2018 FINAL

TOWN OF WAXHAW WASTEWATER SYSTEM PLANNING

Master Plan Addendum

B&V PROJECT NO. 195982

PREPARED FOR

Union County

7 MARCH 2018

BLACK & VEATCH

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1.0 Objectives and Background

The Town of Waxhaw is located in the 12-Mile Creek WWTP service area. The majority of the customers in the Waxhaw area along the Providence Road (Highway 16) corridor are served by an 8-inch trunk sewer. The downtown area in Waxhaw is served by the 21-25 Pump Station (PS21-25) that delivers flow to the upstream end of the 8-inch Waxhaw trunk sewer (Waxhaw Sewer). The trunk sewer increases in diameter to 10-inches near Citation Oaks Ct approximately 1 mile downstream of the PS21-25 force main discharge. The Waxhaw sewer is located between the Millbridge area and the Blythe Creek basin. All three areas drain north toward Twelve Mile Creek. PS21-25 is located in the Rone Creek basin, which flows west towards the South Carolina border. The Waxhaw Area is shown in Figure 1-1.

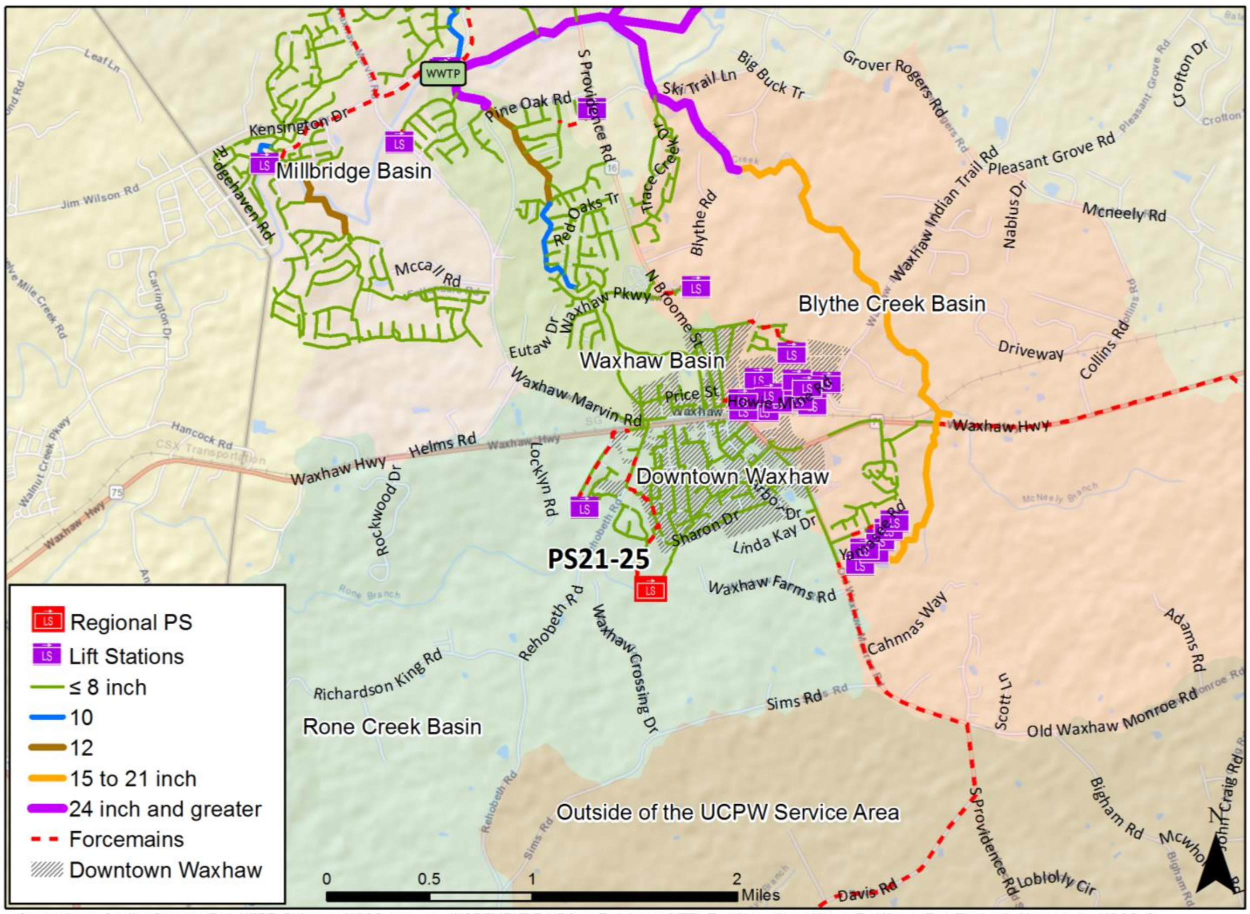

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Figure 1-1 Waxhaw Area Map

Based on the modeling completed during the 2011 Comprehensive Master Plan, the 8-inch Waxhaw Sewer is at capacity (surcharged, but not overflowing) during a large storm event. A detailed study was recommended to analyze the sewers in the Waxhaw area. The main objectives of this study are to:

- Calibrate the Waxhaw flows
- Develop flow projections for the Town of Waxhaw
- Expand the model to include the Millbridge sewers
- Evaluate potential alternatives to relieve capacity constraints
- Recommend improvements for the Waxhaw area

2.0 Model Update and Calibration

2.1 MODEL UPDATE

The Waxhaw Sewer is located in the western portion of Union County. Based on Union County's GIS, the Waxhaw Sewer comprises approximately 27.4 total miles of piping ranging from 4-inch through 24-inch piping with 24 pump stations. Most of the pump stations in the Waxhaw area are small grinder pumps located on individual streets. Many of these stations are being eliminated through gravity conversions as part of UCPW's CIP. As part of the Waxhaw Sewer update, the collection system upstream of and including the Millbridge Lift Station were added to the model. The Millbridge area of the collection system comprises approximately 15.2 total miles of piping ranging from 6-inch through 12-inch piping with 1 pump station.

Union County provided the GIS database of their entire collection system. This was the primary data source used to develop and update the collection system hydraulic model (model), which consisted of the following information:

- Manhole Locations
- Manhole ID
- Manhole Rim Elevation
- Sewer ID
- Sewer Diameter
- Sewer Upstream & Downstream Invert Elevation

The manhole invert elevation was assumed to be the same as the connecting sewer inverts. The model was limited to sewers 10 inches in diameter and larger and any 8 inch sewers considered significant or needed to maintain system connectivity.

Two sewer extension projects were included in the model update. As-built drawings for two sewer extensions in the Blythe Creek basin were used to update the model. The Blythe Creek sewer extension (Wysacky Sewer) was added as well as the outfall near Waxhaw-Indian Trail Rd that eliminated several grinder pump stations east of Providence Rd.

Figure 2-1 shows the GIS database for the entire western portion of the Union County collection system with the sewers/force mains color coded by diameter. New Projects added to the model are called out in the figure.

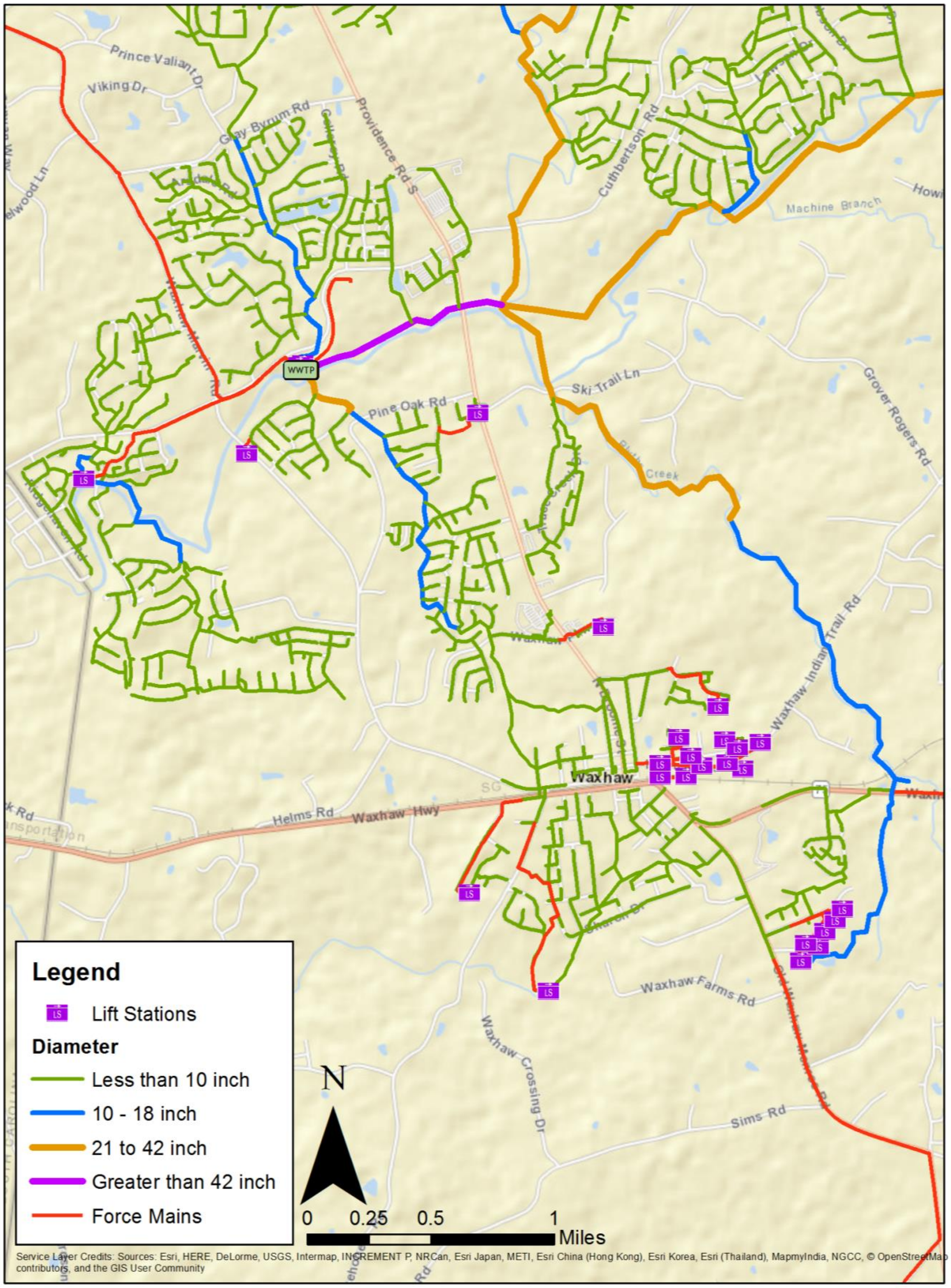


Figure 2-1 Union County GIS Database Western Portion

Record drawings were provided for two pump stations that were included in the model update – Millbridge Pump Station and PS21-25. The wet well dimensions, invert elevations, and pump curves were entered into the model for both. The force main alignment and diameter information was imported into the model from the GIS database and the record drawings were used to verify that key high points along the route were added to the model.

The data imported from the GIS database into the model was reviewed for missing attribute information, adversely sloped sewers, and connectivity.

The Millbridge portion of the collection system needed the flows added to the model so subcatchments were created. The collection system contributing area was determined by creating a 200 foot buffer around the gravity sewers in the collection system. Thiessen polygons were drawn around each manhole. One subcatchment was created for each manhole in the GIS by intersecting the 200 foot contributing area buffer with the manhole Thiessen polygons layer. In the model, upstream subcatchments were assigned to the nearest downstream modeled manhole.

2.2 MODEL CALIBRATION

The model matches the observed peak depths, peak flows, and volumes with reasonable accuracy and generally within the target ranges for those flow meters. In general, the calibration scatter plots demonstrate the model's accuracy in meeting the calibration goals. The approach and results to the model update and calibration can be found in Appendix A.

3.0 Wastewater Flow Projections

UCPW and Black & Veatch met with the Town of Waxhaw during the development of the 2011 Master Plan. Since that time, the Town of Waxhaw has adopted a new Comprehensive Town Plan. During this study, two meetings were held with Town of Waxhaw staff, including the Town Manager, Planning department staff and engineering staff. The goals of the meetings were to understand the objectives of the comprehensive plan, get feedback on short term and long term growth in the area, and discuss impacts of planned transportation infrastructure on the rate of population growth.

3.1 POPULATION PROJECTIONS

3.1.1 Town of Waxhaw Comprehensive Plan

The Town of Waxhaw adopted a new Comprehensive Town Plan in October 2016. The Plan addressed the future growth potential in the Waxhaw area. The Plan identified land area in and surrounding the town that would be available for development in the future. The available land was classified into four "growth sectors". Figure 3-1 shows the study area and the growth sectors from the Comprehensive Plan. The growth sectors were defined as:

- Restricted Growth – defined by rural areas without access to public sewer and limited transportation connectivity
- Controlled Growth – Currently lack transportation and sewer infrastructure, but are located in areas where infrastructure is expected to be extended
- Intended Growth – Areas with public sewer and access to transportation routes
- Infill Growth – Downtown Development

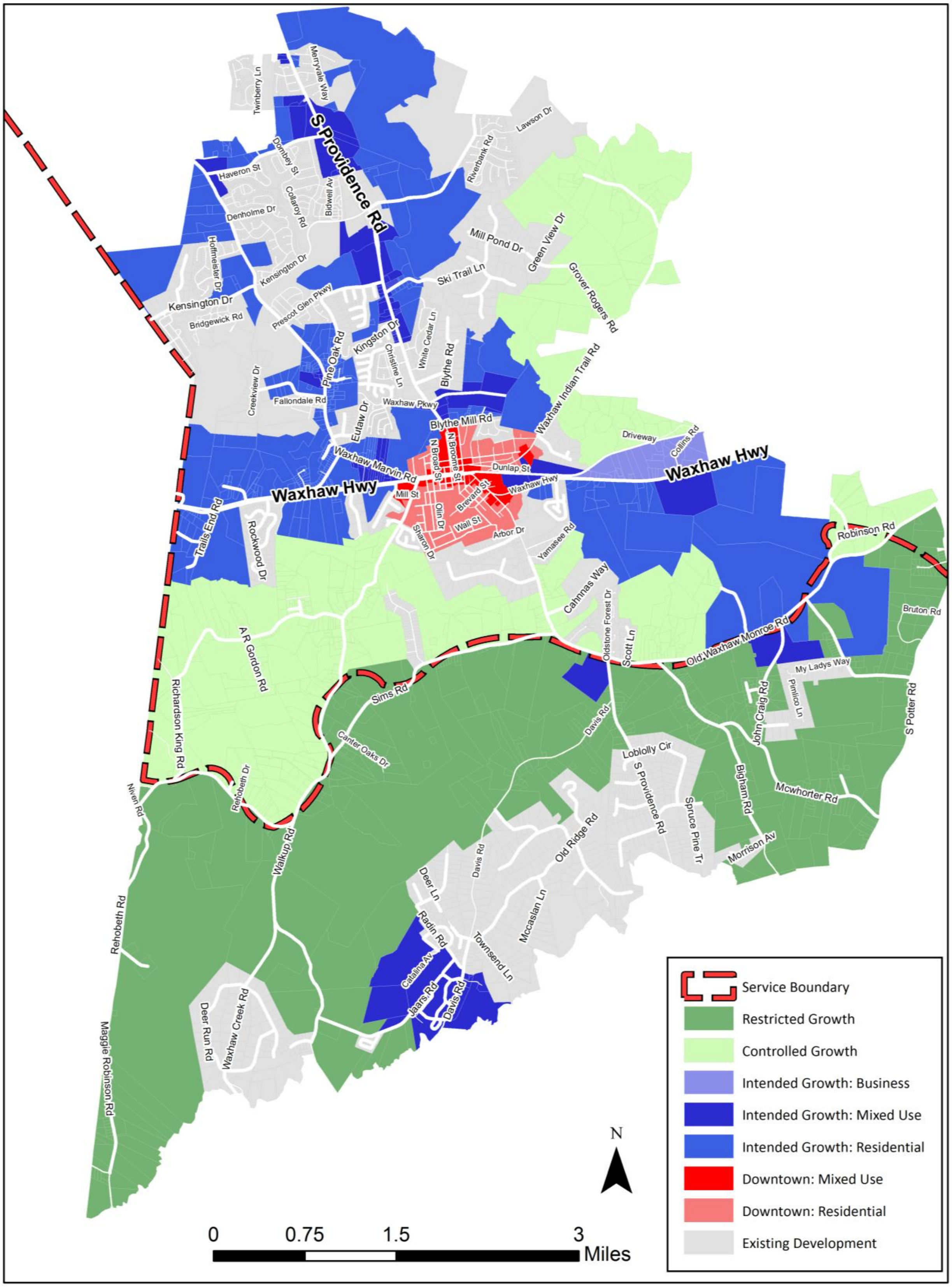


Figure 3-1 Town of Waxhaw Growth Sectors (Source: Waxhaw Comprehensive Plan)

In areas with adequate infrastructure, the town will allow up to 8 units per acre. However, based on recent inquiries, most single family residential developments are in the 4 lots/acre range with some apartments and townhomes near downtown. The apartments and townhomes have more units/acre than single family developments, but generally have less people in each unit.

The town's comprehensive plan introduced three growth concepts for the 30-year planning horizon (2050) based on the percent of the currently undeveloped land that would be developed by that time:

- Low Growth – 30% of available land developed by 2050
- Medium Growth – 50% of available land developed by 2050
- High Growth – 70% of available land developed by 2050

While the Waxhaw planning department indicated that they believe development will occur faster than the low growth scenario, the Waxhaw Parkway and other roadway improvements would be required for the medium or high growth developments to occur.

3.1.2 Transportation

Roadway improvements include the major Hwy 16 widening project (2024), which includes increasing the section from Rea Rd to Waxhaw Parkway to 5 lanes. The timing of the Highway 16 widening was confirmed with County DOT. The Waxhaw Parkway, which is planned to facilitate traffic into and around Waxhaw, is not currently funded. The development of the large parcels, including the Pittenger property, Southeast of Downtown Waxhaw, is contingent on the completion of the Northeastern bypass. The Town anticipated that funding for the Waxhaw Parkway would most likely not occur before 2030, but funding is anticipated before 2050. The Southern Waxhaw Bypass is planned for beyond 2050, but would facilitate East/West traffic relief on Hwy 75 through the middle of town.

3.1.3 Service Area Boundary

The UCPW wastewater service area boundary south of Waxhaw was defined during the 2011 Comprehensive Plan as the southern boundary of the Rone Creek basin. The service area includes the town of Waxhaw, Millbridge, Blythe Creek basin and Rone Creek basin as shown in Figure 1-1. The ridgeline roughly following Sims Rd and Old Waxhaw Monroe Rd acts as the southern limit of the service area. Flows from land south of the UCPW Service area boundary has not been included in previous wastewater studies. The basin south of Rone Creek is called the Waxhaw Creek sub-basin, even though the creek does not run through the town of Waxhaw. There is no existing infrastructure in the Waxhaw Creek sub-basin. Also, there are environmental restrictions related to mussels that would make sewering the entire basin cost prohibitive. There has been some interest in pumping from the upper reaches of the sub-basin near the ridgeline to the UCPW sewer system.

3.1.4 Septic Systems

Currently, there are no known issues with septic tanks in the Waxhaw area. No septic conversions will be included in the near-term planning numbers. Some conversion is expected by 2050, focusing on smaller parcels near town.

3.1.5 Known Developments

Known development plans from the town and the county were reviewed within the Waxhaw study area within the service area boundary. Known developments were assumed to be the first parcels to develop. Outside of known developments, the areas that will be assumed to develop first will depend on infrastructure improvements

3.2 FUTURE POPULATION PROJECTIONS

Future population in the Waxhaw area was estimated using the feedback from UCPW and the Town of Waxhaw. Projections were completed for each of the timing scenarios: Low, Medium and High growth. The projections were developed using the following assumptions based on the Town of Waxhaw growth sectors:

- Known developments will be included based on estimated flows/plans submitted
- 30% of all other available land is reserved for roads, infrastructure and open space

Within the Controlled Growth and Intended Growth Areas, new developments will be assumed to have of four single family homes per acre. Assume 3.17 persons per single family dwelling based on the Census per household estimate for Waxhaw, NC.

In the downtown area, new developments will be assumed to have eight apartment/townhomes per acre. Apartments were assumed to have 2 persons per dwelling.

In the restricted growth area (South of the Rone Creek ridgeline), only 10% of the total area will be considered for potential development. Assuming sewer is extended to these developments a density of four single family homes per acre will be used. Areas not served by Union County wastewater infrastructure would be limited to a maximum development density of one home per 2.3 acres.

Figure 3-2 shows the population projections for the Low, Medium, and High growth scenarios based on the above assumptions. The projections from the 2011 Master Plan are also shown. In 2011, population growth was more concentrated in the Weddington and Marvin areas. Since that time growth in and around Waxhaw has increased. The Low, Medium, and High growth projections were each compared to a growth rate. The projections ranged from 2.3% to 4.6% growth.

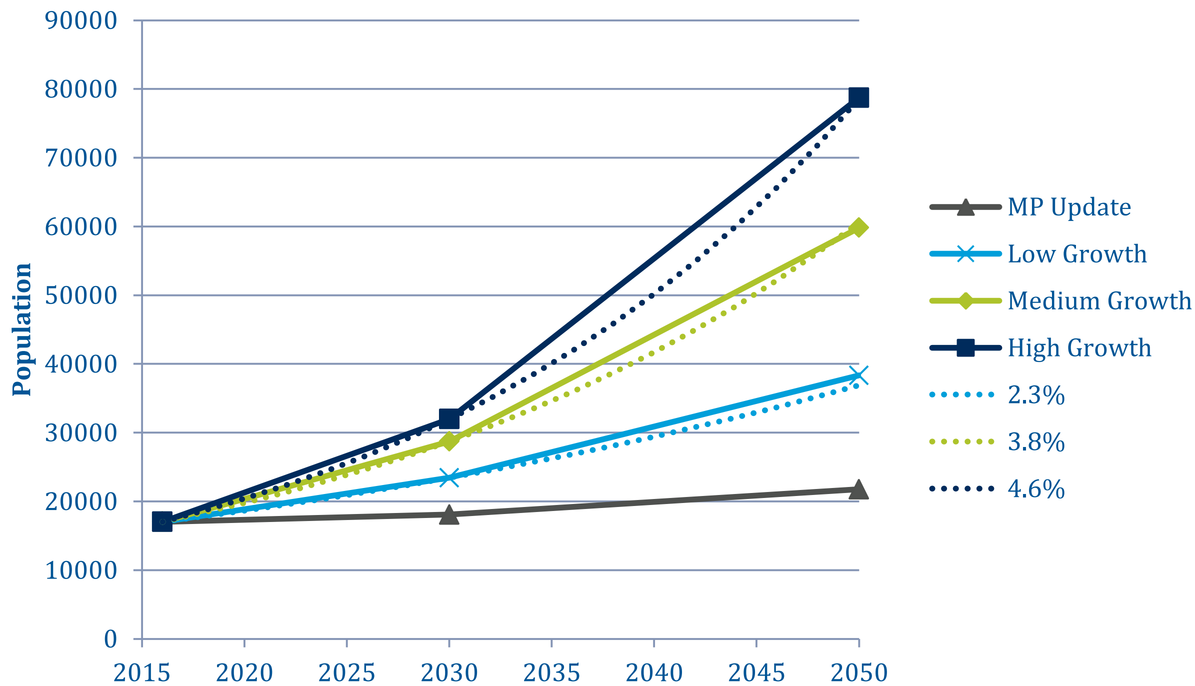


Figure 3-2 Projected Populations for Waxhaw's Low, Medium, and High Growth scenarios

In order for Waxhaw to grow in population beyond the low growth scenario, improvements need to be made to the transportation infrastructure. The Highway 16 widening project is expected to begin in 2024. A project of that scale is anticipated to take several years to complete. Growth is expected to follow the low growth trend until around 2030 due to the lack of transportation infrastructure. After that point, the growth is anticipated to follow the medium growth trend. Figure 3-3 shows the "Union County Modified" population growth trend, which follows the low growth line until 2030. Beyond 2030, the growth rate was increased to 3.8% to be in line with the medium growth scenario. The Union County modified projections will be used for the subsequent improvement planning. The medium and high growth scenarios were also modeled as part of a sensitivity analysis of the improvement sizing.

