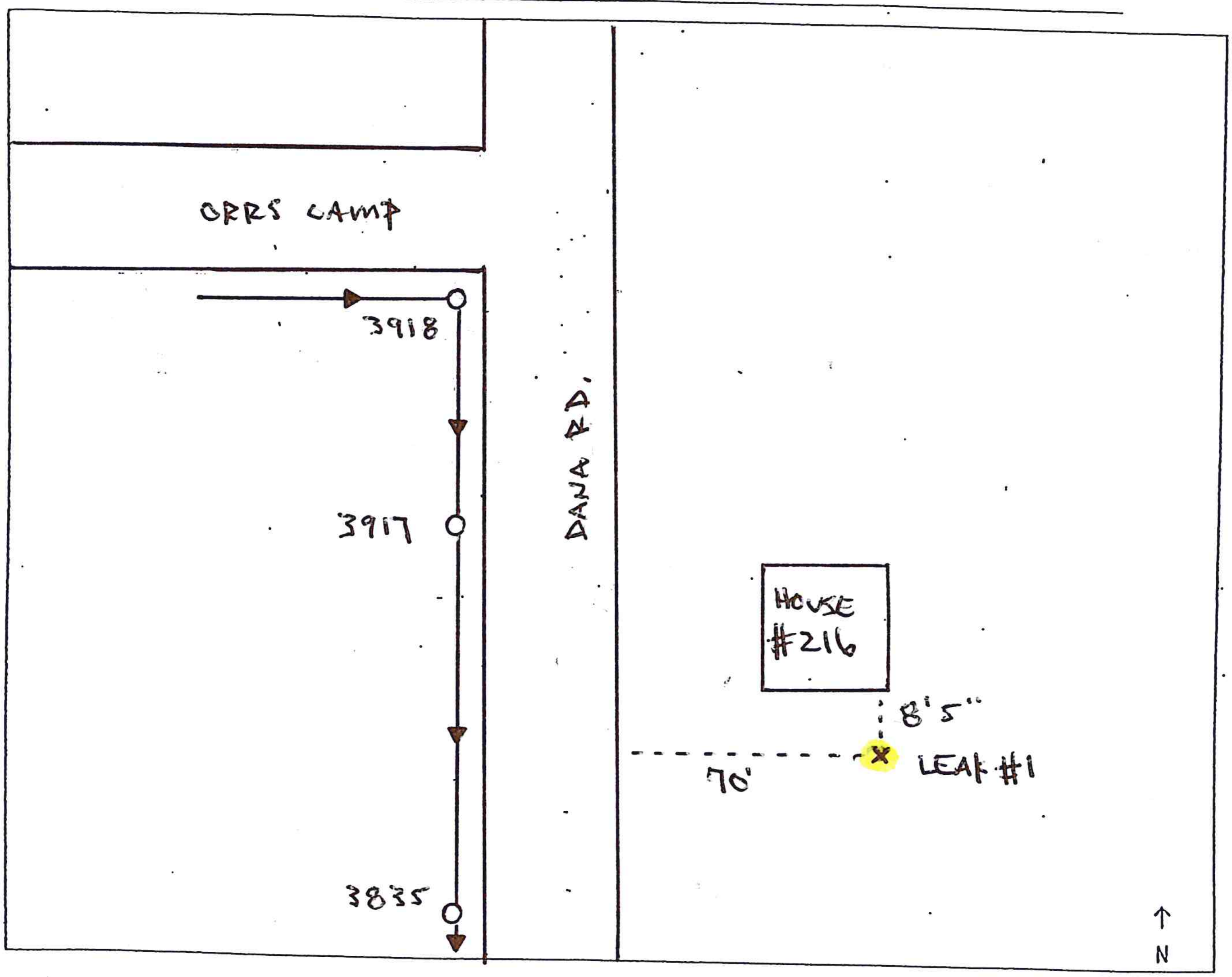
Smoke Sketch 7 Photo: 111-1060

Smoke Testing Sketch

Basin HENDERSONVILLE Inspector JD|KA Date 7|11|17

Photo #(s) 111-1062 Upstream MH 3917 Downstream MH 3835

Location # 216 DANA RD.



1 Source Description 4" PVC CLEANOUT CAP AND INSERT ARE MISSING 1" ABOVE GRADE. IN LOW LYING AREA. 200 SQFT DRAINAGE AREA. MODERATE INFLOW POTENTIAL.

2 Source Description

3 Source Description

4 Source Description

Smoke Sketch 8 Photo: 111-1062

Appendix G: COH Lift Station Inspection Results

Site ID #.	Site Name	Station Age, yrs.	Driveway Condition	Landscaping / Drainage Condition	Pumps: H P	Pumps: Voltage / Phase	Pumps: GPM	Condition (Rate 1 to 5): Influent Piping Size/Type/Condition	Force Main Piping Size/ Type/	Check Valve(s)	Main Breaker Panel	Pump Control Panel	SCADA Panel	Check Valve Vault (Drain)
1	G E	25	4	4	20	3 Phase 480 Volts	500	8" Ductile	6" To 8"	4	4	4	2	NA
3	Garden Lane	42	4	4	2	3 Phase 200 Volts	100	2/ 8" Lines	4" Ductile	2	4	4	2	3
8	Browning Av.	23	4	4	10	3Phase 240 Volts	100	2/ 8" PVC lines	6" Ductile	4	4	4	4	4
10	Outback	18	4	4	10	3 Phase 460 Volts	100	8" PVC	4" Ductile	4	4	4	2	4
11	Bonclarken	18	4	4	50	3 Phase 460 Volts	500	8"PVC-12"Ductile	6" Ductile	4	4	4	4	2
12	Highland lake Dr	17	4	4	7.5	3 Phase 230 Volts	133	8"PVC-8" Ductile	4' Ductile	4	4	4	2	4
14	Dunroy	17	3	3	5	3 Phase 230 Volts	45	8" PVC	2" Galv	4	4	4	2	4
15	Carriage Park Grapevine	17	3	3	5	3 Phase 230 Volts	37	2- 8" PVC Lines	2" Galv	5	4	4	2	4
16	Kenmure Driving Range	17	4	4	30	3 Phase 460 Volts	125	8" PVC & 8" Duct	4" Ductile	4	4	4	2	4
17	Carl Sandburgs	17	4	4	7.5	3 Phase 460 Volts	125	8" Ductile	4" Ductile	4	4	4	2	4
18	Kenmure Brookwood	15	4	4	7.5	3 Phase 200 Volts	24	8" PVC- 8" Ductile	4" PVC	4	4	4	2	None
19	Highland lake Golf	17	4	4	5	3 Phase 230 Volts	37	8"PVC- 8" Ductile	2" Galv	4	4	4	2	4
20	Leverette Dr	18	4	4	7.5	3 Phase 230 Volts	79	8" PVC	4 " Ductile	4	4	4	2	3
23	Lakewood Rv	14	4	4	5	1 Phase 230 Volts	60	8" PVC	3 " Ductile	4	4	4	2	4
24	Shaws Creek	14	3	3	5	1 Phase 230 Volts	62	2- 8" PVC 1-8" Ductile	2 " Ductile	2	4	4	2	4
25	Dana School	13	4	4	15	3 Phase 460 Volts	40	6" PVC	4" Ductile	4	4	4	2	4
26	Clear Creek School	16	4	4	5	3 Phase 460 Volts	50	8 " Pvc	4" Ductile	4	4	4	2	4
28	Eagle Point	11	4	4	3	3 Phase 460 Volts	36	8" Ductile	4" Ductile	4	4	4	2	4
29	Carriage Park Preserve	12	3	3	5	1 Phase 240 Volts	20	8" Ductile	2" Galv	4	4	4	2	4
30	Carriage Park Barnsdale	12	4	3	5	1 Phase 230 Volts	45	8" PVC	3" Ductile	4	4	4	2	4
31	Orchards	11	4	4	15	3 Phase 460 Volts	215	8" Ductile	6" Ductile	4	4	4	2	4
32	Carriage Park Deep Valley	11	4	4	7.5	3 Phase 460 Volts	20	8" PVC	3" Ductile	4	4	4	2	4
33	Carriage Park Wood Owl Ct	11	4	4	5	3 Phase 460 Volts	70	8" Ductile	3" Ductile	4	4	4	2	4
34	Sugarloaf School	11	4	4	7.5	3 Phase 460 Volts	125	8" PVC	4" Ductile	4	4	4	4	4
35	Carriage Park High Fields	11	4	4	5	3 Phase 460 Volts	40	8" Ductile	3" Ductile	4	4	4	2	4
36	Carriage Park Crest	11	4	4	15	3 Phase 460 Volts	80	8" PVC	3" Ductile	4	4	4	2	4
37	Carriage Park Dr.	10	4	2	7.5	3 Phase 230 Volts	79	8" PVC	3" Ductile	4	4	4	4	4
38	Carriage Park West	20	4	4	5	1 Phase 230 Volts	40	8 " Pvc	2 " Ductile	4	4	4	2	None
40	Adkinson School	2.5	4	4	5	3 Phase 460 Volts	80	2/ 8"PVC Lines	4" Ductile	4	4	4	4	4
	Etowah Plant Out Going Station		4	4	20	3 Phase 460 volts		6" PVC	4" Ductile	4	4	4	Dail-up	NA
	Etowah Plant In coming Station		4	4	7.5	3 Phase 230 volts		6"-8"-12" PVC	2"	3	4	1	Dail-up	NA
41	Etowah Reach	23	4	4	5 or 3	1 Phase 230 volts		Two 8' PVC	Rate 4/2"	4	4	4	Dail-up	NA
42	Johnathan Creek	16	4	3	5	1 Phase 230 volts		8"PVC	2"	4	4	4	Dail-up	NA
43	Sunset Ridge	16	4	4	2	1 Phase 230 volts		8" Ductile	2"	4	4	4	Dail-up	NA
44	The Meadows	28	3	4	3	3 Phase 200 volts		12"& 8" PVC	2"	4	3	3	Dail-up	NA
45	Home Place	23	3	4	3	1 Phase 230 volts		12" & 4" PVC	2"	4	4	4	Dail-up	NA
46	Brandy Mills	27	None	4	2	1 Phase 230 volts		12" PVC	2"	4	4	4	Dail-up	NA