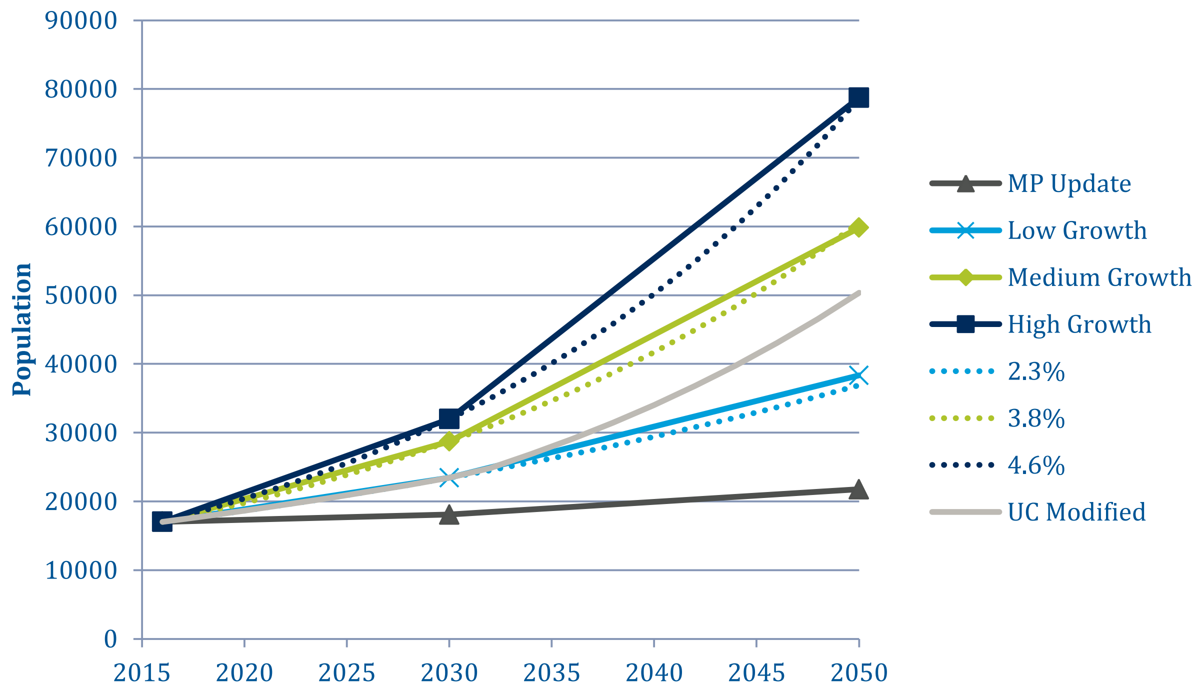


Figure 3-3 Union County Modified Population Projections

4.0 Existing System Capacity Analysis

The calibrated model was used for the existing system capacity analysis. The model was setup using a 1 year 24 hour SCS Type III storm event similar to the 2011 Master Plan and 2016 Master Plan Update.

4.1 WASTEWATER FLOW PROJECTIONS

Future Flows were determined by adding residential and commercial growth on top of the existing flows. The current "base line" flows were established in the calibrated model based on the 2016-2017 flow metering.

The projected population was converted to flow using wastewater flow unit rates. A per capita usage rate of 80 gpcd was applied to all residential population figures. 80 gpcd was established during the Comprehensive Master Plan based on North Carolina standards and was in line with the estimated per capita usage for Union County's existing customer basis. In addition, commercial development will be assumed to contribute 525 gal/acre. The 525 gal/acre rate comes from the City of Monroe's planning numbers for general commercial development and is consistent with previous Union County planning studies. Figure 4-1 shows the total flows estimated for each of the growth scenarios. Figure 4-2 shows the area from the Waxhaw Comprehensive Plan and the parcels assumed to develop by 2030 and 2050 for the UC Modified projection as well as the 2050 Medium and High growth projections shown in Figure 4-1.

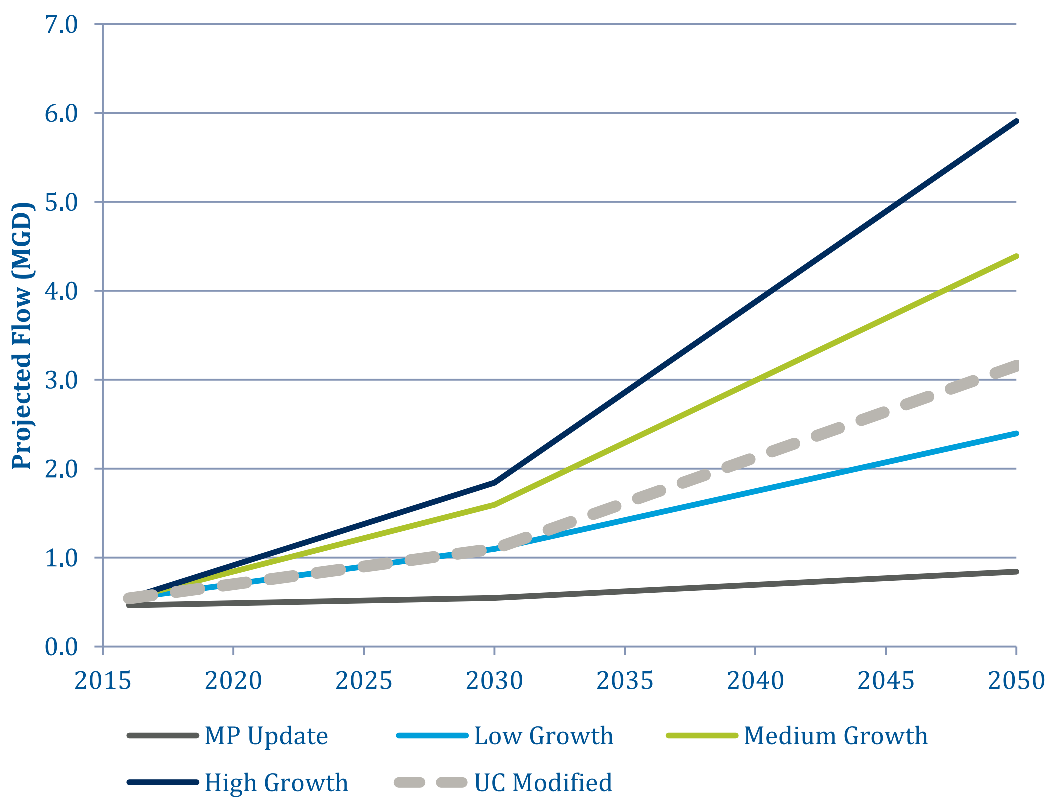


Figure 4-1 Projected Wastewater Flows from the Waxhaw Area

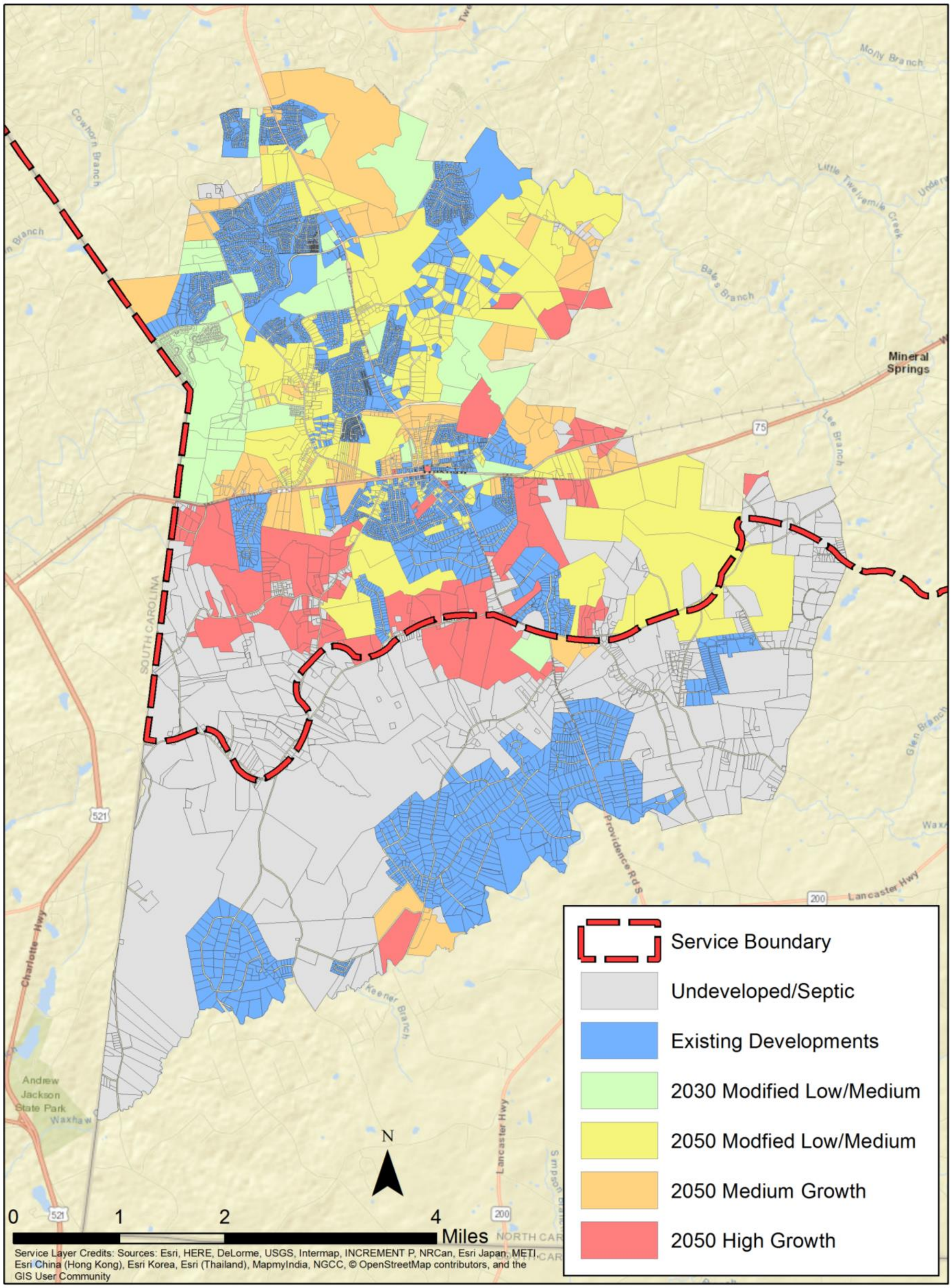


Figure 4-2 Development Year by Parcel - Waxhaw Area

The existing flow from the Waxhaw area is 0.54 MGD. The 2050 projected flows ranged from 2.3 MGD to 5.8 MGD. The Union County modified projection for 2050 is 3.1 MGD. The projected flows were added to the calibrated model for the capacity analysis. Figure 4-2, above, shows the allocation of the new developments projected for the modified growth, medium growth and high growth scenarios.

4.2 MASTER PLAN PERFORMANCE CRITERIA

The following performance parameters and target service levels were used for the wastewater collection system. The parameters were developed as part of the 2011 Master Plan. A new project is recommended whenever the existing infrastructure fails to meet the minimum acceptable performance criteria. A new project is designed so that the new infrastructure can perform at the desired service goal through the established planning horizon.

Table 4-1 Collection System Performance Criteria

FACILITY TYPE	SERVICE GOALS/ DESIGN CRITERIA	MINIMUM ACCEPTABLE PERFORMANCE
Pipe/Manhole	Hydraulic grade line below the pipe crown during the design storm event. $d/D \leq 1$	The modeled hydraulic grade line cannot exceed the manhole rim, i.e. No SSOs
Pump Station	Peak flow less than pump station firm capacity	The modeled peak flow (Q) ≤ the pump station firm capacity (Qf) The firm capacity is the capacity with the largest pump out of service.
Force Main	Peak flow velocity 3 to 4 FPS	The modeled peak velocity exceeds 10 FPS

4.3 EXISTING SYSTEM CAPACITY CONCERNS

The existing system capacity model results indicate capacity concerns in the Waxhaw Sewer and in the downtown Waxhaw area. The existing infrastructure in the Millbridge basins is adequately sized to serve its gravity basins. The existing Blythe Creek sewer was also sized to accommodate future flows.

The capacity concern in the Waxhaw Sewer is from manholes M1845 to M14198. These pipe segments are located in the low area East of Harrison Park Drive. The peak hydraulic grade line does not exceed the rim elevations but the peak depth surcharges a pipe section approximately 5 feet to within 2.5 feet of the manhole rim.

The capacity concerns in the downtown Waxhaw area include the following:

- PS21-25 has insufficient capacity to keep up with the projected peak flows that are attributed to the downtown Waxhaw area. The firm capacity of the existing station is only 260 gpm based on a 2016 drawdown test. By 2030, the projected peak storm flow at the station is expected to exceed 3 MGD.
- The gravity sewer from manhole M2598 to PS21-25 is surcharged to the rim with several model-estimated sanitary sewer overflows (SSOs). These pipe segments are located starting northeast of the intersection of Jerry Lane and Howie Street and flowing to the southwest under Howie Street and under the intersection of Anne Avenue and Sharon Dr to PS21-25.

The existing system capacity concerns are highlighted in Figure 4-3.

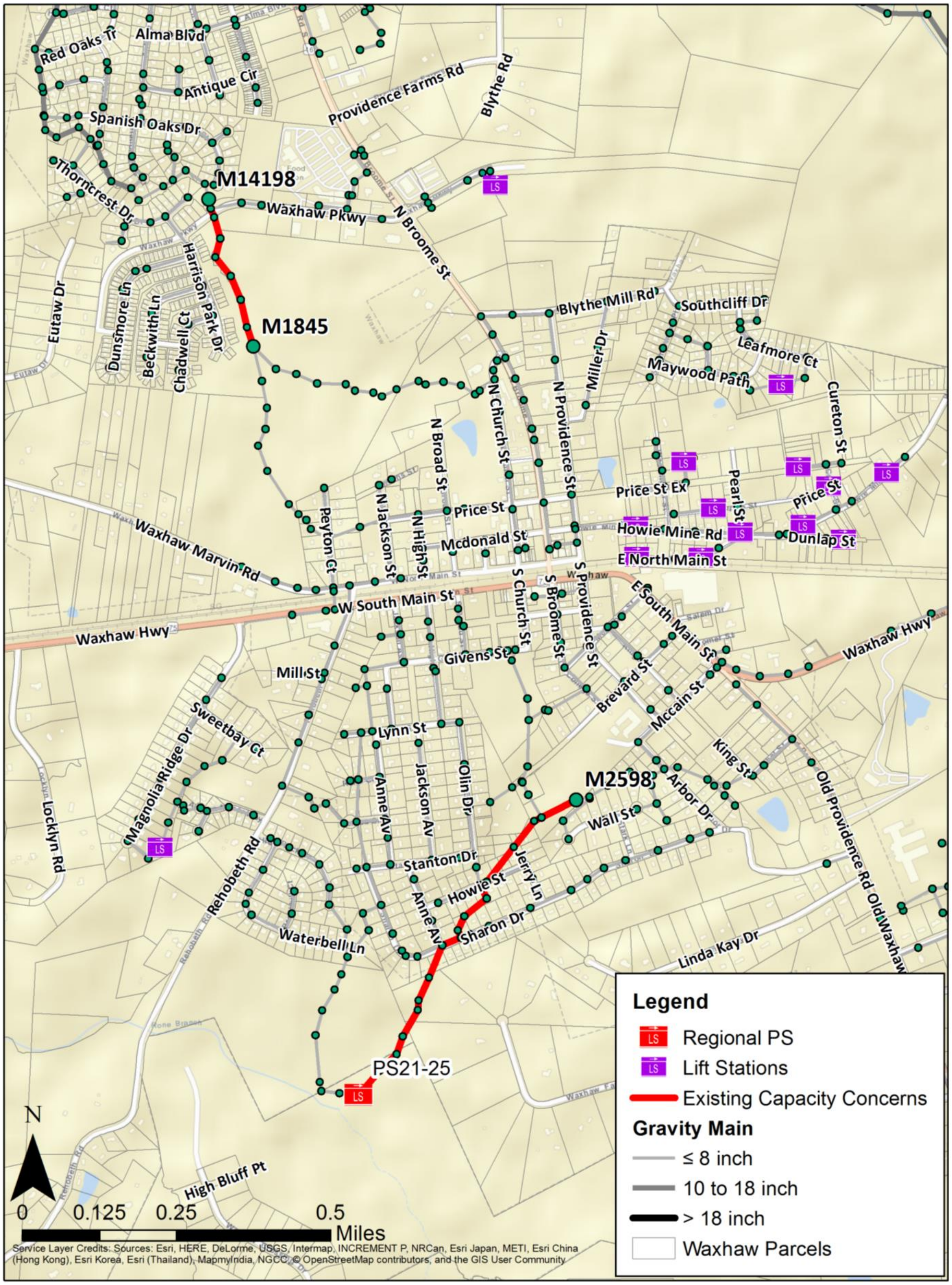


Figure 4-3 Existing System Capacity Concerns

5.0 System Improvement Alternative Analyses

The existing model was updated by adding the flow projections for identified growth areas and assigning them to the existing model. The growth areas represent the future scenarios for serving the Waxhaw downtown area and adjoining areas. The future scenarios include:

- 2030 Development.
- 2050 Modified Low/Medium Development.
- 2050 Medium Development.
- 2050 High Development.

Each growth area was reviewed to determine which existing wastewater sanitary manhole would receive the flow based on being adjacent to the existing system or how the area would most likely be served once sewers were designed and constructed into the area. With the different alternatives analyzed, some growth area loading points changed locations due to the different pipe layouts in the alternatives. The growth areas that were located in the Rone Creek Basin downstream of PS21-25 were considered to be served only in the long term Alternative 2 analysis.

Each scenario was analyzed for the four alternatives for a total of 16 different modeling scenarios and alternatives. The alternatives are highlighted in Figure 5-1 with each alternative described in more detail in the following sections.

Preliminary projects and opinions of probable cost were developed for the projects in each modeling scenario to help compare options. Project costs were based on the unit costs and assumptions from the 2016 Wastewater Treatment Planning Update. Project costs include markups for contingency (20%), engineering (15%-20% dependent on project type), and easements (30 feet wide, 50% of a land value of $10,000 per acre). Unit costs for gravity sewer are shown in Table 5-1. Preliminary cost opinions were done to facilitate the alternative comparison and should not be used as final project cost estimates. A more thorough project refinement and cost estimate will be completed for the final selected alternative.

Table 5-1 Wastewater Pipe Unit Costs

DIAMETER (IN)	UNIT COST ($/LF)
8	111
10	132
12	153
15	187
18	222
21	260
24	300
27	342
30	386
36	480
42	583
48	693
54	812

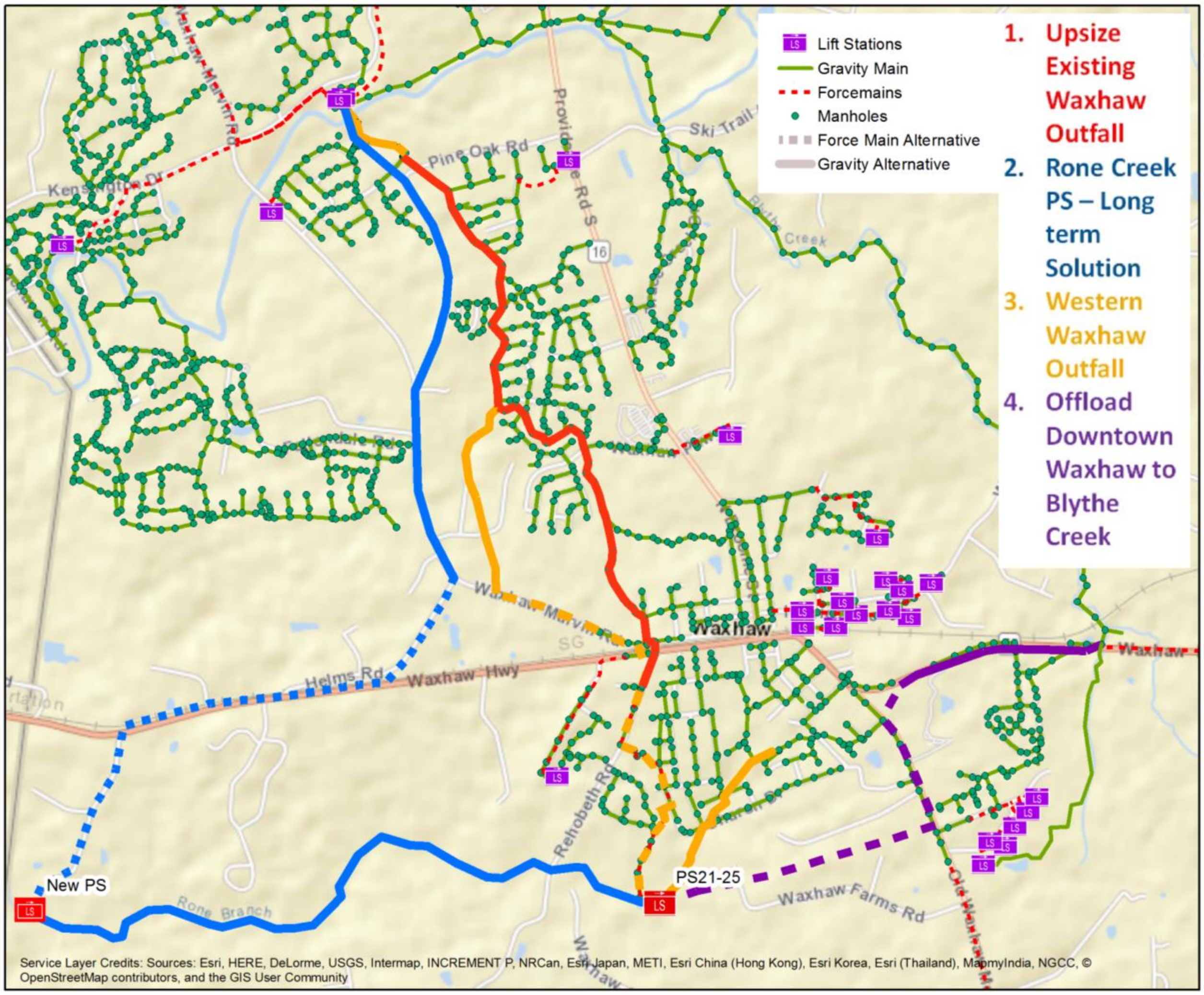


Figure 5-1 Alternative Pipeline Routes

5.1 ALTERNATIVE 1 – UPSIZE THE WAXHAW SEWER

The most simplistic approach to serving the increased future flows for the Town of Waxhaw and the adjoining areas is to expand sewer infrastructure as needed. This alternative would include routing all the flows through PS21-25 and the existing Waxhaw Sewer.