Site ID #.	Site Name		Generator and ATS												
		Lighting	Manufacturer	Size, kW	Transfer Switch Type	Generator Rating	Transfer Switch Rating	Physical Disconnect	Wetwell Conition	Rail system	Chain	Floats	Hatch	Alarm Light	Alarm Sound
1	G E	4	None	NA	NA	NA	NA	None	4	NA	NA	NA	4	4	4
3	Garden Lane	NA	NA	NA	NA	NA	NA	YES	2	2	4	3	2	4	4
8	Browning Av.	NA	Cummins	75 KW	Cummins	4	4	YES	4	4	4	4	4	4	4
10	Outback	NA	Kato Light	30 KW	Asco	4	4	YES	4	4	4	4	4	4	4
11	Bonclarken	NA	Cummins	125	Cummins	4	4	YES	4	4	4	4	4	4	4
12	Highland lake Dr	NA	Cummins	20	Cummins	4	4	YES	4	4	4	4	4	4	4
14	Dunroy	NA	Kato Light	20	Asco	4	4	YES	4	4	4	4	4	4	4
15	Carriage Park Grapevine	NA	Cummins	125	Cummins	4	4	YES	4	4	4	4	4	4	4
16	Kenmure Driving Range	NA	Cummins	60	Cummins	1	4	YES	4	4	4	4	4	4	4
17	Carl Sandburgs	NA	Cummins	16	Cummins	4	4	YES	4	4	4	4	4	4	4
18	Kenmure Brookwood	NA	Cummins	35	Thomson	4	4	YES	4	3	4	4	4	4	4
19	Highland lake Golf	NA	Cummins	20	Cummins	4	4	YES	4	4	4	4	4	4	4
20	Leverette Dr	NA	None	NA	NA	NA	NA	YES	4	4	4	4	4	4	4
23	Lakewood Rv	NA	Coleman	17.5	Onan	4	4	YES	4	4	4	4	4	4	4
24	Shaws Creek	NA	Generac	20	Generac	4	4	YES	4	4	4	4	4	4	4
25	Dana School	NA	School Generator	NA	NA	NA	NA	YES	4	4	4	4	4	4	4
26	Clear Creek School	NA	School Generator	NA	NA	NA	NA	NO	4	4	4	4	4	4	4
28	Eagle Point	NA	Kholer	20	Thomson	4	4	YES	4	4	4	4	4	4	4
29	Carriage Park Preserve	NA	Generac	20	Generac	4	4	YES	4	4	4	4	4	4	4
30	Carriage Park Barnsdale	NA	Generac	20	Generac	4	4	YES	4	4	4	4	4	4	4
31	Orchards	NA	Cummins	60	Thomson	4	4	YES	4	4	4	4	4	4	4
32	Carriage Park Deep Valley	NA	Generac	45	Thomson	4	4	YES	4	4	4	4	4	1	4
33	Carriage Park Wood Owl Ct	NA	Generac	25	Thomson	4	4	YES	4	4	4	4	4	4	4
34	Sugarloaf School	NA	School Generator	Na	Na	Na	Na	No	4	4	4	4	4	4	4
35	Carriage Park High Fields	NA	Generac	25	Thomson	4	4	YES	4	4	4	4	4	4	4
36	Carriage Park Crest	NA	Generac	55	Thomson	4	4	YES	4	4	4	4	4	1	4
37	Carriage Park Dr.	NA	Cummins	35	Thomson	4	4	YES	4	4	4	4	4	4	4
38	Carriage Park West	NA	None	NA	NA	NA	NA	YES	4	3	4	4	3	4	2
40	Adkinson School	NA	School Generator	NA	NA	NA	NA	YES	4	4	4	4	4	4	4
	Etowah Plant Out Going Station	NA	Generac	125	Generac	4	4	None	4	4	4	4	4	4	1
	Etowah Plant In coming Station	NA	Generac	125	Generac	4	4	None	3	1	2	3	3	3	3
41	Etowah Reach	NA	None	None	None	None	None	None	4	4	4	4	4	1	1
42	Johnathan Creek	NA	Kohler	26	Kohler	4	4	None	4	4	4	4	4	4	4
43	Sunset Ridge	NA	Generac	20	Generac	4	4	None	4	4	4	4	4	4	2
44	The Meadows	NA	None	None	None	None	None	None	4	3	4	4	4	1	3
45	Home Place	NA	None	None	None	None	None	None	4	3	3	4	4	1	4
46	Brandy Mills	NA	None	None	None	None	None	None	4	3	3	4	4	4	1

Appendix H – Collection System Inspection Plan

1.0 Background

The condition assessment work in Phase 1 included smoke testing of 20,000 feet of sewer pipe. The Implementation of this inspection plan will provide the basis for the initial assessment of the condition of collection system and, based on these results, more detailed inspection can be conducted to provide an ongoing inspection program. The results for these inspections will provide information that can be used to determine the locations for possible capital improvement planning projects.

The gravity sewer network within the collection system is comprised of various pipe material and sizes as shown in Figure 1. The pipe material includes ductile/cast iron, clay, santite pipe, and PVC. A large quantity of the pipe material is unknown which limits the use of pipe material as a factor in selecting locations. The gravity collection system is composed of pipe ranging in diameter from 4-inch to 42-inch and the length of pipe based on the GIS information by diameter and material is shown in Table 1. There are a few pipe segments designated as "unknowns" or "blank" in the GIS database.

There are 5,285 manholes in the system. They are constructed of various materials including brick and precast concrete, and there are many listed as unknown in the GIS. There are 135 "flush" manholes that were constructed to allow the lines to be cleaned with a water tap inside, that has been disconnect and is no longer in service. There are 36 "drop" manholes that have either an inside or outside drop into the manhole. Also, 32 manholes are designated as "high speed" manholes. The remaining manholes are the standard construction.

There are 30 lift stations located throughout the system. The pump stations vary in size, but they are all submersible pumps in a wet well with floats for control of the water level.

There are about 32 miles of forcemains ranging in size from 2 to 12 inches constructed of various materials, primarily PVC, but some HDPE and ductile iron.

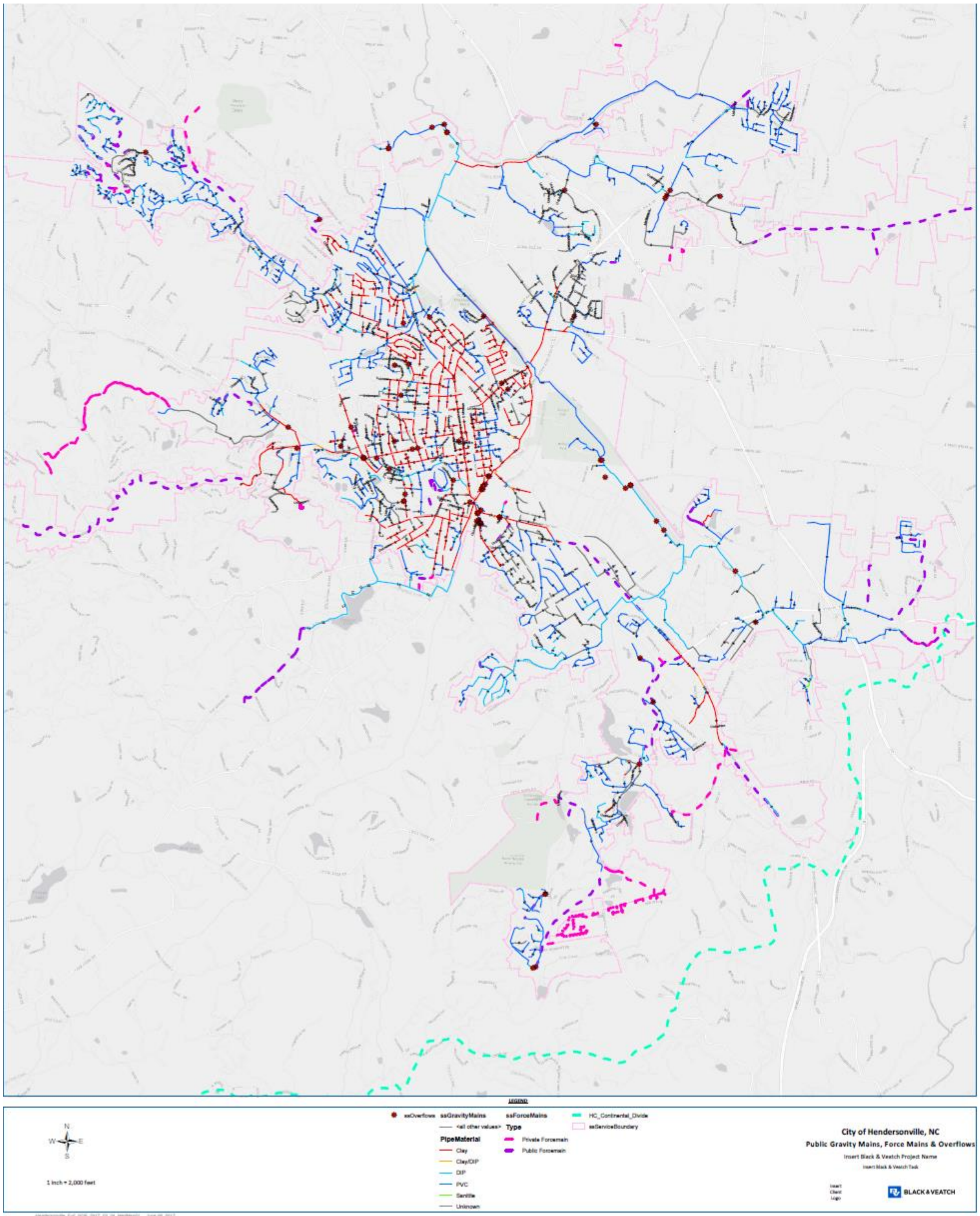


Figure 1 – Hendersonville Collection System

Table 1 – Summary of Collection System Piping

MATERIALS / DIAMETER	4	6	8	10	12	15	18	24	27	30	36	42	UNKNOWN	(BLANK)	TOTAL
Blank	183	524	202											12,419	13,328
Clay		24,773	106,507	13,699	11,969	8,258	5,894	8,131	3,028			2,077	365		184,700
Clay/DIP			469	605	789	817	691		1,132	201					4,704
DIP		657	67,338	3,964	4,769	1,365	16,022	8,146	16	27	28	4,062	1,398		107,793
Other	374														374
PVC	71	5,151	280,176	14,920	13,473	2,958	2,539	10,802					1,661		331,752
PVC/Clay		319	5,168	1,501	2,482	173	224		945						10,812
PVC/DIP			5,312	410	1,746	361									7,829
Sanitite HP					481										481
Unknown	26	4,406	97,906	962	1,229	648	1,140						66,276	188	172,781
Total	**654**	**35,830**	**563,078**	**36,062**	**36,939**	**14,580**	**26,510**	**27,079**	**5,121**	**228**	**28**	**6,140**	**69,700**	**12,606**	**834,555**

2.0 Identification of Inspection Methods

The results of these inspections will be used to estimate inflow and infiltration (I/I), and the location of possible blockages and structural defects so the overall condition assessment of the entire system can be estimated. More detailed inspections will be required to develop specific capital projects but these inspections will provide useful information in development of a plan for this work.

2.1 MANHOLE INSPECTION

The National Association of Sewer Service Companies (NASSCO) has developed comprehensive guidelines for inspection of sewer manholes. This plan is similar to those guidelines but designed to be simpler and collect information required for completing the inventory of the system.

The inspection will consist of visual observations with photographs taken from the surface. The City has developed a form that can be used to assist in gathering the data and to ensure consistency in the data collection.

The inspection will include an approximate measurement from the lip of the ring to the invert. This measurement is based upon the access to the invert with the measuring pole and will vary based on the angle and flow in the manhole.

2.2 SMOKE TESTING

The purpose of smoke testing is to identify defects that allow I/I or cross connections in the sewer pipe by forcing smoke into the pipe. The smoke is introduced into the pipe with a blower that seals a manhole and forces the smoke into the pipe. The smoke will be forced out of the pipe at cross connections with storm drains or cracks in the pipe joints or wall.

The City has the necessary equipment and experience to conduct smoke testing.

2.3 ACOUSTIC INSPECTION

The SL-RAT™ (SL-RAT) utilizes acoustic technology to quickly assess the degree of blockage in sewer lines less than 24-inch diameter. An acoustic transmitter is placed in one manhole and a receiver located in an adjacent manhole. The sound wave propagates in the air gap above the wastewater flow up to 800 feet. The strength of the received signal serves as an indication of the percent of blockage and can be measured in less than three minutes. The results from the acoustic testing are reported in a color-coded rating system from 0 to 9 with 0 being a total blockage and 9 being no blockage depicted directly on the pipeline alignment.

2.4 CCTV INSPECTION

The use of CCTV inspection provides visual documentation of the condition of the interior of the pipe. This can confirm the location of cracks found from smoke testing or blockages or sediment build up found from the acoustic testing. The CCTV data provides confirmation of the actual conditions in the pipe.

The CCTV will be a crawler unit with a camera that will be capable of pan and tilting with a zoom lens to provide detailed observation of defects in the pipe. The camera has a high sensitivity sensor with over 460 lines of resolution for low light locations, 10x optical zoom, a tilt range of 280 degrees and a 360 degrees continuous rotational range. The crawler operates at a speed of up to 30 feet per minute depending on the slope of the pipe and number of bends. The equipment operates in a dry, partially flooded or completely flooded pipe.

The City has the equipment and regularly conducts this work as part of normal operations and as needed in emergency response.

2.5 LIFT STATIONS

The City operates and maintains 30 lift stations as part of the collection system. The lift stations consist of submersible pumps in a wet well and range in age from 3 to 40 years. The City initiated an inspection program of the lift stations in 2016 and inspected all the station in 2017. The inspection included a pump test to validate operation of the pumps. There are 12 stations that do not have generators and the transfer switch systems in the remaining stations are in good condition. The lift stations are inspected on a regular basis and visually whenever a work order is issued that requires maintenance at the lift station. The lift stations are maintained through the work order system with preventative maintenance scheduled in accordance with manufacturers recommendations.

2.6 FORCEMAINS

The inspection of forcemains will vary depending upon the diameter and criticality of the pipeline. The larger diameter (6-inch and larger) and more critical forcemains should be inspected using a non-destructive testing (NDT) method for evaluating the pipe wall condition. The forcemains can also be tested using leak detection methods similar to those used in the water distribution system. This would include a pressure testing method used for new construction.

The forcemains should be inspected within the next 5 years to document their condition and determine if repair and/or replacement is required as part of the capital plan.

3.0 Conducting Inspections

The factors used to select the segments to inspect include previous smoke testing results, recent flow metering results, locations of SSO events, creek crossings, and experience with the various pipe materials.

This section will describe the criteria for prioritizing the inspection work and documenting the results. The documentation of the results is a critical step that will allow the inspections to identify concerns based on the data.

3.1 MANHOLE INSPECTION

The manhole inspection and inventory work is combined with the work order system. When standard work orders are issued that include a manhole, the data will be collected on the City's form as part of the work order. This approach will begin to address the data collection and inventory of the manholes.

Also, a dedicated crew can be used to inventory and gather the data as time permits. The intent would be when weather or other factors prevent other work from being completed this crew could work on manholes. The use of a dedicated crew would improve the consistency of the data and speed in which the data is collected. This crew would become familiar with the data collection method and picture taking.

Based on the pilot inspection conducted in April 2018, the average time for inventory and inspection of a manhole was 10 minutes. This schedule results in the completion of the inspection and inventory of the 5,285 manholes in 147 days, at 6 hours per day.

Pictures are taken to document the condition and allow for later review. The minimum pictures are:

1. The manhole number
2. Manhole location, in the street or easement
3. Manhole ring, lid and cover
4. Manhole cone and invert

The pictures are saved on the server in a folder for manholes. The recommended method is in a power point photo album with the manhole number in the title of the first slide so they can be easily found by searching for the number. The inspection form is submitted to Engineering so the data can be entered into the GIS database and GPS locating can be completed.

The form developed to collect the data on manholes is shown in Figure 2.

City of Hendersonville

Sanitary Sewer Manhole Inspection Sheet

Inspector___________________________ Basin: FM6

Date: _______________________________ Time: _____________________

Weather: (Heavy Rain, Light Rain, Snow, Saturated, Damp, Wet)

Manhole ID: _______________ Manhole Type (Sanitary, Force Main, Vented, Flush Tank)

Manhole Condition: (Good, Fair, Poor, Bad)

Manhole Rim/Lid Condition: (Good, Fair, Poor, Bad) (Standard, Vented, Locking)

Rim to Grade: (Ground Level, Raised) Height if Raised: ________________________

Access: (RR Easement, Street ROW, Easement, DOT ROW)

Surface Type: (Asphalt, Concrete, Gravel, Grass/Dirt, Forest)

Steps: (Yes, No) (Good, Fair, Poor, Bad) (Plastic, Metal)

Cone Condition: (Good, Fair, Poor, Bad) (Concrete, Brick, Lined) (Eccentric, Concentric, Flat Top)

Riser Condition: (Good, Fair, Poor, Bad) (Concrete, Brick, Lined)

Base Condition: (Good, Fair, Poor, Bad) (Concrete, Brick, Lined)

Bench Condition: (Missing, Good, Fair, Poor, Bad) (Concrete, Brick)

Invert Condition: (Missing, Good, Fair, Poor, Bad) (Concrete, Brick)

Invert (diameter & depth) (Measure in Inches and start at 12:00 with invert OUT and go

Clockwise on invert IN)

IN #1 ____________/____________ IN #2 ____________/____________

IN #3 ____________/____________ IN #4____________/____________

IN #5 ____________/____________ OUT ____________/____________

Inflow & Infiltration (Yes, No) Infiltration: (Stain, Dripper, Runner, Gusher,)

I & I Comments: _______________________ ____

____

General Comments: ____

____

____

Figure 2 - Manhole Data Collection Form

3.2 SMOKE TESTING

The City currently conducts smoke testing on a regular basis and have completed testing in many areas. The best smoke testing results are obtained when the soil surrounding the pipe is dry since it will allow the smoke to surface through the voids or cracks in the ground. Therefore it is recommended that smoke testing be conducted, when possible, in the summer months.

The selection of which areas to test is based upon potential for infiltration and inflow. This focuses on areas with known capacity limitations, areas where manholes have been noted with evidence of surcharge, and manhole in close proximity to streams and storm drainage. The recommended priorities for smoke testing areas are:

1. Stream or creek crossings
2. Proximity to large storm drainage infrastructure
3. Surcharged manholes
4. Clay pipe areas
5. Flow monitoring indication of capacity limitations

The GIS maps will be used to illustrate where smoke testing has occurred over the past 5 years so that the planned testing can cover the remaining areas adjacent to streams and storm drainage.

Smoke testing requires notification to the public and fire department in the affected areas prior to the actual testing. The addresses for property owners to be notified by door hangers will be collected from the GIS maps. The door hangers will be distributed approximately one or two days prior to the testing.

The inspection will document locations where smoke is observed coming from the ground, broken cleanouts or cross connections. These locations will be documented and pictures taken.