Pros for the Upsize Waxhaw Sewer alternative include:

- Alleviates immediate capacity concerns.

Cons for the Upsize Waxhaw Sewer alternative include:

- Sewer is very close to existing homes.
- Construction would be difficult in many sections.
- Pipe size increases could be too large for pipe bursting.

The capacity analysis shows that the existing capacity concerns are exacerbated with the increased flows. The pipe upstream of PS21-25 would need to be increased as well as PS21-25, its force main and the entire length of the Waxhaw Sewer. The new firm capacity of PS21-25 would likely require construction of a new station. A summary of the required improvements is presented in Table 5-2 for the modified, medium and high growth population projections. Improvements for the modified projection are shown in Figure 5-2.

Table 5-2 Alternative 1 Required Improvements

GRAVITY	MODIFIED LOW/MEDIUM	MEDIUM	HIGH
15”	6,742’	4,310’	0
18”	7,205’	9,637’	6,742’
21”	1,232’	1,232’	8,437’
Total	15,179’	15,179’	15,179’
PS21-25 New Firm Capacity	3.5 MGD	4.1 MGD	4.7 MGD
Force Main	18”, 4,800’	18”, 4,800’	21”, 4,800’
Cost[1]	$6.6M	$7.0M	$7.9M
[1]Based on Unit costs, does not include any additional costs for difficult construction			

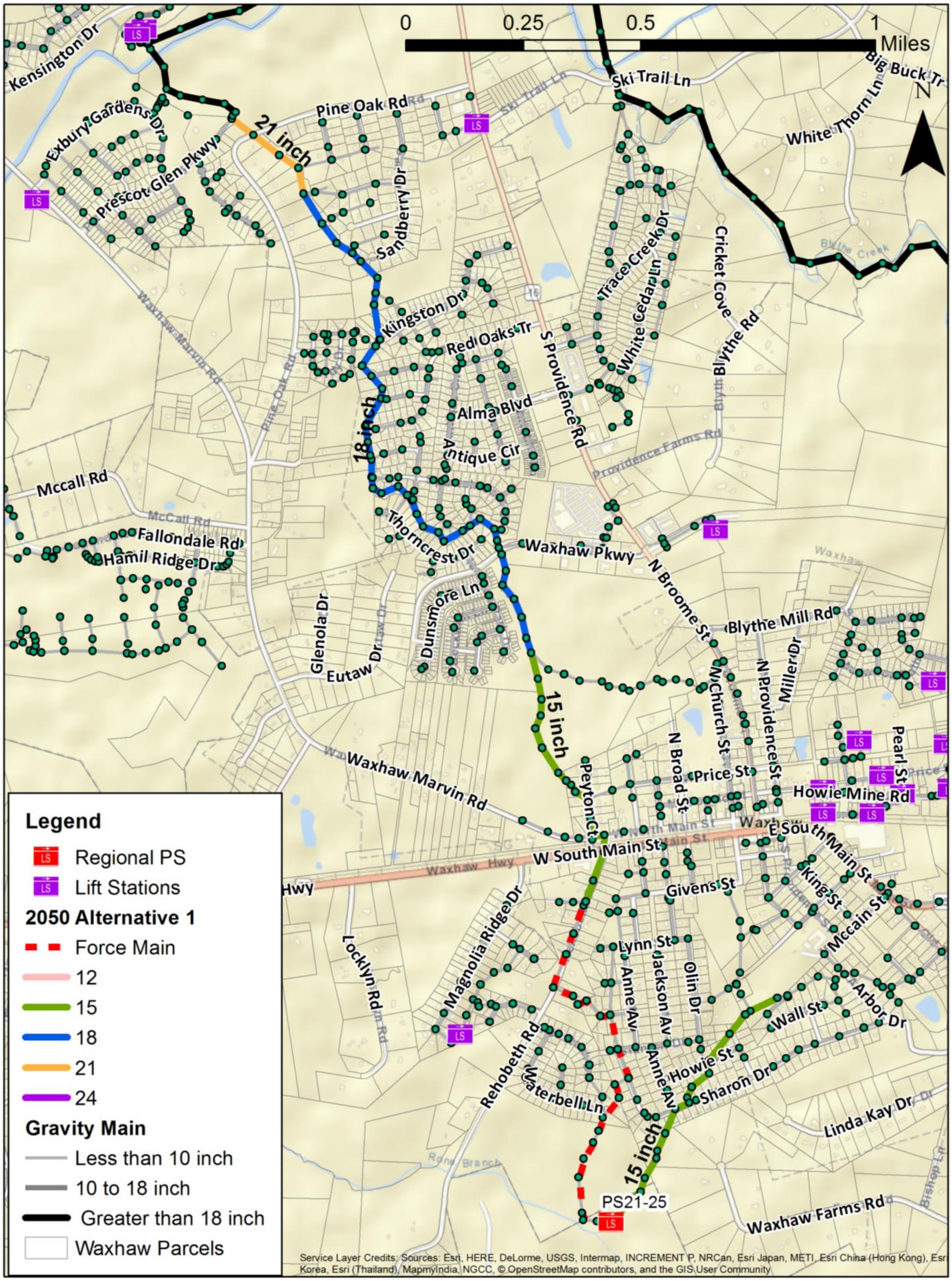


Figure 5-2 Alternative 1 Modified Low/Medium Required Improvements

5.2 ALTERNATIVE 2 – RONE CREEK PUMP STATION

Alternative 2 consists of retiring PS21-25 and allowing gravity flow along Rone Branch Creek to the western edge of Union County where the new Rone PS would be constructed. The force main would follow north along Trails End Road and then east along Waxhaw Highway and northeasterly along Helms Road to Waxhaw-Marvin Road where it would discharge into a gravity sewer. The gravity sewer would follow Waxhaw-Marvin Road to the north and then north along Pine Oak Road where it would discharge into the 12-Mile interceptor.

Pros for the Rone Pump Station alternative include:

- Serves all of Waxhaw area.
- Sized for future development.
- Offloads the existing Waxhaw Sewer.
- Developer driven.

Cons for the Rone Pump Station alternative include:

- Large scope.
- Development funding not yet available.
- Not a good short term solution.

The capacity analysis shows that some of the existing capacity concerns are exacerbated with the increased flows. The pipe upstream of PS21-25 would need to be increased, PS21-25 would be abandoned, and the Waxhaw Sewer may not need to be improved. No improvements to the Waxhaw Sewer would be needed to accommodate the modified low/medium flow projections. Pipe bursting the Waxhaw Sewer (8 inch to 10 inch) would be required to accommodate the flow for the medium and high growth scenarios. A summary of the required improvements are presented in Table 5-3 for the modified, medium, and high growth population projections. Improvements for the modified projections are shown in Figure 5-3.

Table 5-3 Alternative 2 Required Improvements

GRAVITY	MODIFIED LOW/MEDIUM	MEDIUM	HIGH
10”	0	1,933’	1,933’
15”	10,500’	1,068’	0
18”	7,400’	9,432’	3,500’
21”	4,900	9,900’	16,900’
24”	0	2,400’	2,400’
Total	22,800’	24,733’	24,733’
New PS	3.7 MGD	4.4 MGD	5.5 MGD
Force Main	18”, 10,000’	18”, 10,000’	21”, 10,000’
Cost [1]	$9.2M	$10.4M	$15.4M

[1]Based on unit costs, does not include any additional costs for difficult construction

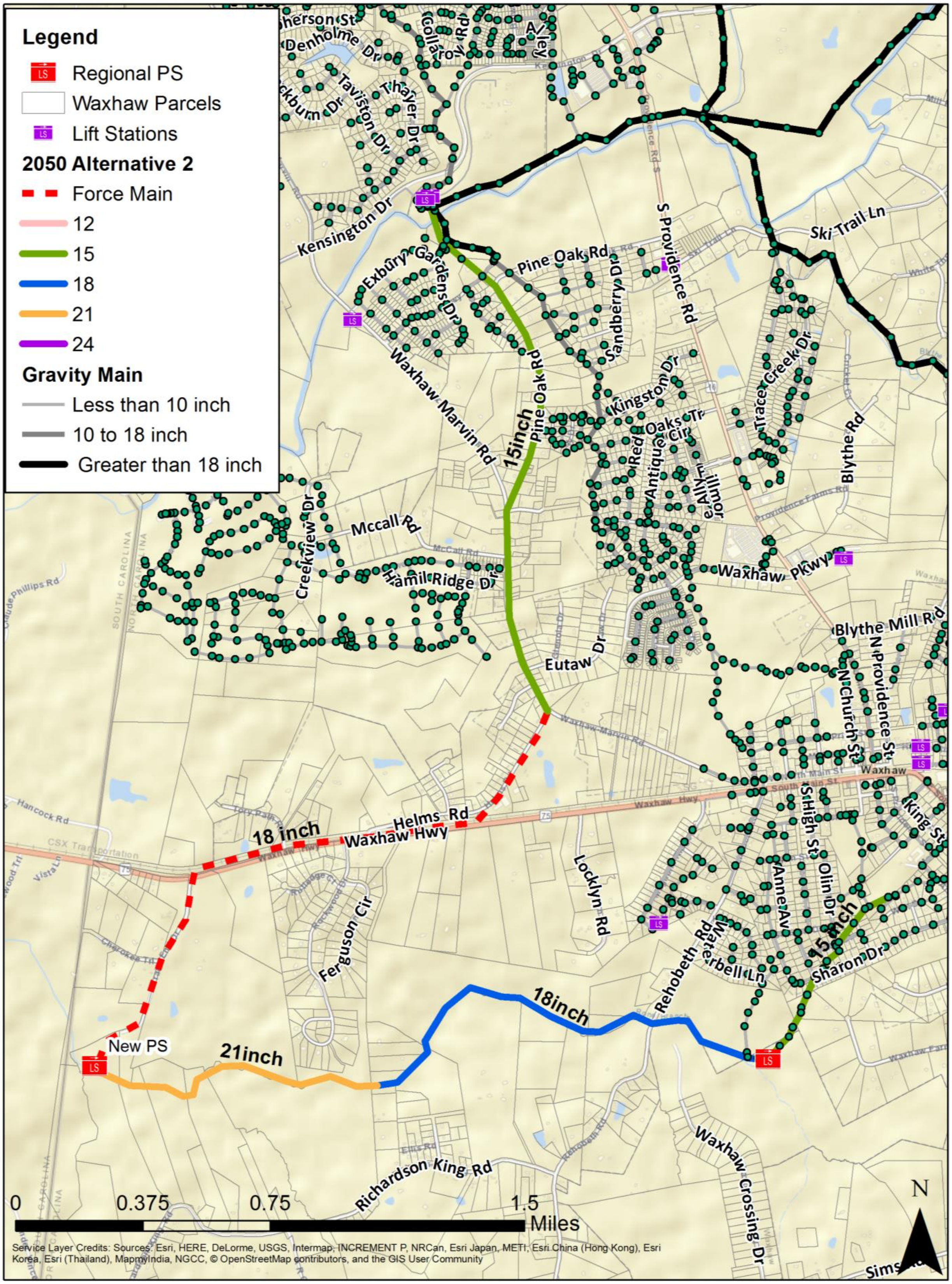


Figure 5-3 Alternative 2 Modified Low/Medium Required Improvements

5.3 ALTERNATIVE 3 – WESTERN WAXHAW SEWER

Alternative 3 is similar to Alternative 1 in that PS21-25 is increased as well as its force main but the force main is extended north across Waxhaw Highway and then northwesterly along Waxhaw-Marvin Road almost to Helms Road where it would discharge into a proposed gravity sewer that flows northerly until it connects with the Waxhaw Sewer at manhole M882, west of Winter Oaks Court.

Pros for the Western Waxhaw Sewer alternative include:

- Offloads downtown Waxhaw flows from the Upper Waxhaw Sewer.
- Avoids construction in some of the tightest easement sections.
- Gravity sewer is needed as a permanent solution for new development in the area.

Cons for the Western Waxhaw Sewer alternative include:

- Construction near some homes.
- Pipe sizes could be too large for pipe bursting on the northern portion of the Waxhaw Sewer.

The capacity analysis shows that the existing capacity concerns are exacerbated with the increased flows. The pipe upstream of PS21-25 would need to be increased as well as PS21-25, its force main and the northern portion of the Waxhaw Sewer. The new firm capacity of PS21-25 would likely require construction of a new station. A summary of the required improvements are presented in Table 5-4 for the modified, medium and high growth population projections. Improvements for the modified projections are shown in Figure 5-4.

Table 5-4 Alternative 3 Required Improvements

GRAVITY	MODIFIED LOW/MEDIUM	MEDIUM	HIGH
10”	0	1,933’	1,933’
12”	1,700’	0	0
15”	5,300’	3,868’	0
18”	3,858’	3,132’	7,000’
21”	1,232’	5,090’	5,090’
Total	12,090’	14,023’	14,023’
PS21-25 New Firm Capacity	3.0 MGD	3.5 MGD	4.1 MGD
Force Main	18”, 8,100’	18”, 8,100’	21”, 8,100’
Cost [1]	$6.1M	$7.4M	$8.0M
[1]Based on unit costs, does not include any additional costs for difficult construction			

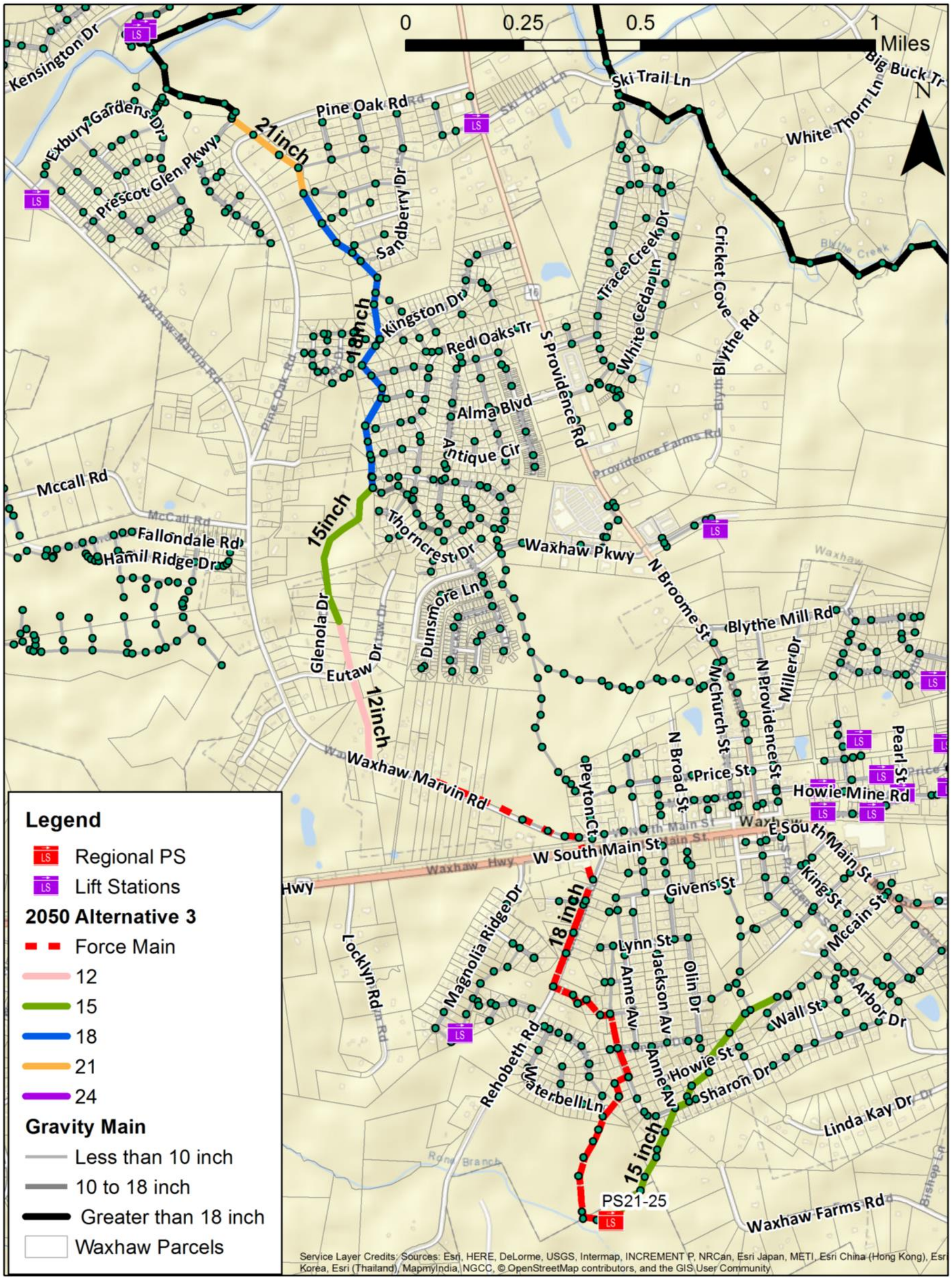


Figure 5-4 Alternative 3 Modified Low/Medium Required Improvements

5.4 ALTERNATIVE 4 – OFFLOAD TO BLYTHE CREEK SEWER

Alternative 4 sends all flows for the downtown Waxhaw area east to the Blythe Creek Sewer. The alternative includes a new force main from PS21-25 heading east to Old Waxhaw Monroe Road and then north to Waxhaw Highway and then east to the Blythe Creek Sewer.

Pros for the Offload to Blythe Creek alternative include:

- Alleviates immediate capacity concern.
- Blythe Creek Sewer has available capacity.
- Could be combined with upsizing the Blythe Creek Sewer extension project currently under design. The project extends gravity sewer to Old Waxhaw Monroe Rd along Blythe Creek. Moving the PS20-25 force main discharge to this new sewer extension would minimize the length of the new force main, but would require the designed sewer diameter to be increased.
- Lowest cost alternative.

Cons for the Offload to Blythe Creek alternative include:

- The location of PS21-25 cannot serve the entire Rone Creek basin. A future development project including the Rone Creek gravity sewer, pump station and force main would be needed to serve the entire basin. The Rone Creek pump station would be located near the South Carolina border, as shown in Alternative 2. The long term Rone Creek PS project is contingent on development southwest of Waxhaw, downstream of PS21-25. In this study, most of the downstream area of the Rone Creek basin was assumed to develop closer to 2050. Even though PS21-25 is not a long term solution for the entire basin, the station could easily be expected to be in service for 20-30 years before the Rone Creek PS would be constructed. At that point in time, PS21-25 could continue to send flows from Waxhaw to Blythe Creek or the station could be decommissioned and the sewer could flow by gravity to the Rone Creek PS.

The capacity analysis shows that some of the existing capacity concerns are exacerbated with the increased flows. The pipe upstream of PS21-25 would need to be increased along with PS21-25 but the Waxhaw Sewer may not need to be improved. No improvements to the Waxhaw Sewer would be needed to accommodate the modified low/medium flow projections. Pipe bursting the Waxhaw Sewer (8 inch to 10 inch) would be required to accommodate the flow for the medium and high growth scenarios. The new firm capacity of PS21-25 would likely require construction of a new station. A summary of the required improvements are presented in Table 5-5 for the modified, medium, and high growth population projections. Improvements for the modified projection are shown in Figure 5-5.

Table 5-5 Alternative 4 Required Improvements

GRAVITY	MODIFIED LOW/MEDIUM	MEDIUM	HIGH
10"	0	1,933'	1,933'
15"	3,500'	1,068'	0
18"	3,500'	5,932'	7,000'
Total	7,000'	8,933'	8,933'
PS21-25 New Firm Capacity	3.0 MGD	3.5 MGD	4.1 MGD
Force Main	18", 5,400'	18", 5,400'	21", 5,400'
Cost[1]	$4.0M	$4.96M	$5.9M
[1]Based on unit costs, does not include any additional costs for difficult construction			

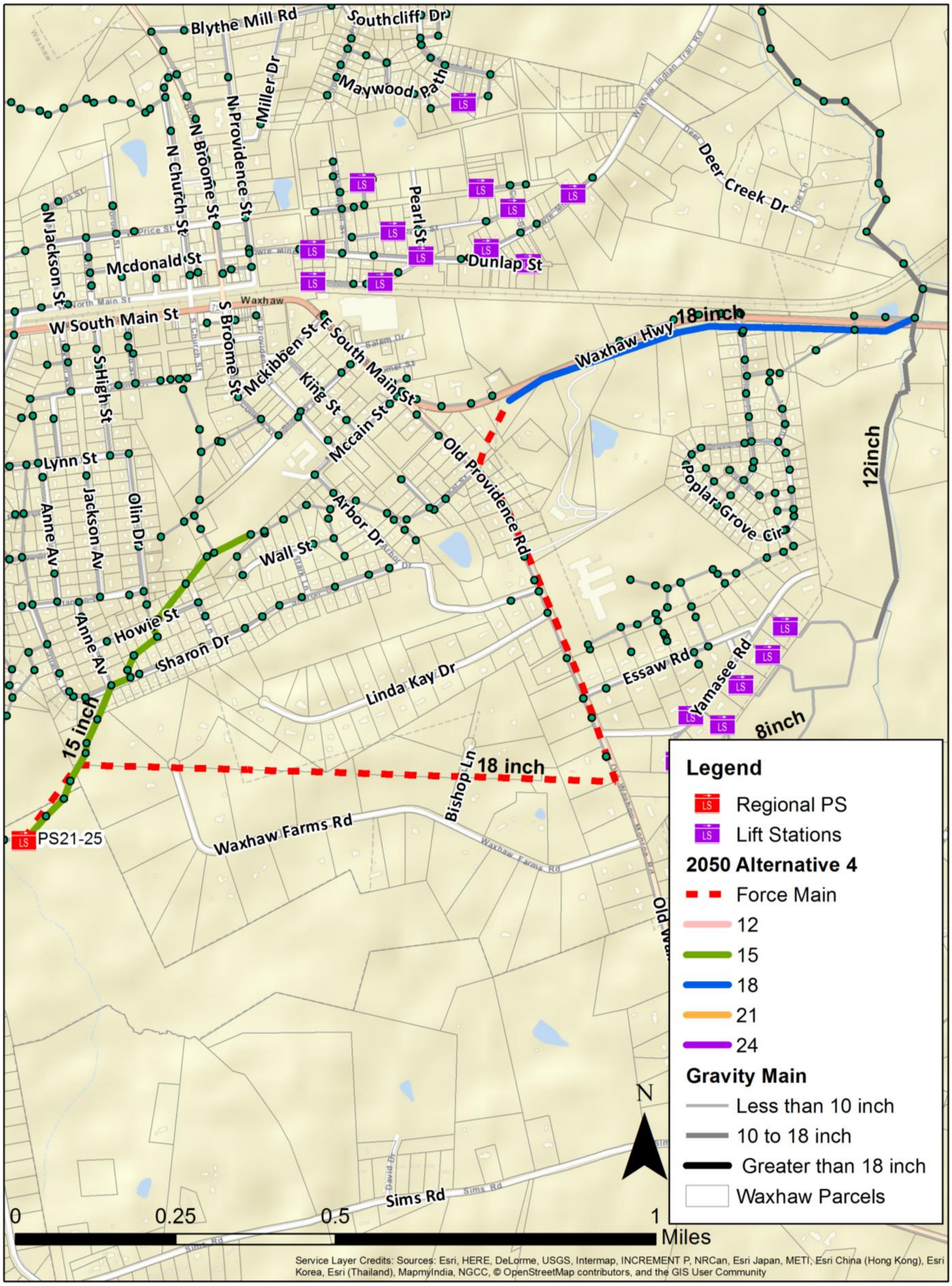


Figure 5-5 Alternative 4 Modified Low/Medium Required Improvements

6.0 Selected Alternative

Based on the alternatives analysis, the routing of the PS21-25 force main to the Blythe Creek Interceptor is the best solution to serve the Downtown Waxhaw area. Alternative 4 was modified to include an upsized Blythe Creek Interceptor Extension. Figure 6-1 shows the selected alternative.

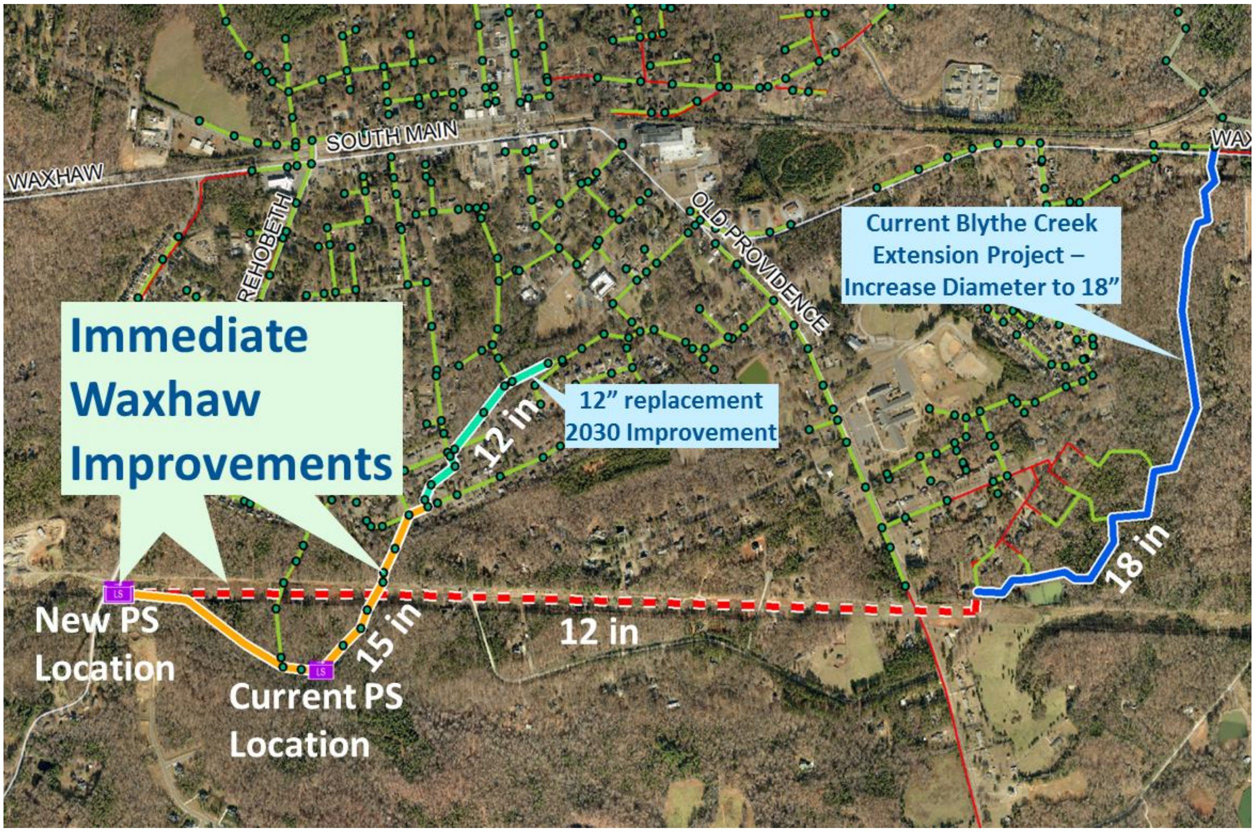


Figure 6-1 Proposed Downtown Waxhaw Improvement

The proposed improvement will:

- Provide adequate pipe and pumping capacity for the downtown area through 2050.
- Alleviate the capacity issues in the Waxhaw sewer collector downstream of the existing PS21-25 force main discharge point.
- Minimize easements and disruption to current residents by following the existing power easement.
- Leverage the current Blythe Creek Extension project to minimize the scope of the Waxhaw improvements.
- Minimize the cost to solve capacity issues in the Waxhaw sewer system.

The existing PS21-25 location is difficult to access. In addition, UCPW would likely need to acquire additional land for the expanded station. Locating the station 1,800 feet downstream along Rone Creek would allow for direct access from Rehobeth Road adjacent to the power easement.

A summary of the project scope and budgetary costs are listed in Table 6-1. Cost opinions were developed using the unit costs established in the 2016 Wastewater System Planning Update. The projects shown would need to be completed immediately to accommodate development in

Waxhaw's downtown core. The costs listed do not include the project costs for the Blythe Creek sewer extension.

Table 6-1 Opinion of Probable Cost – Immediate Downtown Waxhaw Improvements

PROJECT	COST
Downtown 15-inch Sewer Replacement (1,700 ft)	$450,000
PS 21-25 – New Replacement Station (3 MGD)[1]	$2,700,000
New 12-inch Force Main (5,700 ft)	$570,000
Longer Gravity to new Pump Station location (+1,800 ft) and Longer Force Main from new PS location (+1,300 ft)	$600,000
Total	$4,320,000

In addition to the short-term projects, the upstream section of the 8-inch Downtown Waxhaw collector sewer shown in Figure 6-1 would need to be upsized to a 12-inch diameter by 2030. The project would include replacing 1,700 feet of 8-inch sewer with a 12-inch sewer. The cost opinion for the 12-inch replacement was $370,000.

6.1 RONE CREEK DEVELOPER PROJECT

The lower Rone Creek Basin is not currently served by public sewer, but is part of UCPW's service area. The location of the lower Rone Creek Basin is show in Figure 6-2. By application of the UCPW's line extension policy, new developments in this area can be served by installing an interim solution to deliver wastewater to the existing sewer infrastructure and paying into the long term solution for the drainage basin. The long term solution for Rone Creek includes a pump station at the lowest point in the basin that will transfer flows to the Millbridge sewers.

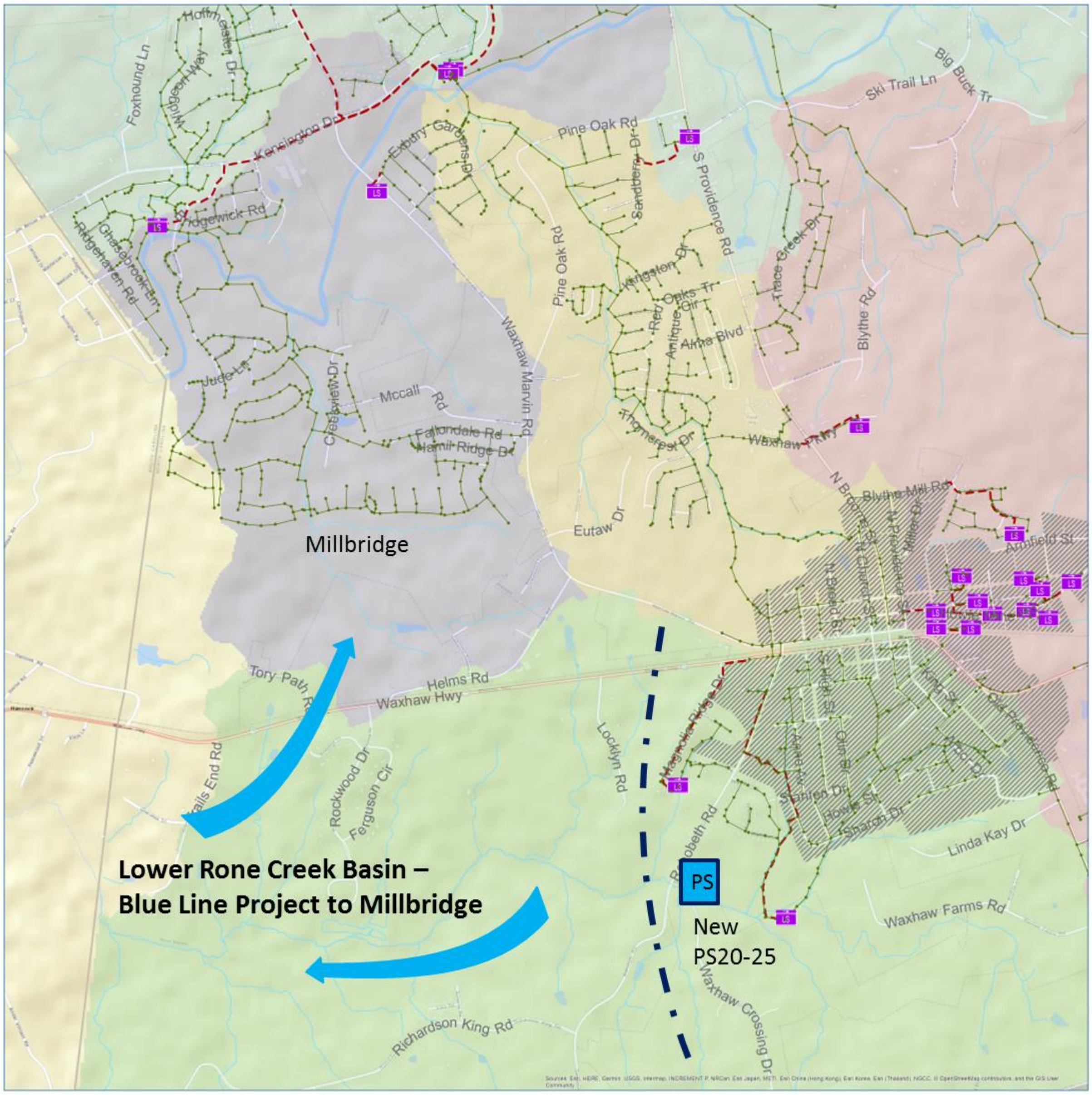


Figure 6-2 Lower Rone Creek Basin

The Millbridge sewer system was designed for buildout flows from the planned Millbridge development phases. The sewer collector extending south toward Rone Creek ranges in size from 8 to 12 inches. The 8-inch and 12-inch sewer collector is adequately sized for all parcels in Millbridge Basin (i.e. the area north of Waxhaw Highway). In addition, the sewer has approximately 0.25 MGD of available peak flow capacity beyond the Millbridge flows. Peak flows of up to 0.25 MGD could be transferred from Rone Creek to Millbridge without impacting the Millbridge sewer system.

The lower Rone Creek developer project was analyzed for two future growth scenarios. The first scenario is for the Planning Level flow in line with assumptions made in the 2016 Wastewater Treatment Master Plan Update (Planning Level Flows). The second scenario is for the build out flow in line with assumptions for development from the Town of Waxhaw Comprehensive Plan (Build Out). The infrastructure needed for each scenario is discussed in the following sections.

6.1.1 Rone Creek Developer Project – Planning Level Flows

The first scenario is for the planning level flow in line with assumptions made in the 2016 Wastewater Treatment Master Plan Update. The basin was assumed to develop as residential 1 acre lots with 2.7 persons per dwelling. 30% of the total land area was reserved for roads, wetlands, and open space. Based on a per capita wastewater flow rate of 80 gpcd, the Rone Creek Basin would produce 0.35 MGD of average wastewater flow for the planning level scenario.

In addition to the average daily flows, the peak wet weather flow was estimated using a peaking factor of 3.0. The peak flow incorporates an infiltration and inflow (I/I) flow component for the proposed development and is in line with the peaking factors observed in the UCPW 2011 Comprehensive Master Plan I/I estimates for new developments. The peak flow of 1.1 MGD will require the 8-inch sewer in Millbridge to be replaced with a 12-inch sewer. Utilizing planning level flows, the Rone Creek developer project will consist of:

- 9,900 feet of 12-inch gravity sewer upstream of the Rone Creek PS
- 1.1 MGD Rone Creek PS
- 7,000 feet of 10-inch force main from the Rone Creek PS to Millbridge
- 6,500 of 12-inch gravity sewer extension/replacement in the Millbridge area

Based on unit costs listed in Section 5.0, the total cost for the developer project is $5.4 Million. Table 6-2 lists the costs for the components of the developer project. The costs shown include markups for contingency and engineering. The Rone Creek Improvements sized for the planning level growth scenario are shown in Figure 6-3.

Table 6-2 Rone Creek Developer Project Opinion of Probable Cost - Planning Level Scenario

PROJECT	COST
12-inch Gravity Sewer – Rone Creek Basin (9,900 ft)	$2,100,000
Rone Creek Pump Station (1.1 MGD)	$1,340,000
10-inch Force Main (7,000 ft)	$600,000
12-inch Gravity Sewer – Millbridge (6,500 ft)	$1,390,000
Total	**$5,430,000**

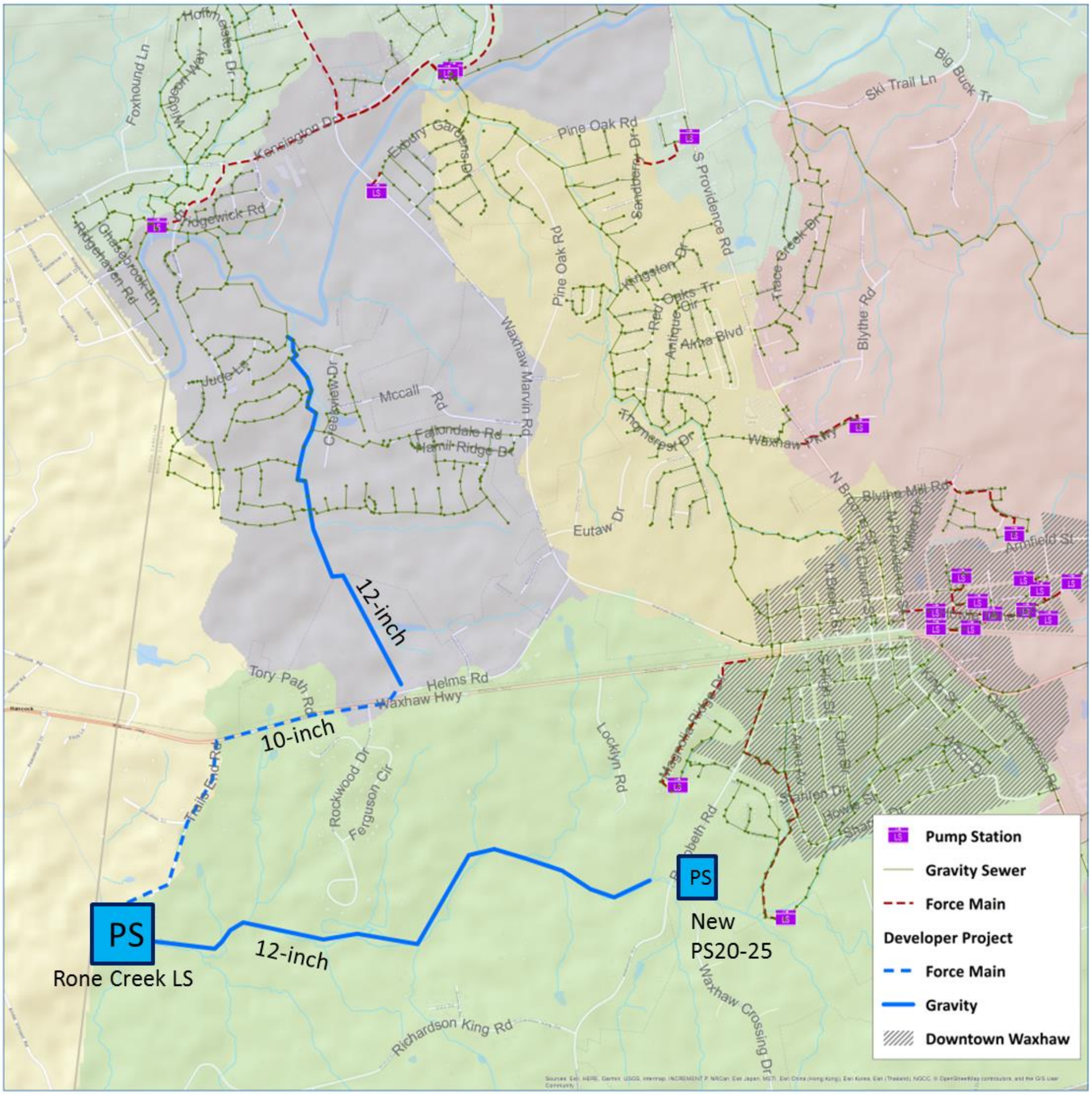


Figure 6-3 Rone Creek Developer Project - Sized for Planning Level Flows

6.1.2 Rone Creek Developer Project – Build Out Flows

The second scenario is for the build out flow in line with assumptions for development from the Town of Waxhaw Comprehensive Plan. The basin was assumed to develop with 4 residential dwellings per acre with 2.7 persons per dwelling. Again, 30% of the total land area was reserved for roads, wetlands, and open space. Based on a per capita wastewater flow rate of 80 gpcd, the Rone Creek Basin would produce 1.31 MGD of average wastewater flow for the build out scenario.

In addition to the average daily flows, the peak wet weather flow was estimated using a peaking factor of 3.0. The peak flow incorporates an infiltration and inflow (I/I) flow component for the proposed development and is in line with the peaking factors observed in the UCPW 2011

Comprehensive Master Plan I/I estimates for new developments. The peak flow of 4.0 MGD will require the sewers in Millbridge to be replaced. Utilizing the build out flows, the Rone Creek developer project will consist of:

- 9,900 feet of 18-inch gravity sewer upstream of the Rone Creek PS
- 4.0 MGD Rone Creek PS
- 7,000 feet of 16-inch force main from the Rone Creek PS to Millbridge
- 8,000 feet of 18-inch gravity sewer extension/replacement in the Millbridge area
- 1,900 feet of 24-inch gravity sewer replacement upstream of the Millbridge PS

Based on unit costs listed in Section 5.0, the total cost for the developer project is $11.2 million. Table 6-3 lists the costs for the components of the developer project. The costs shown include markups for contingency and engineering. The Rone Creek Improvements sized for the build out growth scenario are shown in Figure 6-4.

Table 6-3 Rone Creek Developer Project Opinion of Probable Cost - Build Out Scenario

PROJECT	COST
18-inch Gravity Sewer – Rone Creek Basin (9,900 ft)	$3,020,000
Rone Creek Pump Station (4.0 MGD)	$4,770,000
16-inch Force Main (7,000 ft)	$950,000
18-inch Gravity Sewer – Millbridge (8,000 ft)	$2,450,000
24-inch Gravity Sewer – Millbridge (1,900 ft)	$790,000
Total	**$11,190,000**

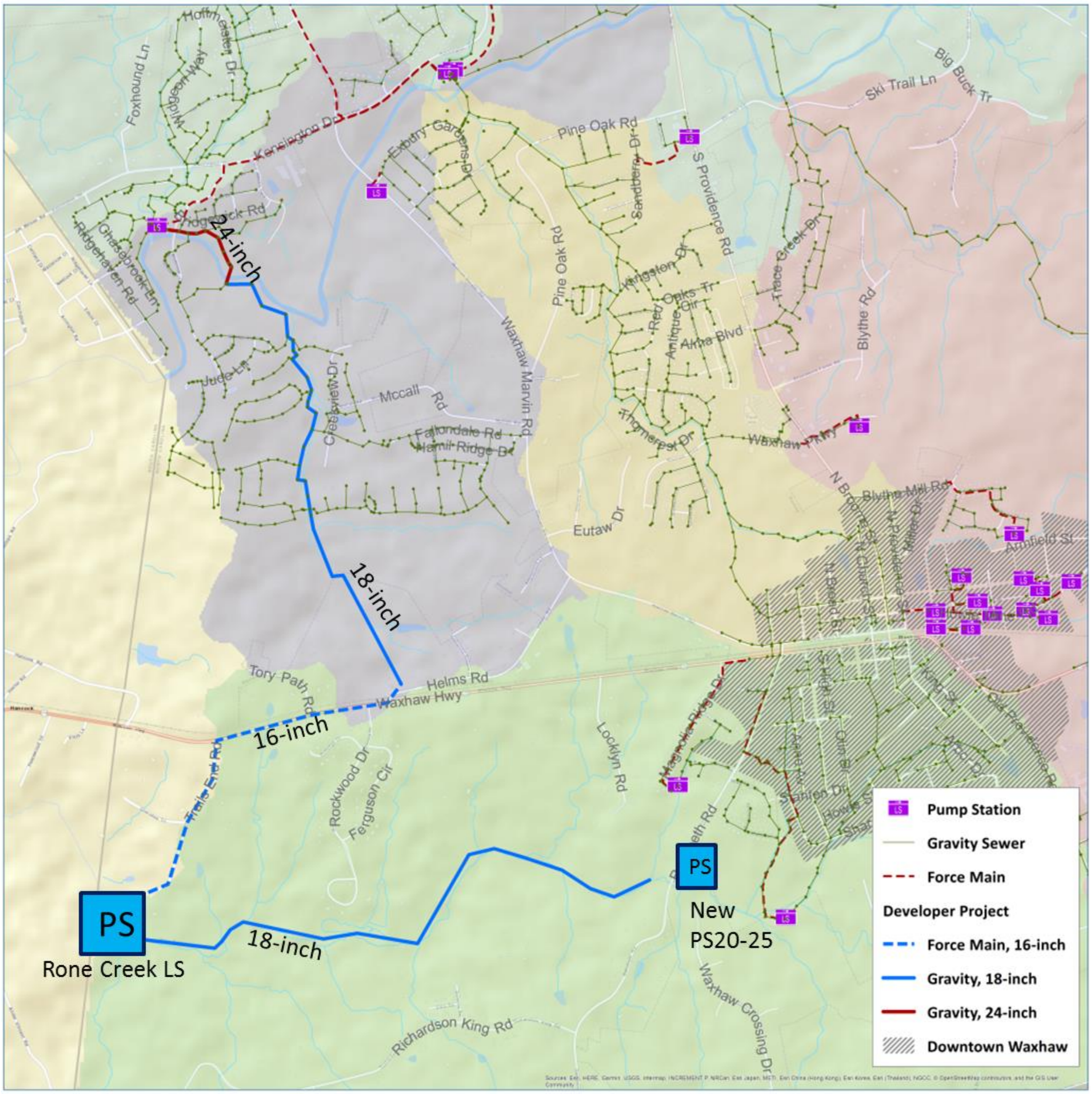


Figure 6-4 **Rone Creek Developer Project - Sized for Waxhaw Comp Plan Flows**

Appendix A. Model Update and Calibration

A.1 FLOW MONITORING DATA

The flow monitoring program was three months long and included data from four temporary flow meters along with one rain gauge site. Details of the flow monitoring program are included in the Waxhaw Flow Monitoring Report by Frasier Engineering dated February 8, 2017. The locations of the flow meters and rain gauge are shown in Figure A-1. Figure A-2 is the schematic of the flow meters in the Waxhaw Sewer and Millbridge PS areas.