The flow metering data can also be used to identify areas for smoke testing. The flow metering covers a large area but can help identify sections where inflow is occurring. Conducting flow metering on a smaller drainage area can provide information to identify areas for smoke testing.

3.3 ACOUSTIC INSPECTION

The acoustic inspection with the SL-RAT™ (SL-RAT) should be conducted by a subcontractor for the initial testing. The recommended initial testing would be at least 20,000 feet in locations that are likely to have roots or sediment build up. This will allow the City the opportunity to see how this work is conducted and evaluate the data collected.

The estimated cost for this inspection is about $0.27 per foot and the work could be completed in about 4 days. Alternative pricing may be by manhole segment since the work is completed manhole to manhole. For the initial inspection, Black & Veatch would provide onsite observation and coordination with the subcontractor to provide documentation for the City.

3.4 CCTV INSPECTION

The CCTV inspection work is closely related to cleaning operations. The CCTV inspection that is required based upon acoustic testing results may require cleaning in conjunction with the inspection.

The CCTV inspection work should be prioritized based upon the results of the smoke testing and acoustic testing. These tests provide information that will make CCTV inspection more efficient by inspecting those areas with suspected defects or blockage.

The results of the CCTV inspections are valuable information to update the GIS inventory for pipe material. There is a significant amount of "unknown" in the GIS database for pipe material that should be addressed with the CCTV inspections.

3.5 LIFT STATIONS

The inspection of lift stations should continue to be included as a part of the routine maintenance. Also, the recently completed review of all the lift stations should be conducted every 3 years to ensure pumps and piping are inspected.

The planned addition of flow meters to the lift stations over the next few years will provide additional data that can be used to monitor the operating condition of the station.

In Phase 1 three lift stations, 011, 012, and 019, were visual inspected to evaluate the condition. As part of the continuation in Phase 2 an additional seven lift stations were evaluated in June 2018. The seven stations are 003, 008, 016, 018, 024, 037 and 038. The lift stations are generally in good condition. The following are observations made during the June inspection:

- 003 – Garden Lane; The wet well is elevated and difficult to access with only a ladder on the side of the wet well. There is no generator and the disconnect has not been updated.
- 008 – Browning Avenue; The valve vault was not accessible. Therefore, it was not possible to verify the condition of the piping and valves.
- 016 – Kenmure Driving Range; The lift station has a rain gauge that can be used to correlate rainfall with the wet well levels. The drain from the wet well allowed grease into the valve vault.

- 018 – Kenmure Brookwood; The wet well is fiberglass with no valve vault.
- 024 – Shaws Creek Farm; The discharge pipe is galvanized steel which has a potential for corrosion depending on the soil characteristics. There was erosion on the road leading to the station.
- 037 – Carriage Park Pigtrail; The wet well piping appeared to have some corrosion and the holding tanks have the potential for odor concerns. The hillside was observed to be sliding into the fence and was pressing on the gas meter for the generator. The bank above the station is undercut from the slope sliding down.
- 038 – Carriage West; There was no generator but the disconnects appeared to be upgraded. There is no valve vault so the piping was not visible.

3.6 FORCEMAINS

The work to inspect the forcemains will require a prioritization of the pipelines that includes pipe material, pipe size and criticality of the operations. The age of the pipeline is also a factor to consider in conducting condition assessment of the pipe. The prioritization is developed using a risk analysis that combines likelihood of failure and consequence of failure.

The recommended process for inspection of forcemains is based upon the priority of the forcemain. The high priority forcemains require more detailed inspections than the lower priority forcemains. The process is a phased approach using indirect testing methods such as soil corrosion potential to identify areas for inspection. Leak detection or pressure testing also provides useful information without disrupting operations. Based upon the indirect testing more direct testing may be required. The direct testing is non-destructive testing (NDT) on metallic pipe and removing samples for testing on plastic pipe.

The NDT methods include ultrasonic wall thickness testing for metallic pipe using A or B scan technology, Guided Wave Technology, or remote field technology for highly critical pipelines.

3.7 PUBLIC NOTIFICATION AND PERMITS

In the areas affected by the smoke testing the public should be notified through the use of door hangers distributed a couple of days prior to the smoke testing work. A list of property owners' names and addresses can be created from the GIS data in the area of the inspection.

The work should be coordinated with the local fire and police departments through the non-emergency dispatch to inform them of the work on a daily basis. The Deputy Fire Chief should be contacted directly each day of the inspection and as needed.

Public notification for CCTV, cleaning and other work provides an opportunity to inform the public on the operations that are being conducted in the area and helps improve community relations.

3.8 RECORDING INSPECTION RESULTS IN GIS

The process for transferring information from the inspection into the GIS database should be a simple process that can be executed in a short period of time. The information should be collected in the work order system as much as possible. The form used to collect the information on the manholes should be given to the engineering department for review and incorporation into the GIS.

The GIS database should address unknowns in pipe material and manhole type as the work is completed to reduce the number of unknowns. Also, maintaining the GIS will allow for the inspection work to be tracked to avoid duplication of work and to monitor for scheduling prioritization.

Coordinating this work with the CMMS work order system will provide a seamless integration of the data into the decision-making process.

4.0 Inspection Schedule

The following proposed schedule is preliminary and can be modified to meet the needs of the City's operations staff. The recommended locations are based upon a review of the criteria and are proposed for review by the City.

4.1 MANHOLE INSPECTION

The manhole inspection and inventory work was initiated in April and the process and data collection form developed. As the work proceeds, improvements can be made by using consistent staff to build experience.

The process was reviewed on June 28, 2018 after the review of the lift stations. Additional assistance will be as requested by the City. .

4.2 SMOKE TESTING

We recommend smoke testing be conducted in the summer because the saturation of the soil can impact the ability of the smoke to surface.

Using the criteria from Section 3, the recommended smoke testing would be in the following areas:

- Manhole 443 to Manhole 297
- Manhole 278 to Manhole 287
- Manhole 255 to Manhole 269

- Manhole 2476 to Manhole 2301
- Manhole 467 to Manhole 1297

4.3 ACOUSTIC TESTING

The acoustic testing is proposed to be conducted by a subcontractor as a "pilot project" to provide the City experience with the process. Based on this testing, additional testing can be provided by the subcontractor or the City can purchase the equipment and receive training. The acoustic testing is a preliminary screening tool and based on the occurrence of overflows in the area the recommended sub-basin areas for conducting this testing are Tebeau Dr., Thornton Place, 9th Ave West and Moss Valley Trail, Borest Hill Drive, Lugano Drive and Scheppergrell Drive. This area could be expanded to Blythe Street to cover a major portion of the sub-basin. Additional areas can be included if requested by the City and based on the amount of area to be covered in the work completed by the pilot project.

The City will be required to contact the subcontractor and schedule this work. This work should be scheduled prior to the CCTV inspection and it is recommended the work be scheduled for the week of August 13, 2018.

4.4 CCTV INSPECTION

The CCTV inspection should be based upon the results of the manhole inspections, smoke testing and acoustic monitoring. These inspections will identify areas where surcharges have occurred or if there are defects in the pipe or manhole. These defects can be confirmed and quantified with the CCTV.

The proposed dates for Black & Veatch to review the CCTV process is the week of September 10, 2018.

4.5 SUBCONTRACTORS

The coordination of the work with subcontractors and the City will reduce the time required to complete the work. We recognize the potential for delays due to access or in collecting the data. The use of a subcontractor can provide the additional manpower as needed. The areas identified for inspection can easily be adjusted if there are restrictions in access or, if the flows are not suitable for data collection. The lengths and locations of the inspection can be adjusted to meet conditions in the field and remain within the agreed distances.

The initial inspection work for acoustic testing should be conducted with a subcontractor. This will provide the City with a point of reference for this work and how effective it can be in evaluation of the collection system.

5.0 Data Collection and Reporting

The data shall be collected by the various technologies using the acceptable industry standards to ensure accurate and complete information is gathered. The smoke testing will provide indications of the condition of the pipeline regarding potential for I/I through cracks or cross connections. The acoustic testing is suitable for identifying potential blockages or capacity issues, and will be confirmed with CCTV. The CCTV will be used to confirm defects, I/I sources, and identified potential blockages. The CCTV will also provide information on the pipe material for sections with "unknown" information.

The process for transfer of the field data into the GIS record is critical for maintaining accurate records and planning future work.

It is vital that crewmembers keep complete and accurate field notes documenting each defect detected during the inspection. The following information should be recorded for each defect or inflow source detected:

- Description of defect.
- Street Address and GPS coordinates.
- Document whether the source is located on the city-maintained portion of the sewer system or on a private service line or private property.
- Estimate flow for the inflow source.

All of the above information shall be recorded in the work order issued for the work.

The field data will be used in preparation of additional work and provide recommendations for additional inspections to be scheduled.

6.0 Safety Plan

The inspection work should be conducted using safe practices. The primary hazards for this work will be traffic, slips, trips, or falls around access sites, exposure to weather and other related concerns. Open manholes are a specific concern that workers should be reminded of as this work is performed.

The workers shall wear reflective vests and hard hats when working in the roadway. The workers shall stay within the cones and use caution when crossing the roads.

Manned entry into the pipe is not anticipated for these inspections. However, if entry into the manholes is required for any reason, the work will require confined space entry compliance. The personnel entering the pipe must be confined space trained.