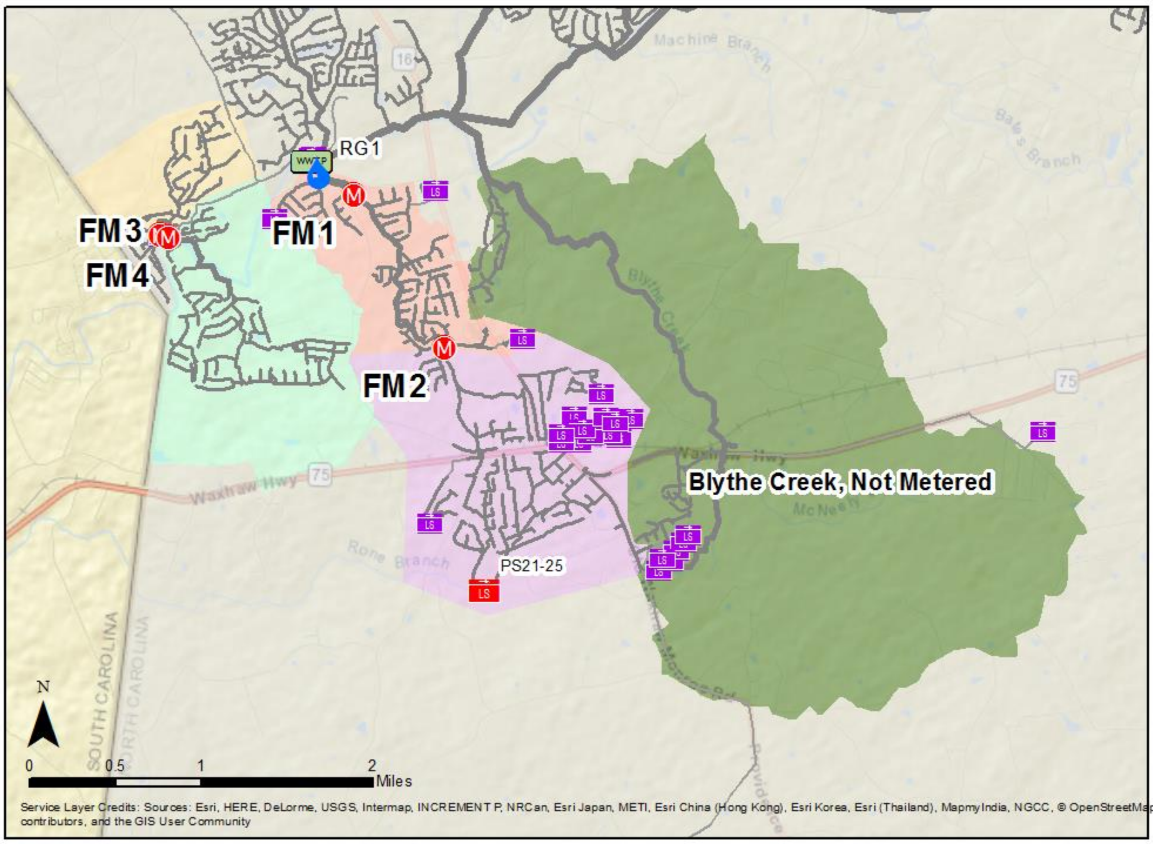


Figure A-1 Waxhaw Sewer Rain Gauge and Flow Meter Locations with Delineated Subsystems

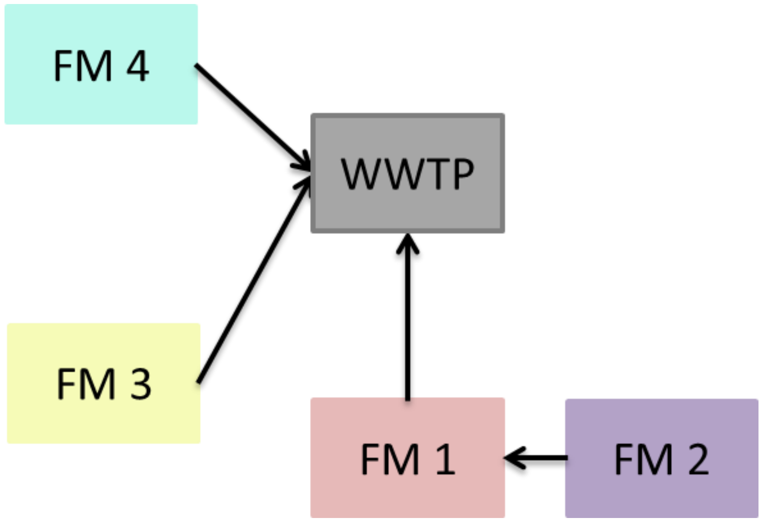


Figure A-2 Waxhaw Sewer Basin Flow Meter Schematic

A.1.1 Flow Data Capacity Concerns

The flow data provided for the model calibration was reviewed to determine if collection system capacity concerns existed at the four monitoring locations. The data was reviewed to look for typical signs of capacity issues which can include the following:

- The monitoring location showed signs of surcharging.
- Reduced velocities at higher depths, usually an indication of a downstream restriction of flow.
- Hydrographs that look attenuated can be an indication of an upstream restriction of flow.

Meter #1 – MH-2099

- Low depths are higher than would be expected indicating a possible blockage or raised pipe invert downstream of the flow meter affecting depths near 3 inches.
- During high flows, the depth does not exceed the pipe crown, but the velocities are lower than would be expected indicating a blockage or reduced capacity downstream.

Meter #2 – MH-1838

- During high wet weather flows, the depth does not exceed the pipe crown, but some of the velocity readings stray from typical velocities.
- It appears that when monitored flows reach about 0.40 MGD, possibly one pump on at PS21-25, the velocities slow down. The scatter graphs show the velocities around 2 fps as opposed to the expected 3 fps.
- It appears that when monitored flows reach about 0.65 MGD, possibly two pumps on at PS21-25, the velocities speed up. The scatter graphs show the velocities about 4.5 fps as opposed to the expected 3 fps.

Meter #3 – MH-11059

- During high wet weather flows, the depth does not exceed the pipe crown.
- One storm event appears to have a downstream blockage or capacity restriction as the increases from about 3 inches to about 6 inches and then drops back down about 20 hours later.

Meter #4 – MH-12482

- During high wet weather flows, the depth does not exceed the pipe crown.
- One storm event appears to have a downstream blockage or capacity restriction as the depth increases from about 3 inches to about 4 inches and then drops back down about 16 hours later.
- Flow data showed some rather high velocities for the depth being monitored. Possibly caused by downstream conditions and the operation of the Millbridge PS.

In general, the flow data showed no specific capacity problems at the monitoring locations, but there were possible issues downstream or upstream of these locations.

A.2 DRY WEATHER FLOW DEVELOPMENT

The average daily dry weather flow (ADDF) includes contributions from all customers (base sanitary) in the collection system as well as groundwater infiltration (GWI) into the collection system. The flows are entered into the model assigning the ADDF to the modeled manholes with a diurnal pattern to represent the increases and decreases in the ADDF throughout the day. A separate diurnal pattern was developed for the weekends to represent the different patterns and flows monitored during the weekend periods.

The rain gauge and flow meter data were reviewed for weekday and weekend periods not influenced by rainfall events. The period between October 31st and November 4th was selected to generate typical weekday diurnal flow patterns per meter and the period of November 5th and November 6th was selected to generate typical weekend diurnal flow patterns per meter.

Table A-1 summarizes the ADDF monitored for both the weekday and weekend periods.

Table A-1 Basin Average Daily Dry Weather Flow Loadings

FLOW METER #	MANHOLE NUMBER	WEEKDAY		WEEKEND	
		CUMULATIVE ADDF (MGD)	BASIN ADDF (MGD)	CUMULATIVE ADDF (MGD)	BASIN ADDF (MGD)
1	MH-2099	0.274	0.137	0.298	0.170
2	MH-1838	0.137	0.137	0.128	0.128
3	MH-11059	0.138	0.138	0.145	0.145
4	MH-12482	0.078	0.078	0.080	0.080
Total			0.490		0.451

Daily fluctuations in dry weather flows are attributable to variations in domestic, industrial, and commercial wastewater production. Figure A-3 shows a typical diurnal pattern.

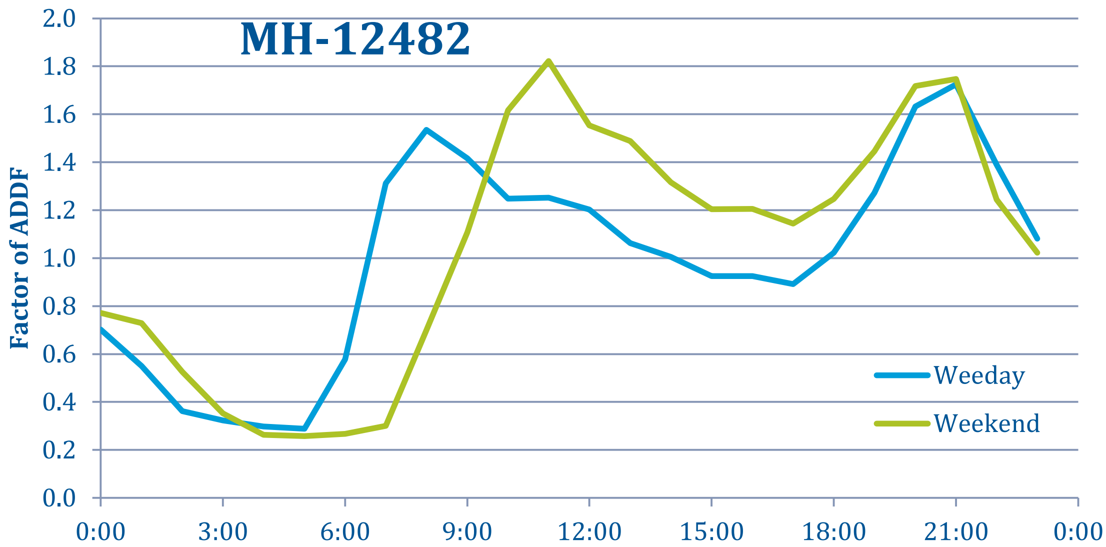


Figure A-3 Typical Diurnal Pattern

The ADDF was entered into the model using the population field and the flow meter basin-specific per capita rate. The monitored ADDF and diurnal pattern data was included in the model in one of two ways depending if the area monitored already existed in the model or if the area was added to the model as part of the update.

The Waxhaw Sewer which already existed in the model with population equivalents, per capita flows and diurnal patterns from the previous modeling work was updated with the information monitored with meters #1 and #2. The per capita flows were adjusted so the total basin flow matched what was monitored and the diurnal patterns were changed.

The Millbridge PS area of the collection system didn't exist in the model so the monitored flows were evenly distributed to all manholes upstream of the flow meters. Population equivalent data was not available for the Millbridge PS area so the population field was defaulted to equal one. Therefore, the population field contains the number of manhole loadings assigned to the manhole. Most manholes will have a loading of one but modeled manholes that received flows from non-modeled manholes will have a higher loading value. The two meters in the Millbridge PS area did not monitor the entire area upstream of the pump station. The two monitored areas were reviewed and compared to the unmonitored area and the area upstream of meter #3 best resembled the unmonitored area so the meter #3 ADDF unit flow and diurnal pattern were assigned to the unmonitored area evenly distributing the flows and totaling to the modeled manholes.

A.3 DRY WEATHER CALIBRATION

The model was calibrated under dry weather flow conditions to a dry weather period between October 31 and November 7, 2016. This period included both weekdays and weekend days. The dry weather calibration included the adjustment of the diurnal patterns to match observed meter flow records, and a validation of model simulated depth and velocity results to observed meter readings utilizing the observed flow meters depth/velocity scatter.

Qualitative and quantitative comparisons were used as metrics for assessing the dry weather calibration. The dry weather goals were developed based on guidelines from the United Kingdom'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 A-2. Specific goals were not met in all cases due to a variety of reasons, including metering equipment failures, unsatisfactory meter location and accuracy, system repairs, system blockages, rainfall variability, and short-term system anomalies, etc. The qualitative comparisons (shape and timing) are the primary goal for assessing the match between the model and metered data. Only after the qualitative goals are met, the quantitative comparisons were determined.

Table A-2 Dry Weather Model Calibration Goals

METRIC	DRY WEATHER CALIBRATION GOAL
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 flow and depth
Peak Flow	±10% of measured values
Volume	±10% of measured values
Peak Depth	± 0.3 feet at non-surcharged locations or −0.3 to +1.67 feet at surcharged locations

The diurnal patterns and wastewater loadings produced in the dry weather flow analyses were input into the model to generate dry weather flow in the model. The model results were compared to the observed flow meter data and the hydrographs were adjusted to reasonably match the typical dry weather flow pattern of each basin.

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

Figure A-4 is a sample calibration plot showing the match between the model results and the metered data for the calibration period. The dry weather flow attributes for the model simulated results (red) are adjusted until it closely matches the observed flow results (blue) at each meter location. Appendix A contains all of the dry weather calibration plots for each of the metering basins. Table A-3 presents the overall results of the dry weather calibration of the metering locations for volume, peak flow and peak depth. The results are summarized for percent error of peak flow and volume along with the absolute difference for the peak depth. All the dry weather calibration goals were met for the four flow meters.

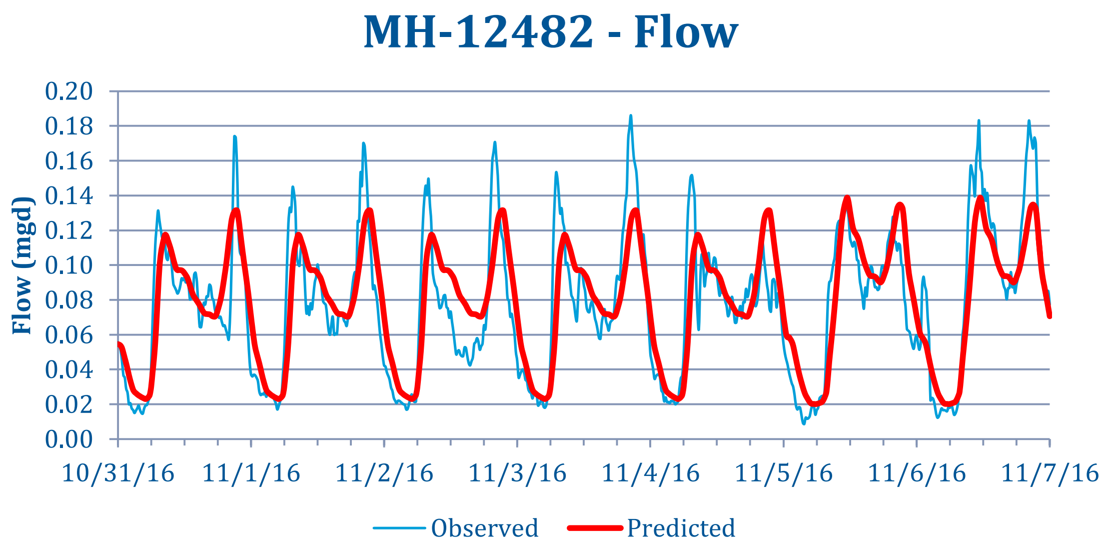


Figure A-4 **Sample Dry Weather Calibration Plot**

Table A-3 **Dry Weather Model Calibration Summary**

FLOW METER #		PEAK FLOW (MGD) *			VOLUME (MG) *			PEAK DEPTH (IN) *		
		GOAL: ±10% OF OBSERVED VALUES, DEVIATION IS BASED ON SIM - OBS			GOAL: ±10% OF OBSERVED VALUES, DEVIATION IS BASED ON SIM - OBS			GOAL: ± 0.3FT (3.6IN) AT NON-SURCHARGED LOCATIONS OR –0.3FT (3.6IN) TO +1.67FT (20 IN) AT SURCHARGED LOCATIONS		
		OBS	SIM	DEVIATION	OBS	SIM	DEVIATION	OBS	SIM	DEVIATION
Weekday	1	0.460	0.423	-8.0%	0.273	0.280	2.7%	6.711	4.104	-2.607
	2	0.240	0.235	-1.9%	0.131	0.135	2.5%	3.730	3.432	-0.298
	3	0.225	0.207	-8.2%	0.139	0.140	0.9%	4.275	2.736	-1.539
	4	0.140	0.132	-6.1%	0.076	0.078	2.9%	3.164	2.304	-0.860
Weekend	1	0.500	0.466	-6.8%	0.286	0.286	-0.1%	6.720	4.308	-2.412
	2	0.250	0.247	-1.2%	0.121	0.129	6.8%	3.757	3.504	-0.253
	3	0.225	0.210	-6.6%	0.152	0.147	-3.0%	4.064	2.760	-1.304
	4	0.154	0.139	-9.9%	0.083	0.081	-2.0%	3.048	2.364	-0.684
	* Negative values mean that the simulated values were less than the metered values									

Figure A-5, Figure A-6 and Figure A-7 summarize the overall agreement between the metered and the modeled results for peak depth, peak flow, and volume for all of the meter locations. The data is 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 in these figures, the model generally matches the observed data within the acceptable range of calibration.