G-01					Gravity Sewer Main
Line Item	**Quantity**		**Unit Cost**		**Construction Cost**
30" sewer (3630 lf)	3,630	lf	$358.00	/lf	$1,299,500
Sewer Manhole (13)	13	ea	$8,000.00	/ea	$104,000
Major Roadway Crossing (210 lf)	210	lf	$1,500.00	/lf	$315,000
Secondary Road Crossing (220 lf)	220	lf	$800.00	/lf	$176,000
Stream Crossing (3)	3	ea	$20,000.00	/ea	$60,000
Erosion Control (3630 lf)	3,630	lf	$3.00	/lf	$10,900
Restoration (3630 lf)	3,630	lf	$2.50	/lf	$9,100
Subtotal					**$1,974,500**
Surveys, Record Documents, GPS Information					$9,900
General Requirements (10%)					$197,500
Contractor Fee (5%)					$98,700
Mobilization (3%)					$59,200
Construction Contingencies (10%)					$197,500
Total Construction Cost					**$2,537,300**
Scope Contingency					$507,000
Engineering Cost					$609,000
Pipeline Easement Cost					$16,300
Total Cost					**$3,669,600**

G-02			Gravity Sewer Main
Line Item	**Quantity**	**Unit Cost**	**Construction Cost**
24" sewer (1700 lf)	1,700 lf	$289.00 /lf	$491,300
Sewer Manhole (6)	6 ea	$6,500.00 /ea	$39,000
Erosion Control (1700 lf)	1,700 lf	$3.00 /lf	$5,100
Restoration (1700 lf)	1,700 lf	$2.50 /lf	$4,300
Subtotal			**$539,700**
Surveys, Record Documents, GPS Information			$2,700
General Requirements (10%)			$54,000
Contractor Fee (5%)			$27,000
Mobilization (3%)			$16,200
Construction Contingencies (10%)			$54,000
Total Construction Cost			**$693,600**
Scope Contingency			$139,000
Engineering Cost			$167,000
Pipeline Easement Cost			$7,700
Total Cost			**$1,007,300**

G-03			Gravity Sewer Main
Line Item	**Quantity**	**Unit Cost**	**Construction Cost**
15" sewer (4480 lf)	4,480 lf	$174.00 /lf	$779,500
Sewer Manhole (15)	15 ea	$6,500.00 /ea	$97,500
Secondary Road Crossing (550 lf)	550 lf	$800.00 /lf	$440,000
Erosion Control (4480 lf)	4,480 lf	$3.00 /lf	$13,400
Restoration (4480 lf)	4,480 lf	$2.50 /lf	$11,200
Subtotal			**$1,341,600**
Surveys, Record Documents, GPS Information			$6,700
General Requirements (10%)			$134,200
Contractor Fee (5%)			$67,100
Mobilization (3%)			$40,200
Construction Contingencies (10%)			$134,200
Total Construction Cost			**$1,724,000**
Scope Contingency			$345,000
Engineering Cost			$414,000
Pipeline Easement Cost			$20,200
Total Cost			**$2,503,200**

G-04			Gravity Sewer Main
Line Item	**Quantity**	**Unit Cost**	**Construction Cost**
36" sewer (6310 lf)	6,310 lf	$443.00 /lf	$2,795,300
54" sewer (1180 lf)	1,180 lf	$760.00 /lf	$896,800
Sewer Manhole (25)	25 ea	$8,000.00 /ea	$200,000
Secondary Road Crossing (110 lf)	110 lf	$800.00 /lf	$88,000
Stream Crossing (1)	1 ea	$20,000.00 /ea	$20,000
Erosion Control (7490 lf)	7,490 lf	$3.00 /lf	$22,500
Restoration (7490 lf)	7,490 lf	$2.50 /lf	$18,700
Subtotal			**$4,041,300**
Surveys, Record Documents, GPS Information			$20,200
General Requirements (10%)			$404,100
Contractor Fee (5%)			$202,100
Mobilization (3%)			$121,200
Construction Contingencies (10%)			$404,100
Total Construction Cost			**$5,193,000**
Scope Contingency			$1,039,000
Engineering Cost			$1,246,000
Pipeline Easement Cost			$33,700
Total Cost			**$7,511,700**

G-05			Gravity Sewer Main
Line Item	**Quantity**	**Unit Cost**	**Construction Cost**
21" sewer (3070 lf)	3,070 lf	$249.00 /lf	$764,400
Sewer Manhole (11)	11 ea	$6,500.00 /ea	$71,500
Secondary Road Crossing (515 lf)	515 lf	$800.00 /lf	$412,000
Stream Crossing (1)	1 ea	$20,000.00 /ea	$20,000
Erosion Control (3070 lf)	3,070 lf	$3.00 /lf	$9,200
Restoration (3070 lf)	3,070 lf	$2.50 /lf	$7,700
Subtotal			**$1,284,800**
Surveys, Record Documents, GPS Information			$6,400
General Requirements (10%)			$128,500
Contractor Fee (5%)			$64,200
Mobilization (3%)			$38,500
Construction Contingencies (10%)			$128,500
Total Construction Cost			**$1,650,900**
Scope Contingency			$330,000
Engineering Cost			$396,000
Pipeline Easement Cost			$13,800
Total Cost			**$2,390,700**

G-06					Gravity Sewer Main
Line Item	Quantity		Unit Cost		Construction Cost
21" sewer (3150 lf)	3,150	lf	$249.00	/lf	$784,400
36" sewer (6820 lf)	6,820	lf	$443.00	/lf	$3,021,300
Sewer Manhole (34)	34	ea	$6,500.00	/ea	$221,000
Secondary Road Crossing (600 lf)	600	lf	$800.00	/lf	$480,000
Railroad Crossing (200 lf)	200	lf	$2,000.00	/lf	$400,000
Erosion Control (9970 lf)	9,970	lf	$3.00	/lf	$29,900
Restoration (9970 lf)	9,970	lf	$2.50	/lf	$24,900
Subtotal					**$4,961,500**
Surveys, Record Documents, GPS Information					$24,800
General Requirements (10%)					$496,200
Contractor Fee (5%)					$248,100
Mobilization (3%)					$148,800
Construction Contingencies (10%)					$496,200
Total Construction Cost					**$6,375,600**
Scope Contingency					$1,275,000
Engineering Cost					$1,530,000
Pipeline Easement Cost					$44,900
Total Cost					**$9,225,500**

G-07					Gravity Sewer Main
Line Item	Quantity		Unit Cost		Construction Cost
18" sewer (4320 lf)	4,320	lf	$209.00	/lf	$902,900
Sewer Manhole (15)	15	ea	$6,500.00	/ea	$97,500
Secondary Road Crossing (220 lf)	220	lf	$800.00	/lf	$176,000
Stream Crossing (1)	1	ea	$20,000.00	/ea	$20,000
Erosion Control (4320 lf)	4,320	lf	$3.00	/lf	$13,000
Restoration (4320 lf)	4,320	lf	$2.50	/lf	$10,800
Subtotal					**$1,220,200**
Surveys, Record Documents, GPS Information					$6,100
General Requirements (10%)					$122,000
Contractor Fee (5%)					$61,000
Mobilization (3%)					$36,600
Construction Contingencies (10%)					$122,000
Total Construction Cost					**$1,567,900**
Scope Contingency					$314,000
Engineering Cost					$376,000
Pipeline Easement Cost					$19,400
Total Cost					**$2,277,300**

G-08			Gravity Sewer Main
Line Item	**Quantity**	**Unit Cost**	**Construction Cost**
21" sewer (4150 lf)	4,150 lf	$249.00 /lf	$1,033,400
Sewer Manhole (14)	14 ea	$6,500.00 /ea	$91,000
Secondary Road Crossing (660 lf)	660 lf	$800.00 /lf	$528,000
Stream Crossing (4)	4 ea	$20,000.00 /ea	$80,000
Erosion Control (4150 lf)	4,150 lf	$3.00 /lf	$12,500
Restoration (4150 lf)	4,150 lf	$2.50 /lf	$10,400
Subtotal			**$1,755,300**
Surveys, Record Documents, GPS Information			$8,800
General Requirements (10%)			$175,500
Contractor Fee (5%)			$87,800
Mobilization (3%)			$52,700
Construction Contingencies (10%)			$175,500
Total Construction Cost			**$2,255,600**
Scope Contingency			$451,000
Engineering Cost			$541,000
Pipeline Easement Cost			$18,700
Total Cost			**$3,266,300**

G-09			Gravity Sewer Main
Line Item	**Quantity**	**Unit Cost**	**Construction Cost**
15" sewer (1950 lf)	1,950 lf	$174.00 /lf	$339,300
Sewer Manhole (7)	7 ea	$6,500.00 /ea	$45,500
Secondary Road Crossing (330 lf)	330 lf	$800.00 /lf	$264,000
Stream Crossing (1)	1 ea	$20,000.00 /ea	$20,000
Erosion Control (1950 lf)	1,950 lf	$3.00 /lf	$5,900
Restoration (1950 lf)	1,950 lf	$2.50 /lf	$4,900
Subtotal			**$679,600**
Surveys, Record Documents, GPS Information			$3,400
General Requirements (10%)			$68,000
Contractor Fee (5%)			$34,000
Mobilization (3%)			$20,400
Construction Contingencies (10%)			$68,000
Total Construction Cost			**$873,400**
Scope Contingency			$175,000
Engineering Cost			$210,000
Pipeline Easement Cost			$8,800
Total Cost			**$1,267,200**

G-10			Gravity Sewer Main
Line Item	**Quantity**	**Unit Cost**	**Construction Cost**
30" sewer (5970 lf)	5,970 lf	$358.00 /lf	$2,137,300
Sewer Manhole (20)	20 ea	$8,000.00 /ea	$160,000
Secondary Road Crossing (330 lf)	330 lf	$800.00 /lf	$264,000
Stream Crossing (2)	2 ea	$20,000.00 /ea	$40,000
Erosion Control (5970 lf)	5,970 lf	$3.00 /lf	$17,900
Restoration (5970 lf)	5,970 lf	$2.50 /lf	$14,900
Subtotal			**$2,634,100**
Surveys, Record Documents, GPS Information			$13,200
General Requirements (10%)			$263,400
Contractor Fee (5%)			$131,700
Mobilization (3%)			$79,000
Construction Contingencies (10%)			$263,400
Total Construction Cost			**$3,384,800**
Scope Contingency			$677,000
Engineering Cost			$812,000
Pipeline Easement Cost			$26,900
Total Cost			**$4,900,700**