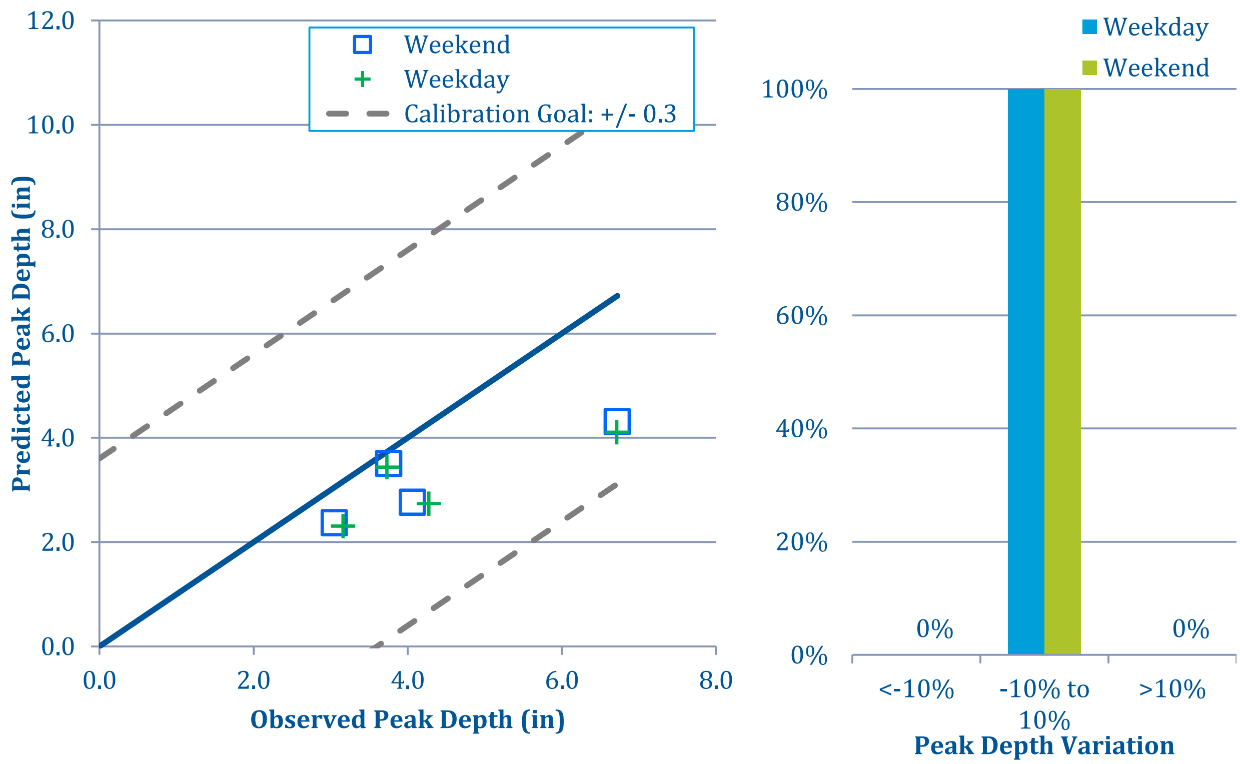


Figure A-5 Peak Depth Scatter Plot – Dry Weather

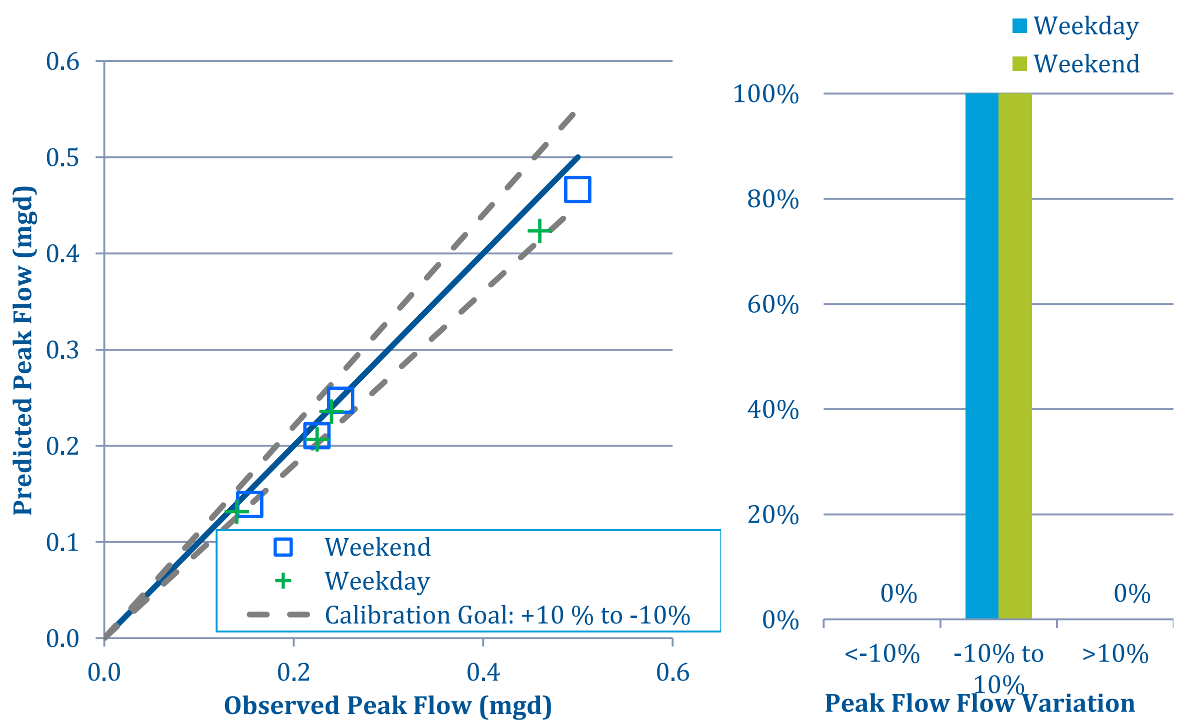


Figure A-6 Peak Flow Scatter Plot – Dry Weather

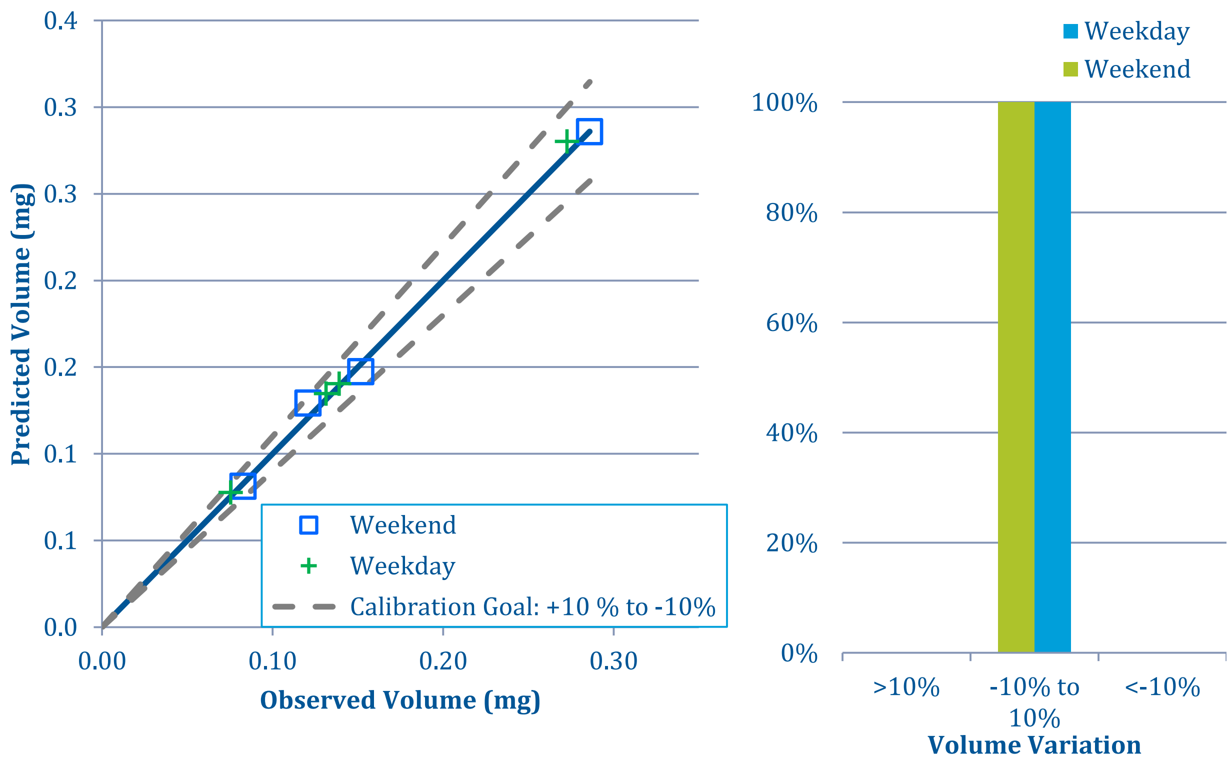


Figure A-7 Flow Volume Scatter Plot – Dry Weather

A.4 WET WEATHER CALIBRATION

The Waxhaw Sewer model was also calibrated to wet weather conditions. To perform a wet weather calibration, significant storm events are used in the wet weather calibration of the model.

A.4.1 Calibration Events

Significant storm events were identified by collecting rainfall data from the rain gauge used in the monitoring study. The process of choosing a significant storm event required the following:

- Rainfall gauge collected data with a significant depth (> 0.75 inches)
- Sewer flow data recorded an observable wet weather response at specific locations within the system

Wet weather responses were evaluated at each of the flow monitor sites during the temporary monitoring period which took place between October 2016 and January 2017 for a total of three months. The first six weeks of the monitoring period had only one rain event of 0.14 inches. The first significant event occurred nine weeks into the monitoring period on December 29th. Though some rainfall events occurred during weeks seven through nine, the flow monitors did not show significant wet weather responses. The reasons for this are most likely due to antecedent soil moisture conditions related to the drought experienced in the area during summer and fall of 2016. This resulted in most of the rainfall being absorbed into the soil verses entering the sanitary sewer system until the winter when more rain fell and the evapotranspiration decreased.

One of the criteria in choosing a storm event for the wet weather calibration is a storm event with high rainfall depth. The storm events with the greater rainfall depths were selected from the rain gauge data. It should be noted that rainfall that occurred within twelve hours was classified as the same event. This caused some back-to-back storm events to be grouped together as a larger event. It should be noted that meter #4 had a malfunction so some of the flow data was not collected which caused the selection of different storms for calibration.

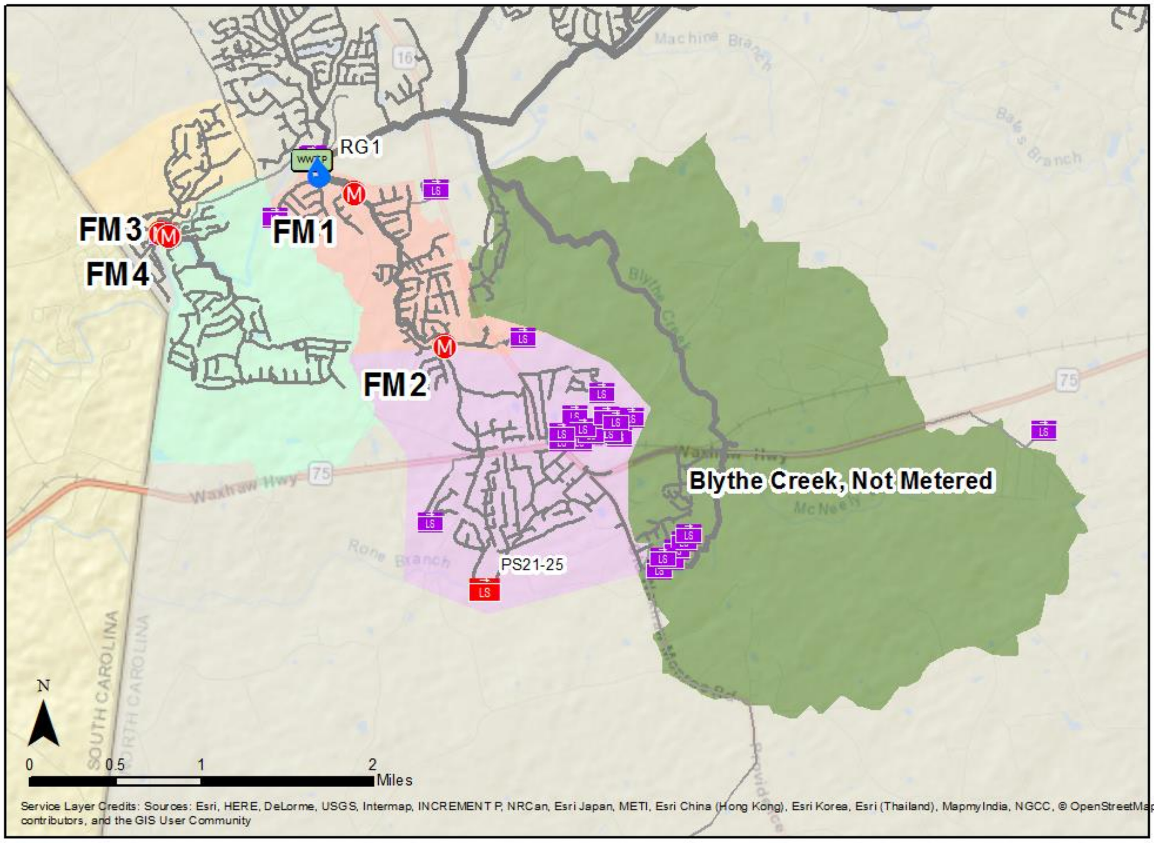


Figure A-8 Meter Basins

Figure A-8 illustrates the location of the rainfall gauge in relation to the monitoring basins discussed in previous sections. The following rainfall events were selected for the wet weather calibration events:

Meters #1, #2, #3

- December 18th, 2017 (0.61 Inches)
- January 1st, 2017 (1.65 Inches)
- January 6th, 2017 (0.74 Inches)
- January 21st, 2017 (3.35 Inches)

Meter #4

- December 18th, 2017 (0.61 Inches)
- January 21st, 2017 (3.35 Inches)

A.4.2 Wet Weather Flows

In a sanitary system, the rainfall dependent inflow and infiltration (RDII) is the increase in flows in a collection system resulting directly from rain events. RDII is driven by a myriad of factors including:

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

InfoWorks ICM uses a minimum of two hydrologic models, a volume and a routing model, to represent wet weather flows into the collection system. The model also contains additional parameters to predict how dry the soils will be prior to a rainfall event. The volume model parameter, initial loss, is the amount water that must be stored in the basin before any system response is observed. The initial loss can be set specifically for each of the responses (fast, medium, slow, etc.).

A.4.3 Calibration Results

A sample calibration plot is illustrated below in Figure A-9. The wet weather calibration plots can be found in Appendix C. The wet weather response parameters for the model simulated results (red) were adjusted until closely matching the observed flow results (blue) at each meter location.

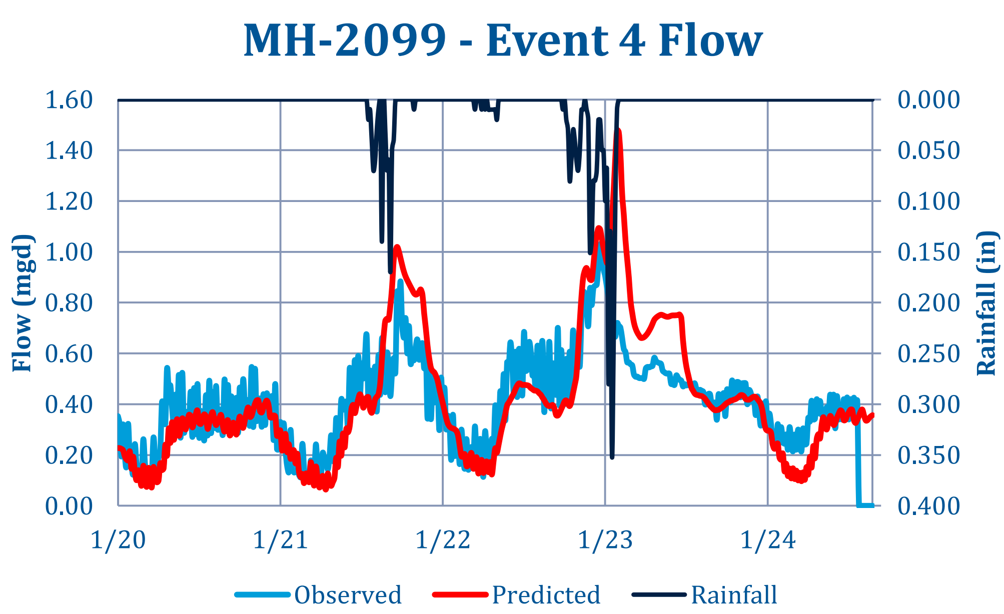


Figure A-9 Sample Calibration Model Result Graphs for a Wet Weather Event

The goal of the calibration was to represent the flows and depths of each storm for each meter and develop a simulation 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 parameters within realistic ranges. The wet weather flow parameters were adjusted in order 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.

The quantitative wet weather goals are Union County's guidelines as shown in Table A-4. Specific goals cannot be met in all cases due to a variety of reasons, including metering equipment failures, unsatisfactory meter location and accuracy, system repairs, system blockages, rainfall variability, and 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, the quantitative comparisons are determined.

Table A-4 Wet Weather Model Calibration Goals

METRIC	WET WEATHER CALIBRATION GOALS
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
Volume	-10% to +20% of measured values
Peak Depth	-0.3 feet to +1.67 feet at surcharged locations ±0.3 feet at non surcharged locations

The calibration process was difficult to get meter #2 to match because there was a dampened extended peak monitored after the peak rainfall events. During discussions with County Staff it was decided to break the area upstream of meter #2 into two sections, one for the area upstream of PS21-25 and one downstream. The area upstream of PS21-25 was calibrated using different parameters than the area downstream. The pump station has inadequate capacity for larger storm events and this allowed for better calibration results.

Perhaps 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 below in Table A-5 including the PS21-25 service area as a separate result. 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 percentage of greater than 3% is considered moderate in a separate sanitary sewer system. The only area with a total percentage above 3% is PS21-25 area. The PS21-25 service area captures flows from the downtown Waxhaw area where most of the pipes are considerably older than other parts of the Waxhaw Sewer service area.

Table A-5 Calibrated Total Runoff Percentage

METER #	TOTAL RUNOFF PERCENT CONTRIBUTING
1	0.85%
2	0.70%
3	1.05%
4	0.35%
PS21-25	3.50%

Table A-6 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 periods of low or high dry weather loadings, possible flow meter errors, temporary system blockages, and erroneous meter spikes.

Table A-6 Wet Weather Calibration Results

FLOW METER #		PEAK FLOW (MGD) *			VOLUME (MG) *			PEAK DEPTH (IN) *		
		GOAL: -10% TO +20% OF OBSERVED VALUES, DEVIATION IS BASED ON SIM - OBS			GOAL: -15% TO +25% OF OBSERVED VALUES, DEVIATION IS BASED ON SIM - OBS			GOAL: ± 0.3FT (3.6IN) AT NON-SURCHARGED LOCATIONS OR –0.3FT(3.6IN) TO +1.67FT(20IN) AT SURCHARGED LOCATIONS		
		OBS	SIM	DEVIATION	OBS	SIM	DEVIATION	OBS	SIM	DEVIATION
12/18/16	1	0.725	0.724	-0.1%	0.524	0.523	-0.2%	7.314	6.888	-0.426
	2	0.471	0.493	4.6%	0.288	0.262	-9.1%	4.610	5.772	1.162
	3	0.238	0.247	3.7%	0.208	0.243	17.2%	3.933	2.028	-1.905
	4	0.195	0.140	-28.4%	0.143	0.136	-5.1%	3.311	3.612	0.301
1/1/17	1	0.540	0.860	59.3%	0.574	0.618	7.7%	7.412	7.740	0.328
	2	0.726	0.594	-18.1%	0.458	0.343	-25.1%	5.100	7.020	1.920
	3	0.280	0.271	-3.4%	0.249	0.251	1.0%	3.994	2.100	-1.894
	4	-	-	-	-	-	-	-	-	-
1/6/17	1	0.523	0.581	11.1%	0.416	0.442	6.2%	7.251	6.012	-1.239
	2	0.442	0.419	-5.1%	0.282	0.228	-19.0%	5.140	5.136	-0.004
	3	0.287	0.214	-25.5%	0.200	0.197	-1.5%	4.016	1.956	-2.060
	4	-	-	-	-	-	-	-	-	-
1/21/17	1	1.033	1.478	43.1%	0.634	0.653	3.0%	10.260	17.328	7.068
	2	0.730	0.782	7.1%	0.408	0.376	-7.7%	6.930	21.432	14.502
	3	0.471	0.479	1.7%	0.259	0.244	-5.9%	6.299	2.496	-3.803
	4	0.469	0.268	-42.9%	0.175	0.142	-18.9%	4.783	5.064	0.281

* Negative values mean that the simulated values were less than the metered values
- Not used as a calibration point

Figure A-10, Figure A-11, and Figure A-12 present the 1:1 scatter plots for the wet weather calibration for peak flow, volume, and peak depth for each of the storm events, respectively. 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 model and monitored data. The figure also includes the red and orange lines to represent the percent difference (or absolute difference for peak depth) defining the wet weather calibration goals. These calibration comparisons were calculated based on the monitoring data and model results for each of the calibration storms. Each of the calibration storms are represented by a set of individual points for each monitoring location, which are the paired monitored values and modeled results for a specific flow during the calibration period. Adjacent to each scatter plot is a histogram indicating the percentage of the calibration points that fell within the calibration goals.

The peak flow calibration goals were met for three meters for the first event, two meters for the second event, two meters for the third event and two meters for the forth event. The volume calibration goal was met for all the meters for the first event, third, and forth events and two meters for the second event. The calibration goals for peak depth were met all the meters for the first, second and third events and one of the meters for the forth event.

The results balance 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 for a variety of storm events.

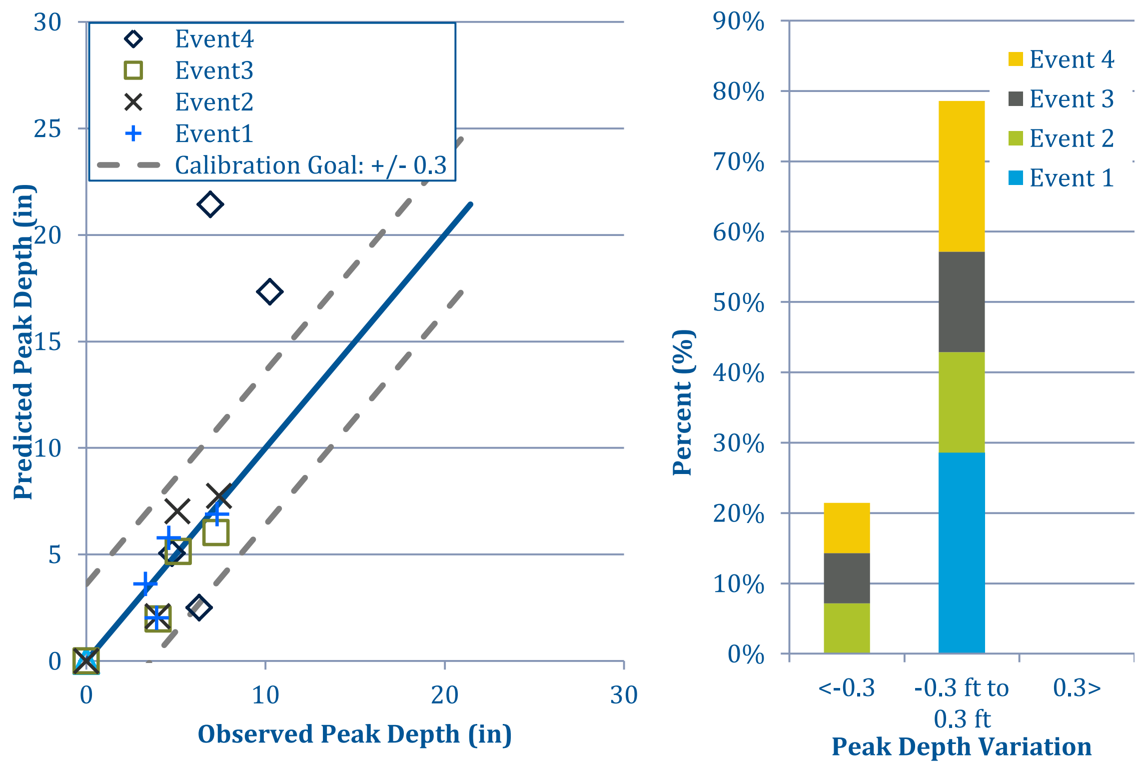


Figure A-10 Peak Depth Scatter Plot – Wet Weather

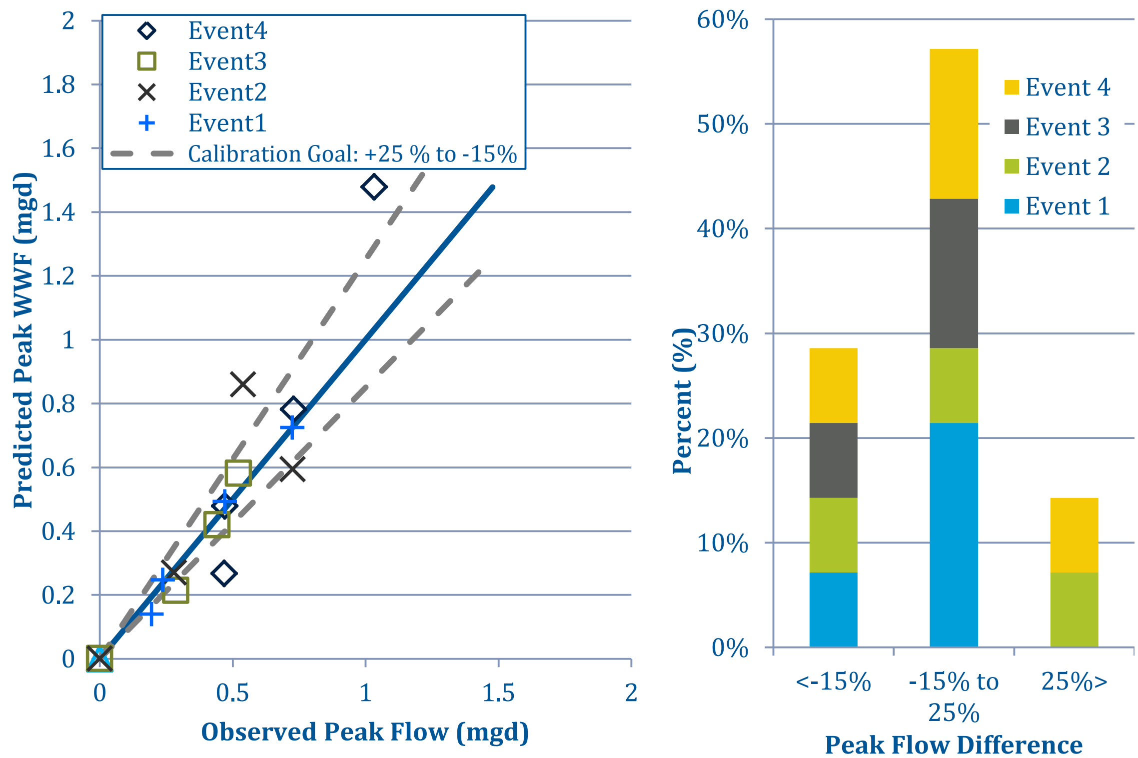


Figure A-11 **Peak Flow Scatter Plot – Wet Weather**

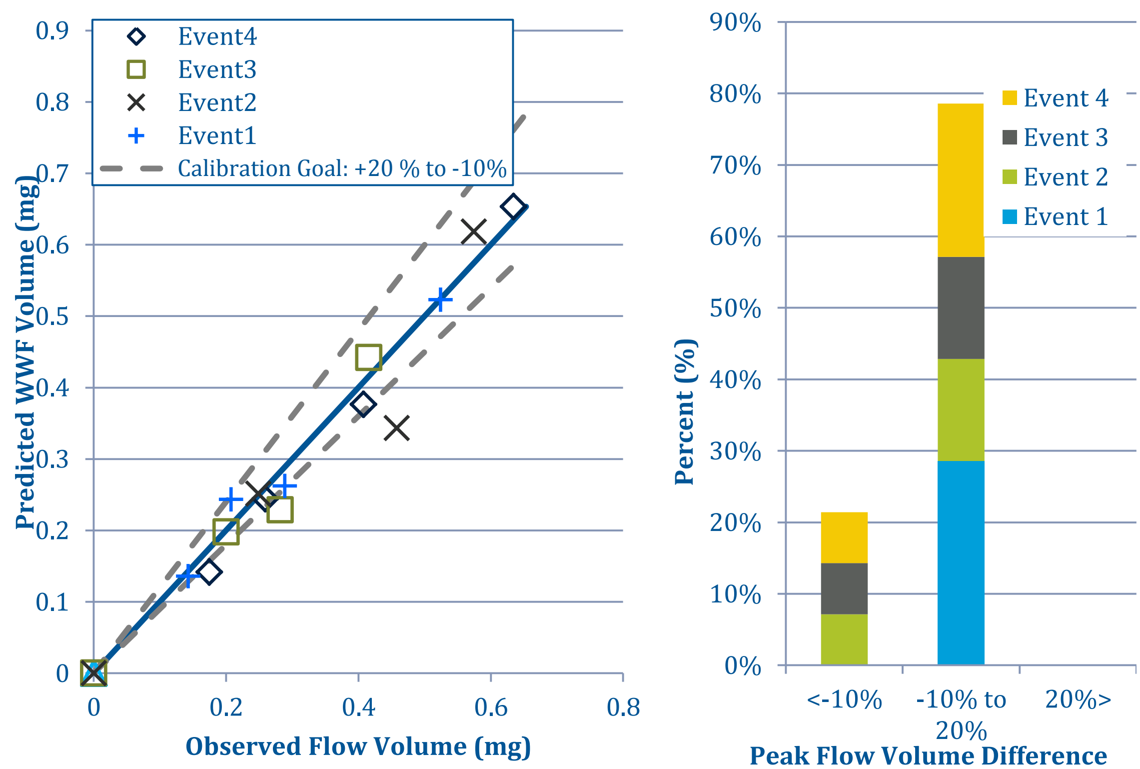


Figure A-12 **Flow Volume Scatter Plot – Wet Weather**

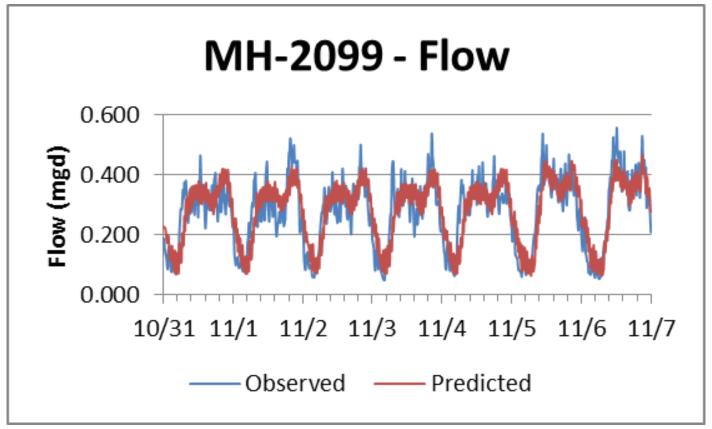
The model matches the observed peak depths, peak flows, and volumes with reasonable accuracy and generally within the target ranges for those flow meters. In general, the calibration scatter plots demonstrate the model's accuracy in meeting the calibration goals. A summary of the calibration confidence at each meter is given in Table A-7.

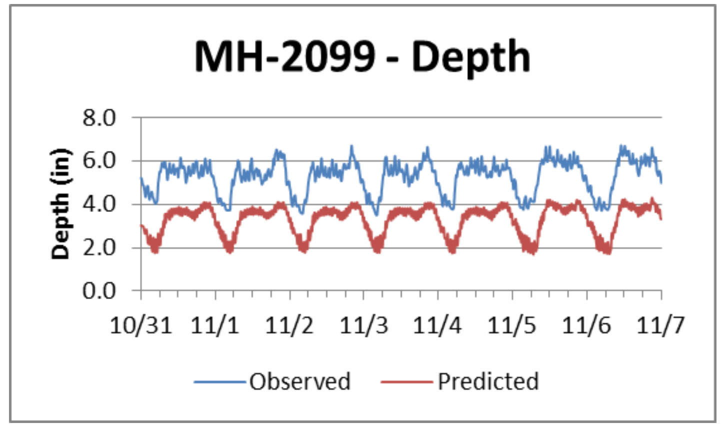
Table A-7 Calibration Confidence

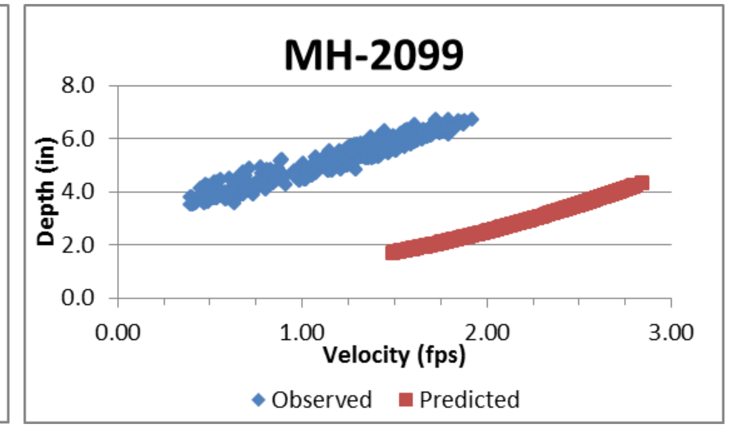
METER	DESCRIPTION	CALIBRATION GOALS	CONFIDENCE / CALIBRATION RESULTS
1	12" Waxhaw Outfall; MH-2099 (Downstream of Site 2)	Reasonably match flow and depth throughout each calibration event.	Good – reasonable calibration to both flow and depth under storm events. During storm events, Meter 1 recorded less flow than the Meter 1 dry weather flows plus the storm flow recorded at the upstream meter. The small incremental increase in flow from Meter 2 to Meter 1 caused the model to over predict the peak flow at Meter 1 compared to the observed value. The over prediction on Meter 1 is balance by under prediction of peak flow for Meter 2.
2	8" Waxhaw Outfall; MH-1838	Reasonably match flow and depth throughout each calibration event. This meter is located downstream of PS21-25. The peak flows from the station appeared were limited based on the station capacity.	Good - reasonable calibration to both flow and depth under storm events. The capacity restriction at PS21-25 affected the peak flows measured at the downstream meter. The goal was to match volume from PS21-25 in order to determine the I/I volume from the downtown Waxhaw area. The modeled peak flow values were balanced with some of the modeled values being reported higher and some lower than the observed values.
3	10" Millbridge Sewer MH-11059 (North side of Millbridge)	Reasonably match flow and depth throughout each calibration event.	High – reasonable calibration to both flow and depth under storm events. This meter also recorded higher depths several hours after the January 22nd rainfall. The timing of the peaks could be due to back up from the Millbridge PS during that single event.
4	12" Millbridge Sewer; MH-12482 (South side of Millbridge)	Reasonably match flow and depth throughout each calibration event. This meter did not record the January 2nd and January 6th events.	Good – reasonable calibration to both flow and depth under storm events. This meter also recorded higher depths several hours after the January 22nd rainfall. The timing of the peaks could be due to back up from the Millbridge PS during that single event, as a similar increase was seen at the other Millbridge meter.

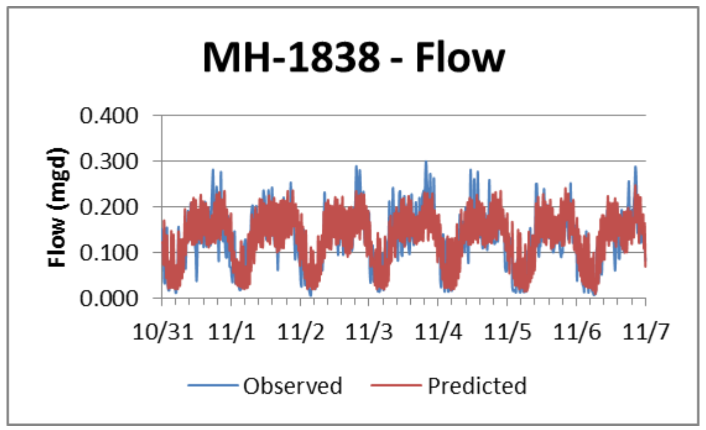
Appendix B. Dry Weather Calibration Plots

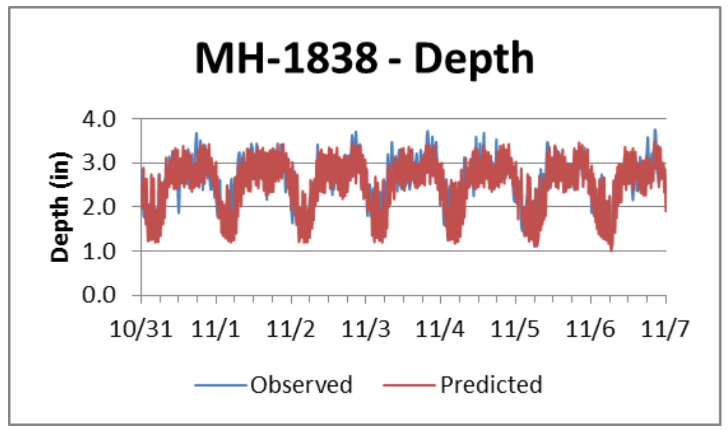
The primary calibration goal for the dry weather calibration is the shape and timing of the modeled and metered curves shown in the calibration plots.

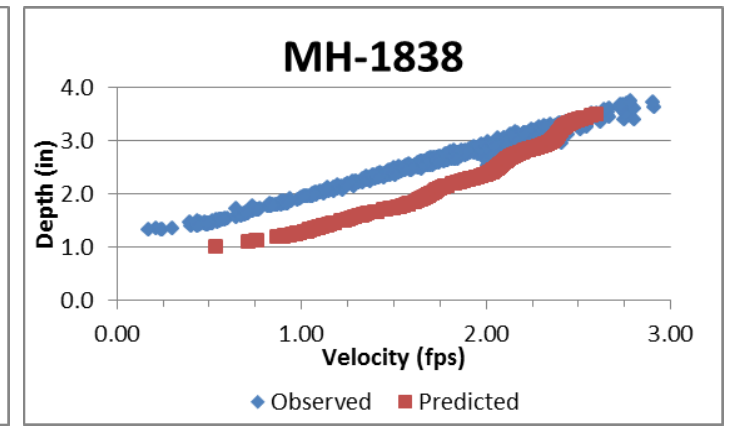

MH-2099 - Flow
0.600
0.400
0.200
0.000
Flow (mgd)
10/31 11/1 11/2 11/3 11/4 11/5 11/6 11/7
Observed
Predicted

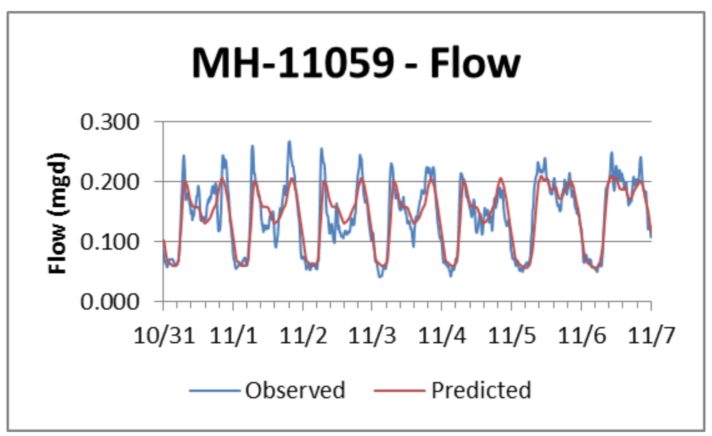

MH-2099 - Depth
8.0
6.0
4.0
2.0
0.0
Depth (in)
10/31 11/1 11/2 11/3 11/4 11/5 11/6 11/7
Observed
Predicted

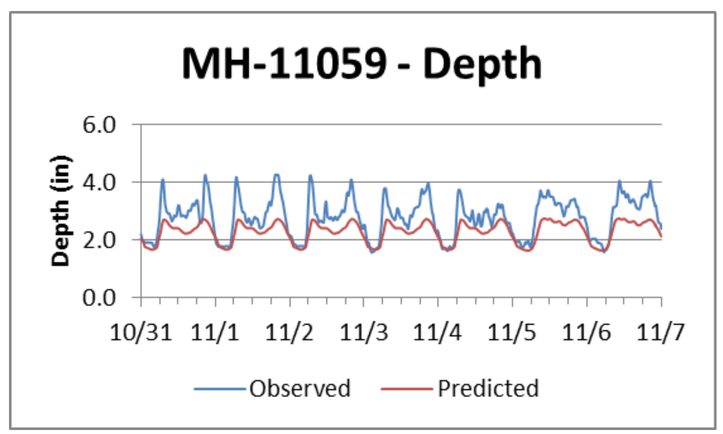

MH-2099
8.0
6.0
4.0
2.0
0.0
Depth (in)
0.00 1.00 2.00 3.00
Velocity (fps)
Observed
Predicted

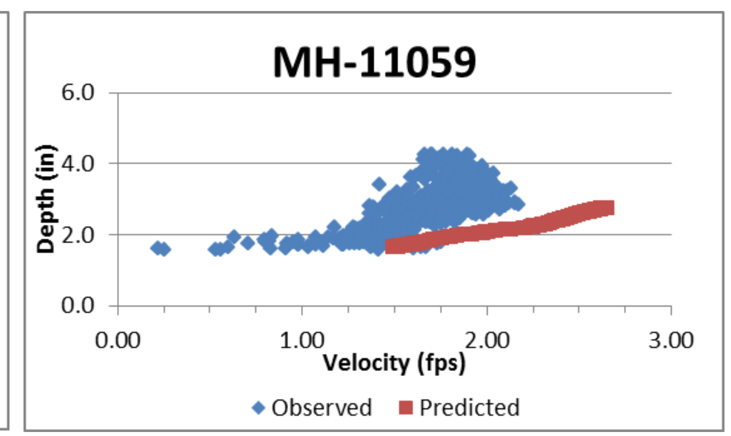

MH-1838 - Flow
0.400
0.300
0.200
0.100
0.000
Flow (mgd)
10/31 11/1 11/2 11/3 11/4 11/5 11/6 11/7
Observed
Predicted

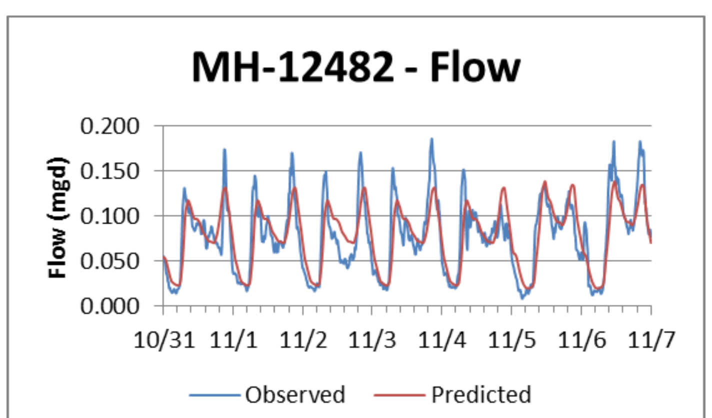

MH-1838 - Depth
4.0
3.0
2.0
1.0
0.0
Depth (in)
10/31 11/1 11/2 11/3 11/4 11/5 11/6 11/7
Observed
Predicted

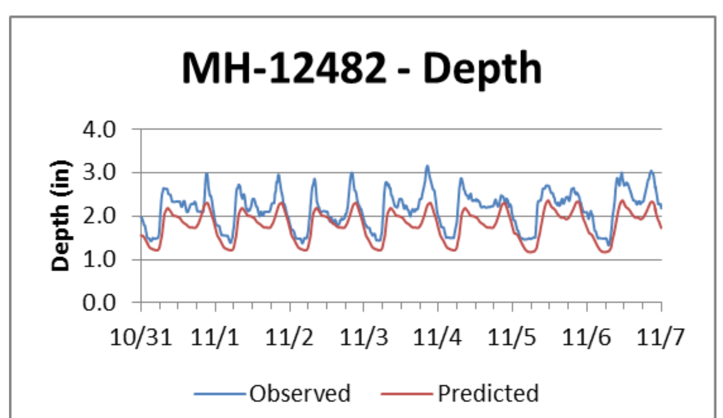

MH-1838
4.0
3.0
2.0
1.0
0.0
Depth (in)
0.00 1.00 2.00 3.00
Velocity (fps)
Observed
Predicted

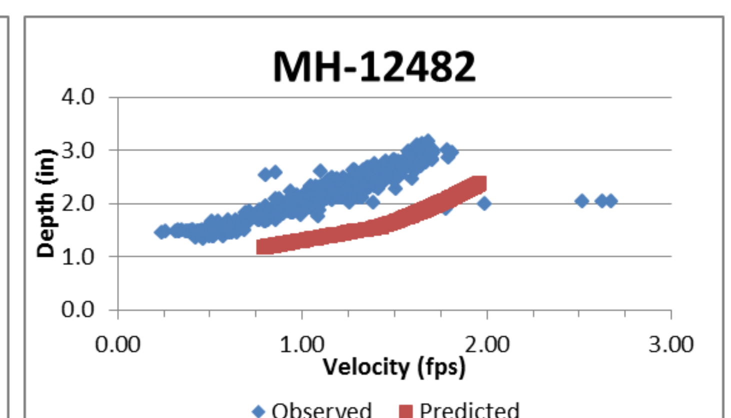

MH-11059 - Flow
0.300
0.200
0.100
0.000
Flow (mgd)
10/31 11/1 11/2 11/3 11/4 11/5 11/6 11/7
Observed
Predicted


MH-11059 - Depth
6.0
4.0
2.0
0.0
Depth (in)
10/31 11/1 11/2 11/3 11/4 11/5 11/6 11/7
Observed
Predicted


MH-11059
6.0
4.0
2.0
0.0
Depth (in)
0.00 1.00 2.00 3.00
Velocity (fps)
Observed
Predicted

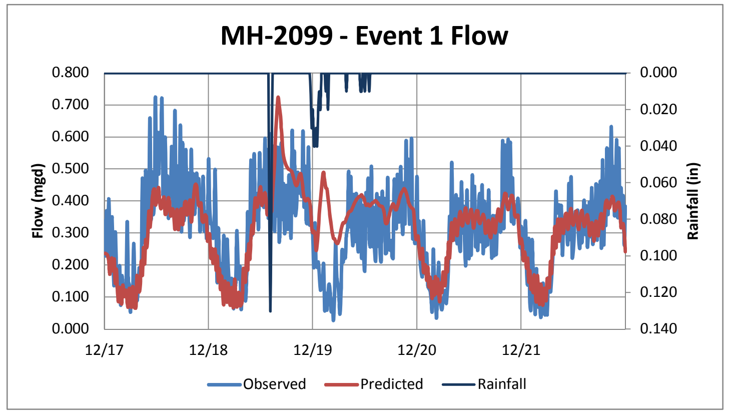

MH-12482 - Flow
0.200
0.150
0.100
0.050
0.000
Flow (mgd)
10/31 11/1 11/2 11/3 11/4 11/5 11/6 11/7
Observed
Predicted

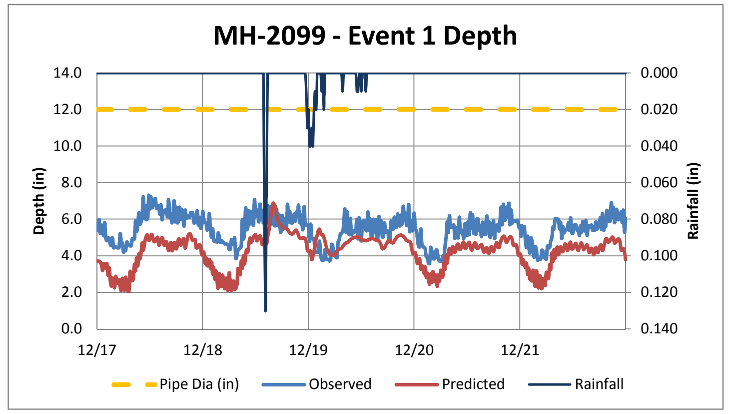

MH-12482 - Depth
4.0
3.0
2.0
1.0
0.0
Depth (in)
10/31 11/1 11/2 11/3 11/4 11/5 11/6 11/7
Observed
Predicted

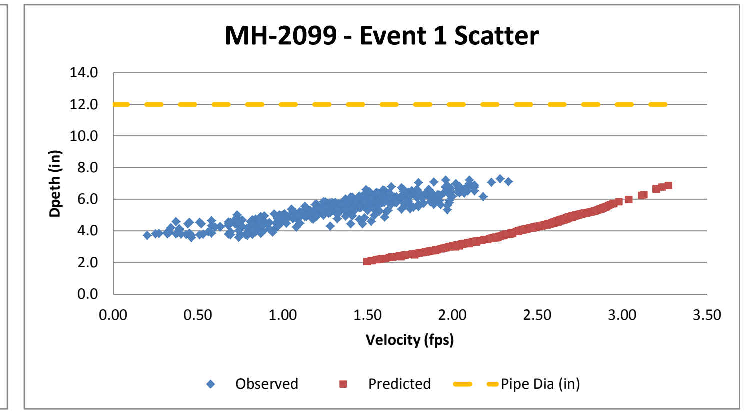

MH-12482
4.0
3.0
2.0
1.0
0.0
Depth (in)
0.00 1.00 2.00 3.00
Velocity (fps)
Observed
Predicted

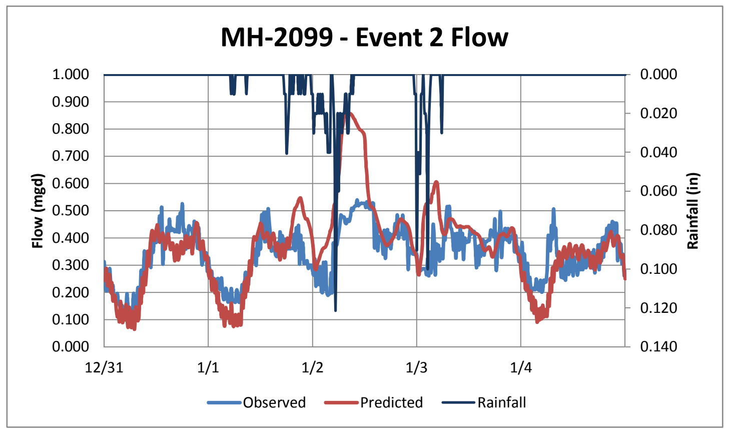
Appendix C. Wet Weather Calibration Plots

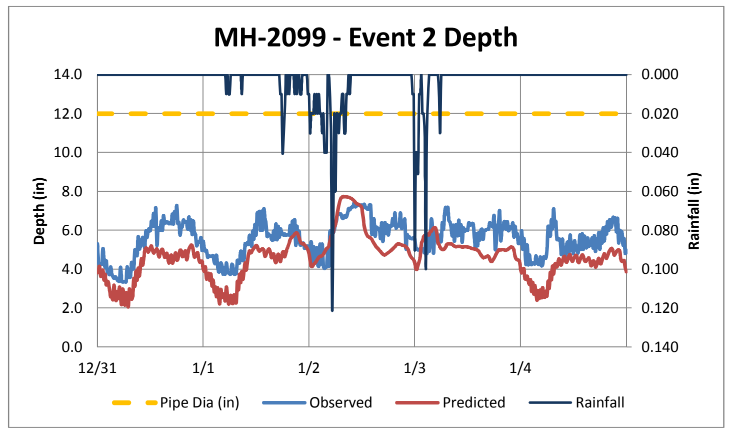
The attached plots show the storm calibrations for each calibrated flow meter. The primary calibration goal for the wet weather calibration is the shape and timing of the modeled and metered curves shown in the calibration plots.