G-11			Gravity Sewer Main
Line Item	**Quantity**	**Unit Cost**	**Construction Cost**
24" sewer (4810 lf)	4,810 lf	$289.00 /lf	$1,390,100
Sewer Manhole (17)	17 ea	$6,500.00 /ea	$110,500
Secondary Road Crossing (185 lf)	185 lf	$800.00 /lf	$148,000
Stream Crossing (1)	1 ea	$20,000.00 /ea	$20,000
Erosion Control (4810 lf)	4,810 lf	$3.00 /lf	$14,400
Restoration (4810 lf)	4,810 lf	$2.50 /lf	$12,000
Subtotal			**$1,695,000**
Surveys, Record Documents, GPS Information			$8,500
General Requirements (10%)			$169,500
Contractor Fee (5%)			$84,800
Mobilization (3%)			$50,900
Construction Contingencies (10%)			$169,500
Total Construction Cost			**$2,178,200**
Scope Contingency			$436,000
Engineering Cost			$523,000
Pipeline Easement Cost			$21,600
Total Cost			**$3,158,800**

G-12			Gravity Sewer Main
Line Item	**Quantity**	**Unit Cost**	**Construction Cost**
15" sewer (1530 lf)	1,530 lf	$174.00 /lf	$266,200
18" sewer (1640 lf)	1,640 lf	$209.00 /lf	$342,800
Sewer Manhole (11)	11 ea	$6,500.00 /ea	$71,500
Major Roadway Crossing (200 lf)	200 lf	$1,500.00 /lf	$300,000
Secondary Road Crossing (110 lf)	110 lf	$800.00 /lf	$88,000
Stream Crossing (1)	1 ea	$20,000.00 /ea	$20,000
Erosion Control (3170 lf)	3,170 lf	$3.00 /lf	$9,500
Restoration (3170 lf)	3,170 lf	$2.50 /lf	$7,900
Subtotal			**$1,105,900**
Surveys, Record Documents, GPS Information			$5,500
General Requirements (10%)			$110,600
Contractor Fee (5%)			$55,300
Mobilization (3%)			$33,200
Construction Contingencies (10%)			$110,600
Total Construction Cost			**$1,421,100**
Scope Contingency			$284,000
Engineering Cost			$341,000
Pipeline Easement Cost			$14,300
Total Cost			**$2,060,400**

EX-01			Gravity Sewer Main
Line Item	**Quantity**	**Unit Cost**	**Construction Cost**
24" sewer (14500 lf)	14,500 lf	$289.00 /lf	$4,190,500
Sewer Manhole (49)	49 ea	$6,500.00 /ea	$318,500
Erosion Control (14500 lf)	14,500 lf	$3.00 /lf	$43,500
Restoration (14500 lf)	14,500 lf	$2.50 /lf	$36,300
Subtotal			**$4,588,800**
Surveys, Record Documents, GPS Information			$22,900
General Requirements (10%)			$458,900
Contractor Fee (5%)			$229,400
Mobilization (3%)			$137,700
Construction Contingencies (10%)			$458,900
Total Construction Cost			**$5,896,600**
Scope Contingency			$1,179,000
Engineering Cost			$1,415,000
Pipeline Easement Cost			$65,300
Total Cost			**$8,555,900**

EX-02			Gravity Sewer Main
Line Item	**Quantity**	**Unit Cost**	**Construction Cost**
18" sewer (14500 lf)	14,500 lf	$209.00 /lf	$3,030,500
Sewer Manhole (49)	49 ea	$6,500.00 /ea	$318,500
Erosion Control (14500 lf)	14,500 lf	$3.00 /lf	$43,500
Restoration (14500 lf)	14,500 lf	$2.50 /lf	$36,300
Subtotal			**$3,428,800**
Surveys, Record Documents, GPS Information			$17,100
General Requirements (10%)			$342,900
Contractor Fee (5%)			$171,400
Mobilization (3%)			$102,900
Construction Contingencies (10%)			$342,900
Total Construction Cost			**$4,406,000**
Scope Contingency			$881,000
Engineering Cost			$1,057,000
Pipeline Easement Cost			$65,300
Total Cost			**$6,409,300**

EX-03			Gravity Sewer Main
Line Item	**Quantity**	**Unit Cost**	**Construction Cost**
8" sewer (12000 lf)	12,000 lf	$134.00 /lf	$1,608,000
Sewer Manhole (40)	40 ea	$4,000.00 /ea	$160,000
Erosion Control (12000 lf)	12,000 lf	$3.00 /lf	$36,000
Restoration (12000 lf)	12,000 lf	$2.50 /lf	$30,000
Subtotal			**$1,834,000**
Surveys, Record Documents, GPS Information			$9,200
General Requirements (10%)			$183,400
Contractor Fee (5%)			$91,700
Mobilization (3%)			$55,000
Construction Contingencies (10%)			$183,400
Total Construction Cost			**$2,356,700**
Scope Contingency			$471,000
Engineering Cost			$566,000
Pipeline Easement Cost			$54,000
Total Cost			**$3,447,700**

EX-04			Gravity Sewer Main
Line Item	**Quantity**	**Unit Cost**	**Construction Cost**
10" sewer (10000 lf)	10,000 lf	$134.00 /lf	$1,340,000
Sewer Manhole (34)	34 ea	$4,000.00 /ea	$136,000
Erosion Control (10000 lf)	10,000 lf	$3.00 /lf	$30,000
Restoration (10000 lf)	10,000 lf	$2.50 /lf	$25,000
Subtotal			**$1,531,000**
Surveys, Record Documents, GPS Information			$7,700
General Requirements (10%)			$153,100
Contractor Fee (5%)			$76,600
Mobilization (3%)			$45,900
Construction Contingencies (10%)			$153,100
Total Construction Cost			**$1,967,400**
Scope Contingency			$393,000
Engineering Cost			$472,000
Pipeline Easement Cost			$45,000
Total Cost			**$2,877,400**

EX-05			Gravity Sewer Main
Line Item	**Quantity**	**Unit Cost**	**Construction Cost**
18" sewer (8000 lf)	8,000 lf	$209.00 /lf	$1,672,000
Sewer Manhole (27)	27 ea	$6,500.00 /ea	$175,500
Erosion Control (8000 lf)	8,000 lf	$3.00 /lf	$24,000
Restoration (8000 lf)	8,000 lf	$2.50 /lf	$20,000
Subtotal			**$1,891,500**
Surveys, Record Documents, GPS Information			$9,500
General Requirements (10%)			$189,200
Contractor Fee (5%)			$94,600
Mobilization (3%)			$56,700
Construction Contingencies (10%)			$189,200
Total Construction Cost			**$2,430,700**
Scope Contingency			$243,000
Engineering Cost			$535,000
Pipeline Easement Cost			$36,000
Total Cost			**$3,244,700**

EX-06			Gravity Sewer Main
Line Item	**Quantity**	**Unit Cost**	**Construction Cost**
8" sewer (6500 lf)	6,500 lf	$134.00 /lf	$871,000
Sewer Manhole (22)	22 ea	$4,000.00 /ea	$88,000
Erosion Control (6500 lf)	6,500 lf	$3.00 /lf	$19,500
Restoration (6500 lf)	6,500 lf	$2.50 /lf	$16,300
Subtotal			**$994,800**
Surveys, Record Documents, GPS Information			$5,000
General Requirements (10%)			$99,500
Contractor Fee (5%)			$49,700
Mobilization (3%)			$29,800
Construction Contingencies (10%)			$99,500
Total Construction Cost			**$1,278,300**
Scope Contingency			$256,000
Engineering Cost			$307,000
Pipeline Easement Cost			$29,300
Total Cost			**$1,870,600**

PS-01			Pump Station
Line Item	**Quantity**	**Unit Cost**	**Construction Cost**
8" sewer (1800 lf)	1,800 lf	$134.00 /lf	$241,200
Sewer Manhole (6)	6 ea	$4,000.00 /ea	$24,000
Erosion Control (1800 lf)	1,800 lf	$3.00 /lf	$5,400
Restoration (1800 lf)	1,800 lf	$2.50 /lf	$4,500
Subtotal			**$275,100**
Surveys, Record Documents, GPS Information			$1,400
General Requirements (10%)			$27,500
Contractor Fee (5%)			$13,800
Mobilization (3%)			$8,300
Construction Contingencies (10%)			$27,500
Total Construction Cost			**$353,600**
Scope Contingency			$71,000
Engineering Cost			$85,000
Pipeline Easement Cost			$8,100
Total Cost			**$517,700**

PS-02			Pump Station
Line Item	**Quantity**	**Unit Cost**	**Construction Cost**
10" sewer (8000 lf)	8,000 lf	$134.00 /lf	$1,072,000
Sewer Manhole (27)	27 ea	$4,000.00 /ea	$108,000
Stream Crossing (1)	1 ea	$20,000.00 /ea	$20,000
Erosion Control (8000 lf)	8,000 lf	$3.00 /lf	$24,000
Restoration (8000 lf)	8,000 lf	$2.50 /lf	$20,000
Subtotal			**$1,244,000**
Surveys, Record Documents, GPS Information			$6,200
General Requirements (10%)			$124,400
Contractor Fee (5%)			$62,200
Mobilization (3%)			$37,300
Construction Contingencies (10%)			$124,400
Total Construction Cost			**$1,598,500**
Scope Contingency			$320,000
Engineering Cost			$384,000
Pipeline Easement Cost			$36,000
Total Cost			**$2,338,500**

PS-03			Pump Station
Line Item	**Quantity**	**Unit Cost**	**Construction Cost**
8" sewer (1200 lf)	1,200 lf	$134.00 /lf	$160,800
Sewer Manhole (4)	4 ea	$4,000.00 /ea	$16,000
Stream Crossing (1)	1 ea	$20,000.00 /ea	$20,000
Erosion Control (1200 lf)	1,200 lf	$3.00 /lf	$3,600
Restoration (1200 lf)	1,200 lf	$2.50 /lf	$3,000
Subtotal			**$203,400**
Surveys, Record Documents, GPS Information			$1,000
General Requirements (10%)			$20,300
Contractor Fee (5%)			$10,200
Mobilization (3%)			$6,100
Construction Contingencies (10%)			$20,300
Total Construction Cost			**$261,300**
Scope Contingency			$52,000
Engineering Cost			$63,000
Pipeline Easement Cost			$5,400
Total Cost			**$381,700**

PS-04			Pump Station
Line Item	**Quantity**	**Unit Cost**	**Construction Cost**
8" sewer (2300 lf)	2,300 lf	$134.00 /lf	$308,200
Sewer Manhole (8)	8 ea	$4,000.00 /ea	$32,000
Secondary Road Crossing (110 lf)	110 lf	$800.00 /lf	$88,000
Stream Crossing (1)	1 ea	$20,000.00 /ea	$20,000
Erosion Control (2300 lf)	2,300 lf	$3.00 /lf	$6,900
Restoration (2300 lf)	2,300 lf	$2.50 /lf	$5,800
Subtotal			**$460,900**
Surveys, Record Documents, GPS Information			$2,300
General Requirements (10%)			$46,100
Contractor Fee (5%)			$23,000
Mobilization (3%)			$13,800
Construction Contingencies (10%)			$46,100
Total Construction Cost			**$592,200**
Scope Contingency			$118,000
Engineering Cost			$142,000
Pipeline Easement Cost			$10,400
Total Cost			**$862,600**

PS-05			Pump Station
Line Item	**Quantity**	**Unit Cost**	**Construction Cost**
8" sewer (2200 lf)	2,200 lf	$134.00 /lf	$294,800
Sewer Manhole (8)	8 ea	$4,000.00 /ea	$32,000
Stream Crossing (1)	1 ea	$20,000.00 /ea	$20,000
Erosion Control (2200 lf)	2,200 lf	$3.00 /lf	$6,600
Restoration (2200 lf)	2,200 lf	$2.50 /lf	$5,500
Subtotal			**$358,900**
Surveys, Record Documents, GPS Information			$1,800
General Requirements (10%)			$35,900
Contractor Fee (5%)			$17,900
Mobilization (3%)			$10,800
Construction Contingencies (10%)			$35,900
Total Construction Cost			**$461,200**
Scope Contingency			$92,000
Engineering Cost			$111,000
Pipeline Easement Cost			$9,900
Total Cost			**$674,100**

FRAZIER
ENGINEERING P.A.

CITY OF HENDERSONVILLE

FLOW MONITORING REPORT

JUNE 12, 2017

6592 Bob White Trail Stanley, NC 2816 | Office: 704.822.8444 | Fax: 704.822.8666

www.Frazier-Engineering.com

HENDERSONVILLE
FLOW MONITORING REPORT
JUNE 12, 2017

In accordance with a professional services agreement with Black & Veatch International Company, Frazier Engineering monitored wastewater flow at eight sites in the City of Hendersonville's service area. This report summarizes the flow monitoring work.

Monitor Location and Monitoring Period

Figure 1 shows the locations of flow meters within Hendersonville's collection system. Table 1 lists the manhole numbers and pipe sizes that were metered in addition to the date when each meter was installed and removed.

Table 1. Temporary Meter Sites

Site	Manhole	Pipe Size	Pipe Material	Location Description	Installation Date	Removal Date
1	3836	18	PVC	outfall west of Clear Creek Road, north of Carolina Village Road	2/20/17	5/23/17
2	196	42	DIP	outfall west of Pinehurst Drive	2/21/17	5/23/17
3	2008	24	DIP	outfall east of Asheville Highway, near Oakhurst Street	2/20/17	5/23/17
4	1476	12	VCP	outfall west of Orleans Avenue, south of Whitmire Circle	2/20/17	5/23/17
5	2278	24	VCP	outfall south of 1st Avenue East, upstream of the Jackson Park force main discharge	2/20/17	5/23/17
6	917	18	PVC	outfall crossing West Allen Street	2/20/17	5/23/17
7	2773	24	DIP	south of New Hope Road, near Powell Street	2/20/17	5/23/17
8	3792	16.5	DIP	southwest of Spartanburg Highway, southeast of Shepard Street, near the abandoned Rhodys pump station	2/20/17	5/23/17

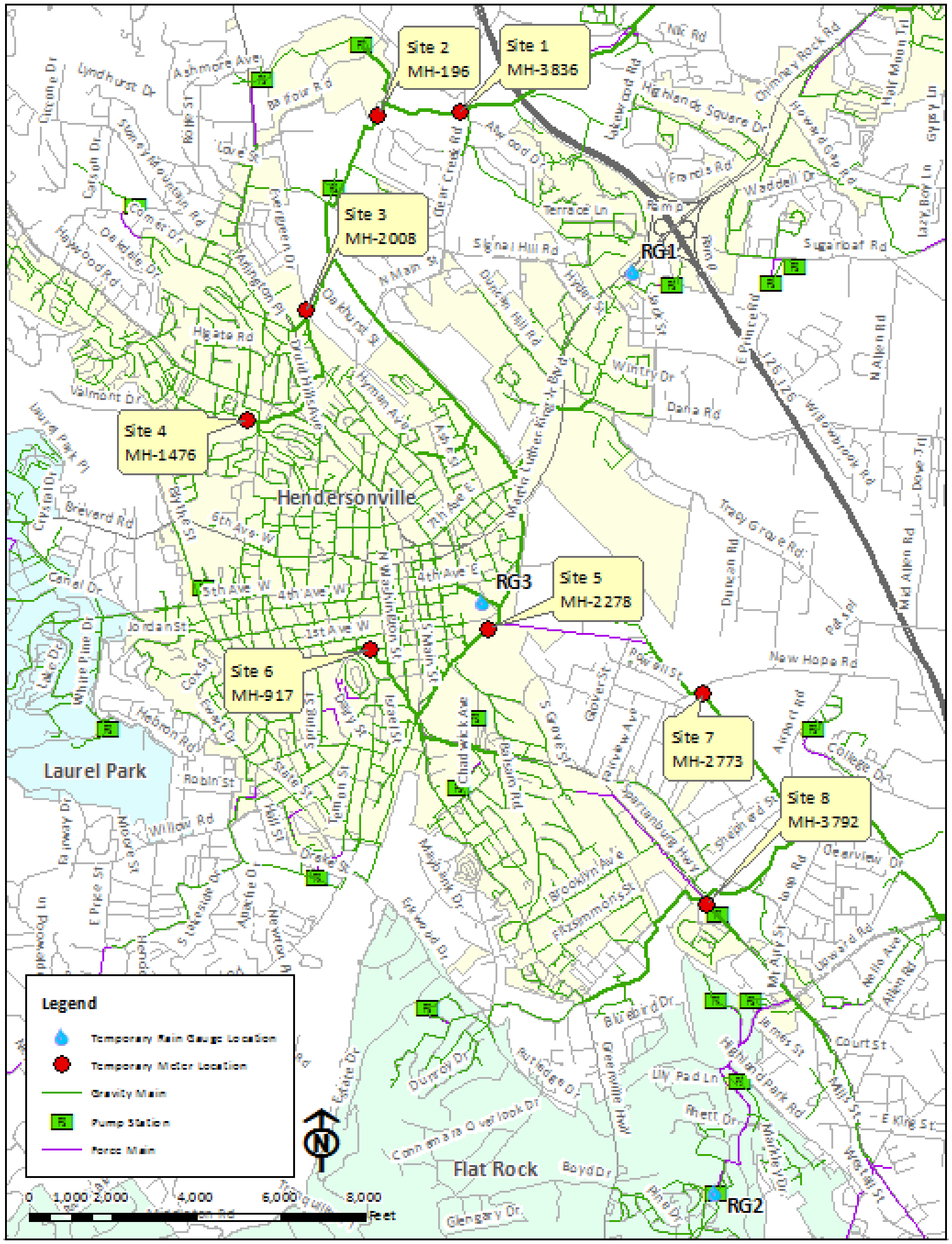


Figure 1. Flow Meter Locations

Flow Meter Information
The temporary meters installed and maintained by Frazier Engineering were Sigma 920 meters with submerged area-velocity sensors. The 920 meter measures average velocity using twin piezoelectric crystals utilizing ultrasonic one-MHz Doppler technology. Multiple measurements are taken by bouncing the Doppler signal off any and all particulates found throughout the flow stream and then averaged. Flow depth is measured using a pressure transducer.

Flow Meter Installation, Calibration, and Maintenance
The sensor for each meter was installed at the 6 o'clock position of the incoming sewers of the manholes listed in Table 1, except for the meter at Site 5. The sensor at Site 5 was rotated to the 5 o'clock position due to approximately four inches of silt in the pipe. Each meter was calibrated at installation by adjusting the depth of flow recorded by the meter to match a manual depth measurement. Meters were set up to record depth and velocity at 15-minute intervals.

Each meter was visited periodically to download the data, to perform any necessary maintenance (such as scrubbing sensors to remove debris), and to calibrate the meters per the methodology outlined above. Data was reviewed on site for overall data quality and any problems were immediately addressed.

Equipment and site information is provided below. If a specific site is not listed, no notable equipment or site issues occurred during the study period.

- Site 1: The sensor was replaced on April 26.

- Site 2: This site could not be accessed on April 26 due to flooded conditions surrounding the manhole.

- Site 5: The sensor was rotated to 4 o'clock on March 29. Silt was noted on February 28 and April 10. Heavy gravel and silt were noted on March 13. Rocks and sand were noted on May 8.

- Site 6: The sensor was replaced on March 29. Heavy gravel was noted on April 10.

- Site 8: Grease was scrubbed off the sensor on March 29.

Average Flow During Monitoring Period
Average daily flows facilitate capacity analyses and decisions on whether the sewers can handle additional flow. Dry-weather flows are directly compared with flows during rain events, and the difference between these flows is the estimated infiltration/inflow (I/I) volume entering the system.

Table 2 summarizes the average depth, velocity, and flow during the monitoring period. The sites are listed from the upstream-most monitoring point to the downstream-most monitoring point in Table 2. No flow balancing issues resulted when comparing flow averages during this time period.

Table 2. Average Depth, Velocity, and Flow Summary

Site	Pipe Diameter (in)	Primary Outfall				Tributary Outfalls			
		Depth (in)	% Pipe Diameter	Velocity (fps)	Flow (mgd)	Depth (in)	% Pipe Diameter	Velocity (fps)	Flow (mgd)
6	18	5.08	28%	2.02	0.55				
5	24	14.86	62%	0.74	0.98				
8	16.5					9.22	56%	1.32	0.24
7	24					5.23	22%	2.03	0.69
4	12					2.93	24%	2.04	0.41
3	24					5.02	21%	1.87	0.59
2	42	8.36	20%	3.36	2.99				
1	18					4.28	24%	2.03	0.44

Wet-Weather Flow During Monitoring Period

The five events that caused the largest responses in the collection system during the monitoring period were evaluated. Table 3 summarizes these events from each of the three rain gauges.

Table 3. Rain Events Summary

Date	Rain Gauge	Total Rain (in)	Peak Intensity (in/hr)	Duration (hrs:min)
March 30 - 31, 2017	RG1	3.68	1.24	11:00
	RG2	2.55	0.75	11:15
	RG3	3.00	1.16	11:15
April 3, 2017	RG1	2.03	0.86	5:30
	RG2	2.17	1.03	5:30
	RG3	2.02	0.82	5:45
April 23 - 24, 2017	RG1	2.41	0.40	22:00
	RG2	2.52	0.28	22:00
	RG3	2.02	0.26	21:30
May 4 - 5, 2017	RG1	2.16	0.84	12:15
	RG2	1.98	0.88	11:30
	RG3	1.93	0.72	11:30
May 21, 2017	RG1	2.53	0.41	17:45
	RG2	2.62	0.47	18:30
	RG3	2.34	0.35	18:15

Figure 2 graphically represents these rain events in comparison to 1-year, 2-year, and 5-year average recurrence rainfall intervals. The average recurrence interval information was obtained from “Precipitation-Frequency Atlas of the United States” NOAA Atlas 14, Volume 2, Version 3 for the Hendersonville, North Carolina area.

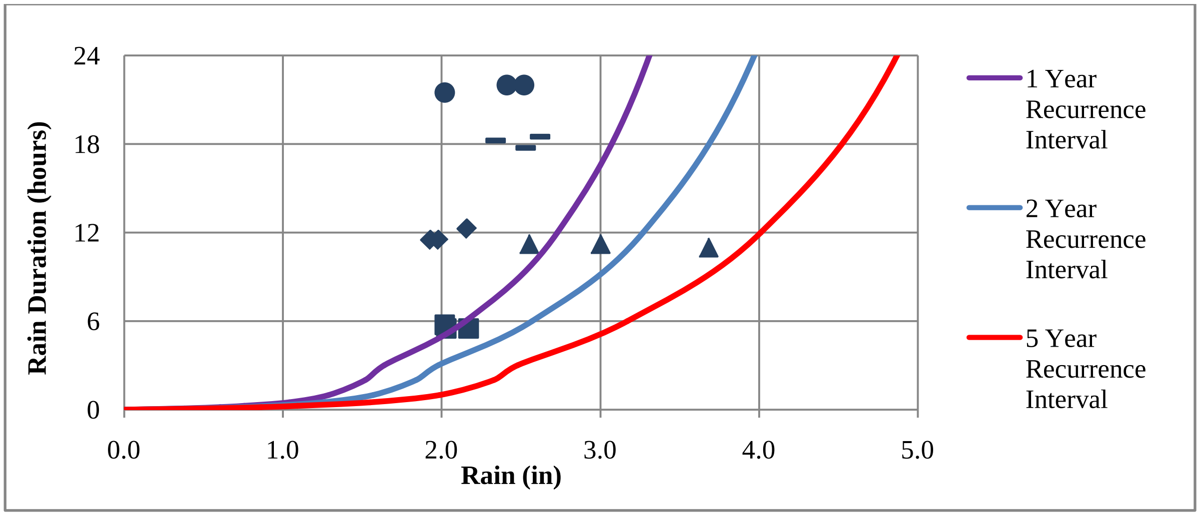


Figure 2: Rain Events Compared to Return Interval Frequencies

The late March rain event (represented by triangles) had the highest recurrence interval and the greatest variability between the three gauges - rainfall at RG2 had less than a 1-year return and the rainfall at RG1 had between a 2-year and 5-year return. The April 3, 2017 rainfall (represented by squares) was the next most significant event during the study period. All three rain gauges recorded approximately a 1-year recurrence interval for the early April event. The late April rainfall (represented by circles) had less than a 1-year recurrence interval, but produced the highest estimated I/I volumes during the study period. Table 4 lists the estimated I/I volume at each site for these three storms along with the duration of the response and the peak depth. Sites that experienced surcharged conditions have peak depth values that are highlighted in yellow.

Table 4a. Response to the March 30 - 31, 2017 Rain Event

Site	Pipe Diameter (in)	Primary Outfall				Tributary Outfalls			
		Estimated I/I (gal)	Duration (hr:min)	Peak Depth (in)	Peak Flow (mgd)	Estimated I/I (gal)	Duration (hr:min)	Peak Depth (in)	Peak Flow (mgd)
6	18	1,794,558	48:00	49.58	3.997				
5	24	2,860,230	51:45	91.85	5.580				
8	16.5					344,778	31:15	8.99	1.407
7	24					1,201,582	58:00	84.16	3.511
4	12					434,286	32:45	5.75	1.407
3	24					1,322,473	35:15	67.29	3.796
2	42	5,218,524	61:30	138.18	12.205				
1	18					362,068	36:00	122.30	1.840

Table 4b. Response to the April 3, 2017 Rain Event

Site	Pipe Diameter (in)	Primary Outfall				Tributary Outfalls			
		Estimated I/I (gal)	Duration (hr:min)	Peak Depth (in)	Peak Flow (mgd)	Estimated I/I (gal)	Duration (hr:min)	Peak Depth (in)	Peak Flow (mgd)
6	18	1,251,042	42:45	53.69	4.112				
5	24	2,124,231	44:30	82.92	5.680				
8	16.5					240,793	18:30	9.22	1.369
7	24					1,011,596	47:00	67.30	4.073
4	12					318,192	28:15	5.92	1.486
3	24					1,179,185	45:00	51.91	4.224
2	42	3,348,779	45:00	138.56	9.147				
1	18					187,187	30:45	122.47	1.557

Table 4c. Response to the April 23 - 24, 2017 Rain Event

Site	Pipe Diameter (in)	Primary Outfall				Tributary Outfalls			
		Estimated I/I (gal)	Duration (hr:min)	Peak Depth (in)	Peak Flow (mgd)	Estimated I/I (gal)	Duration (hr:min)	Peak Depth (in)	Peak Flow (mgd)
6	18	3,274,381	81:30	40.66	4.161				
5	24	5,889,393	153:30	101.27	5.306				
8	16.5					758,381	64:45	8.66	1.214
7	24					2,986,042	151:00	97.80	3.197
4	12					856,272	79:15	5.18	1.159
3	24					3,001,474	103:45	79.49	3.470
2	42	12,861,816	163:15	138.72	12.319				
1	18					202,484	31:30	116.74	1.332

One half or more of the I/I volume appears to be entering the system upstream of Site 5. The presence of silt, rocks, and sand at this meter location may also indicate a possible defect or defects upstream of this location. Approximately half of the I/I upstream of Site 5 appears to be entering the system upstream of Site 6 and half downstream of Site 6. The remaining I/I entering the system appears to be nearly equally divided between Sites 3 and 7. Approximately two thirds of the I/I volume entering upstream of Sites 3 and 7, is entering downstream of Sites 4 and 8.

The combined estimated I/I volume from Sites 5, 7, and 3 exceeds the estimated I/I volume at Site 2 for the April 3, 2017 event. The duration of this rainfall was less than six hours, but the response lasted nearly two days. Flow dampening may have contributed to this volume discrepancy.

The peak depths at Site 2 were nearly identical for the three rainfall events in Table 4, around 138 inches. According to Hendersonville's manhole attribute data, Manhole 196 is about 153 inches deep. Level readings under surcharge conditions and greater than ten feet are not as accurate. Likely, the level at Site 2 was restricted by the bolt-down manhole cover. According to field personnel, this manhole was located in an area that was subject to flooding. Likewise, Site 1 had a bolt-down manhole cover. The peak depths for the late March and early April rainfall events were nearly identical, around 122 inches. Manhole 3836 is 143 inches deep according to Hendersonville's GIS. The level at Site 1 may have been restricted by the bolt-down cover or the similar peak depths may have been coincidental.

Six of the eight metered locations surcharged for all three of the events detailed in Table 4. Sites 8 and 4 did not surcharge.

Summary

At Site 5, the average daily dry-weather depth of flow utilized 62% of the pipe diameter and at Site 8 the average daily dry-weather depth of flow utilized 56% of the pipe diameter. There is limited capacity available for future dry-weather flows at these sites due to the average depth of flow utilizing over one-half of the pipe diameter. The average daily dry-weather depth of flow at the remaining sites utilized approximately one-quarter of the pipe diameter or less and significant capacity is available.

During rain events, significant I/I appears to be entering the system. The areas upstream of Site 5 should be targeted to isolate and eliminate the most significant sources of I/I. Recommended activities include manhole inspections, smoke testing, and targeted television inspections. Rehabilitation based on the results of these inspections is likely needed.