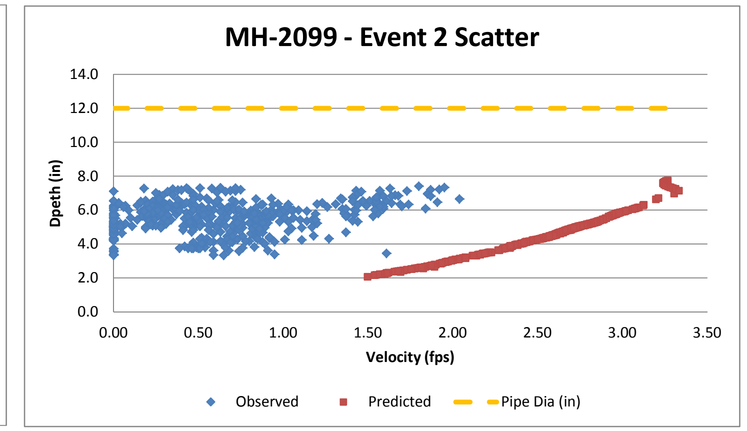

MH-2099 - Event 1 Flow
Flow (mgd)
Rainfall (in)
Observed
Predicted
Rainfall

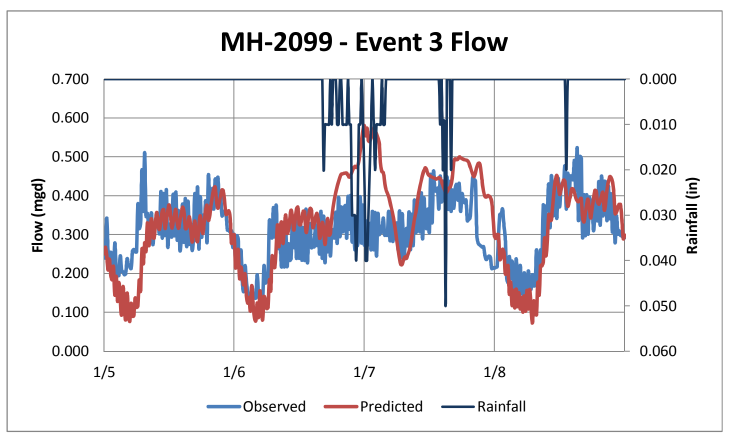

MH-2099 - Event 1 Depth
Depth (in)
Rainfall (in)
Pipe Dia (in)
Observed
Predicted
Rainfall

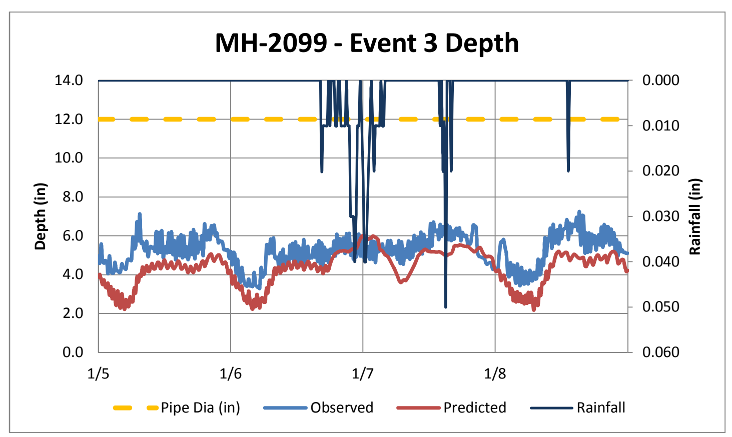

MH-2099 - Event 1 Scatter
Depth (in)
Velocity (fps)
Observed
Predicted
Pipe Dia (in)

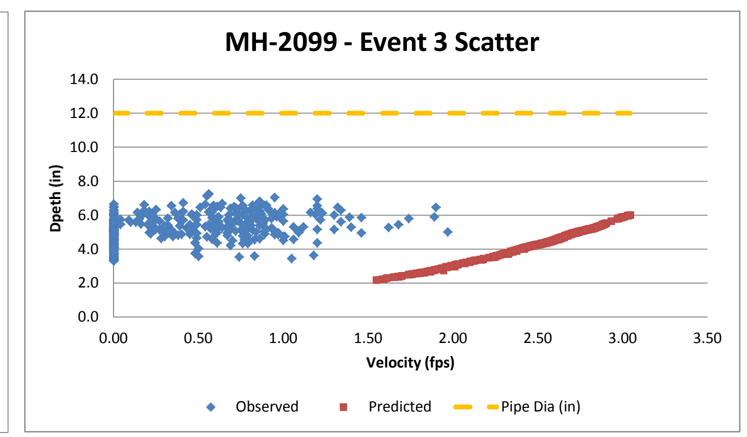

MH-2099 - Event 2 Flow
Flow (mgd)
Rainfall (in)
Observed
Predicted
Rainfall

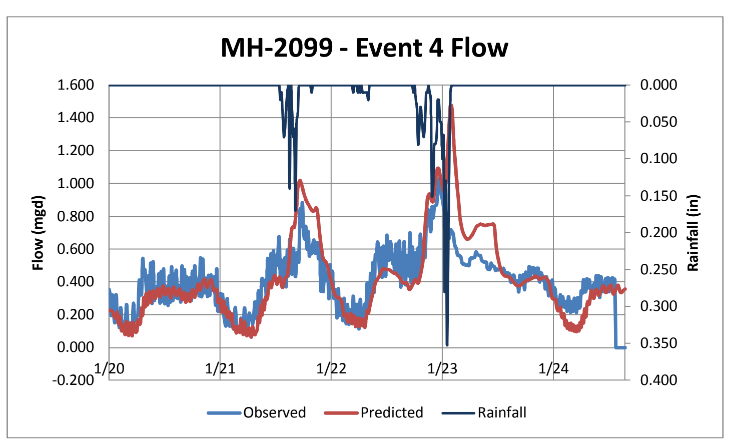

MH-2099 - Event 2 Depth
Depth (in)
Rainfall (in)
Pipe Dia (in)
Observed
Predicted
Rainfall

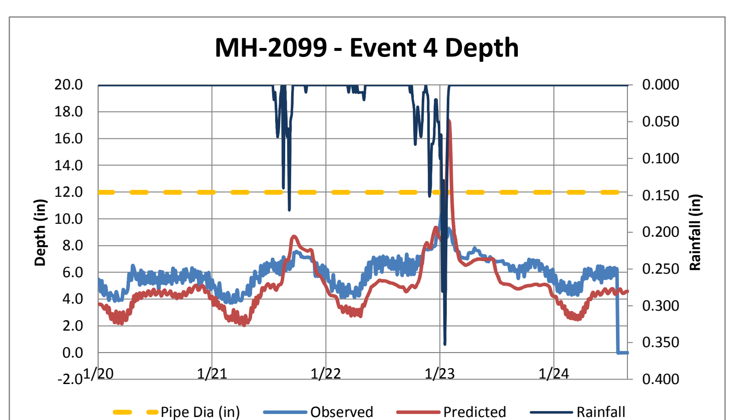

MH-2099 - Event 2 Scatter
Depth (in)
Velocity (fps)
Observed
Predicted
Pipe Dia (in)

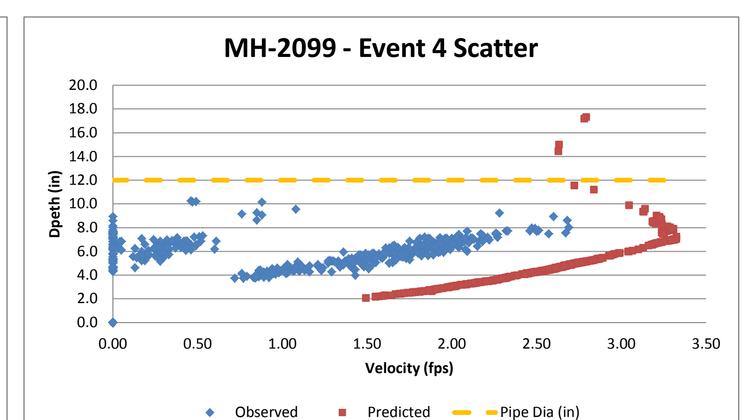

MH-2099 - Event 3 Flow
Flow (mgd)
Rainfall (in)
Observed
Predicted
Rainfall

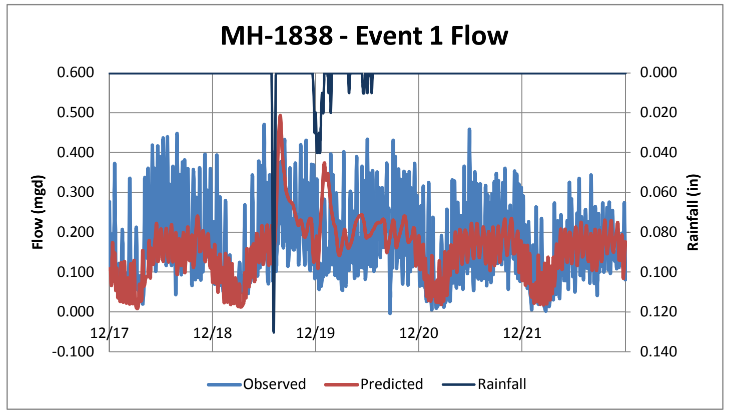

MH-2099 - Event 3 Depth
Depth (in)
Rainfall (in)
Pipe Dia (in)
Observed
Predicted
Rainfall

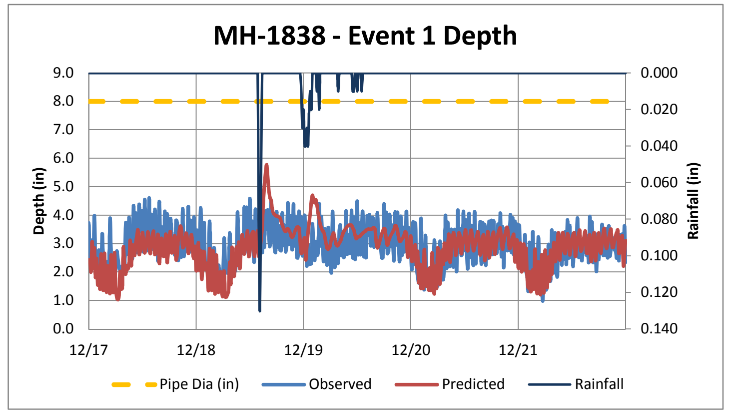

MH-2099 - Event 3 Scatter
Depth (in)
Velocity (fps)
Observed
Predicted
Pipe Dia (in)

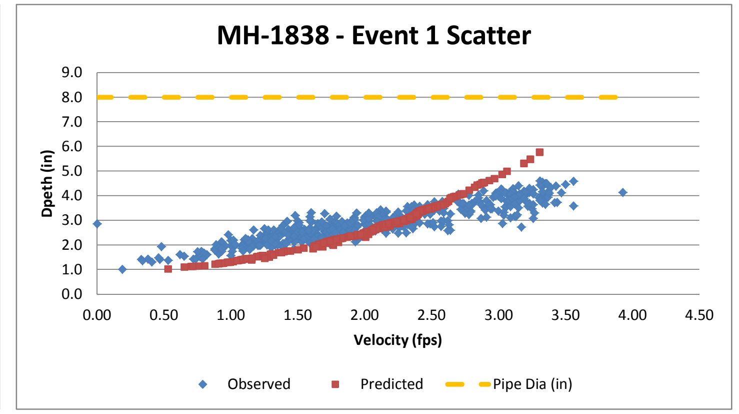

MH-2099 - Event 4 Flow
Flow (mgd)
Rainfall (in)
Observed
Predicted
Rainfall

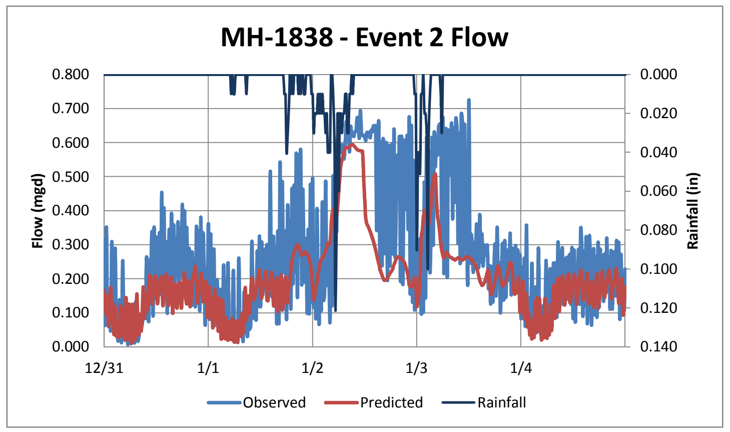

MH-2099 - Event 4 Depth
Depth (in)
Rainfall (in)
Pipe Dia (in)
Observed
Predicted
Rainfall

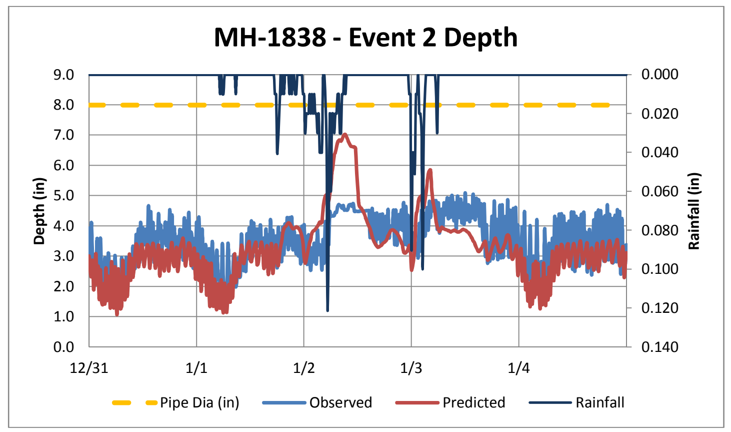

MH-2099 - Event 4 Scatter
Depth (in)
Velocity (fps)
Observed
Predicted
Pipe Dia (in)

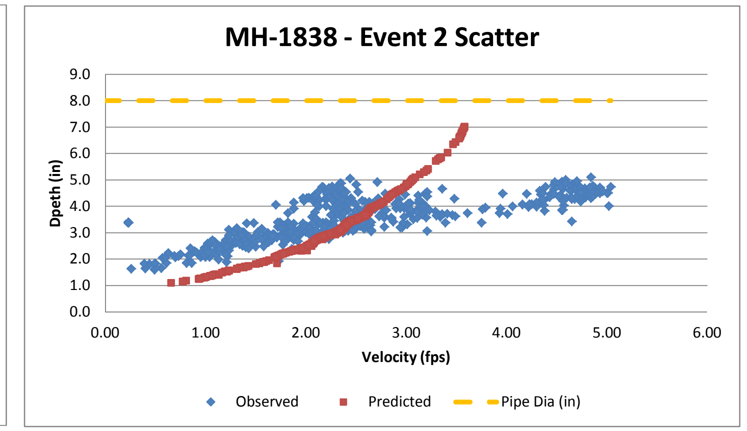

MH-1838 - Event 1 Flow
Flow (mgd)
Rainfall (in)
Observed
Predicted
Rainfall

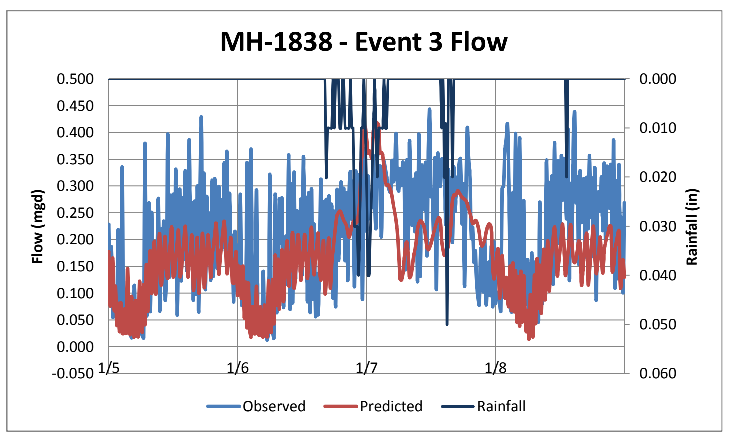

MH-1838 - Event 1 Depth
Depth (in)
Rainfall (in)
Pipe Dia (in)
Observed
Predicted
Rainfall

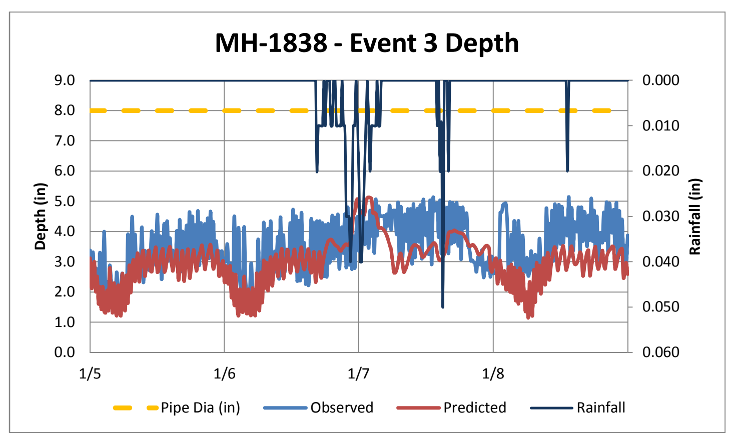

MH-1838 - Event 1 Scatter
Depth (in)
Velocity (fps)
Observed
Predicted
Pipe Dia (in)

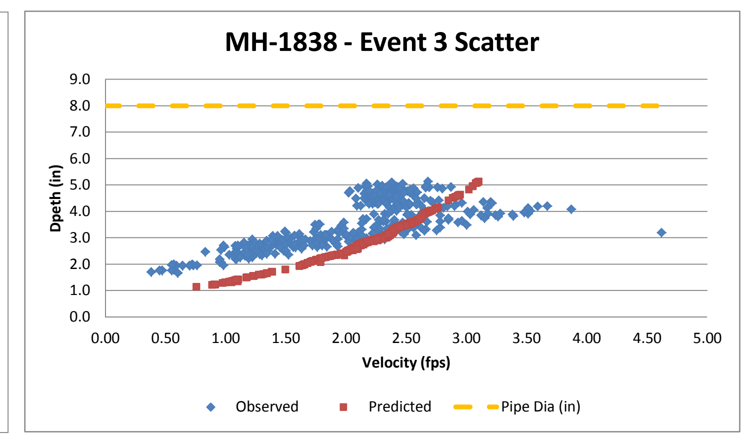

MH-1838 - Event 2 Flow
Flow (mgd)
Rainfall (in)
Observed
Predicted
Rainfall

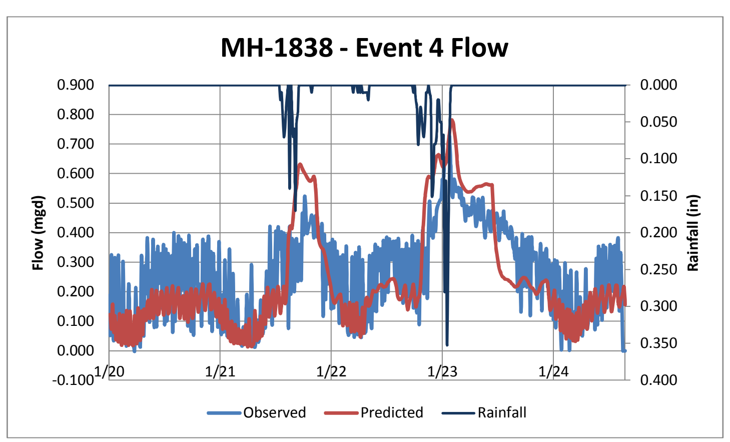

MH-1838 - Event 2 Depth
Depth (in)
Rainfall (in)
Pipe Dia (in)
Observed
Predicted
Rainfall

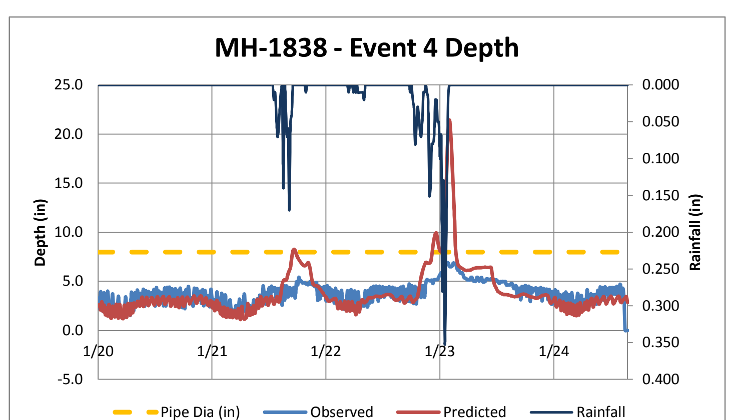

MH-1838 - Event 2 Scatter
Depth (in)
Velocity (fps)
Observed
Predicted
Pipe Dia (in)

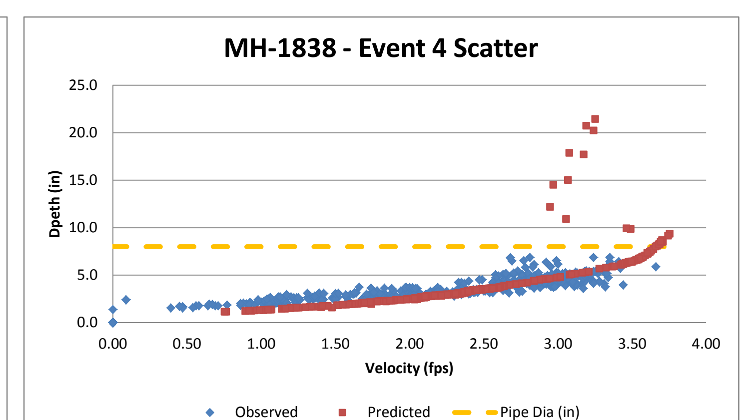

MH-1838 - Event 3 Flow
Flow (mgd)
Rainfall (in)
Observed
Predicted
Rainfall

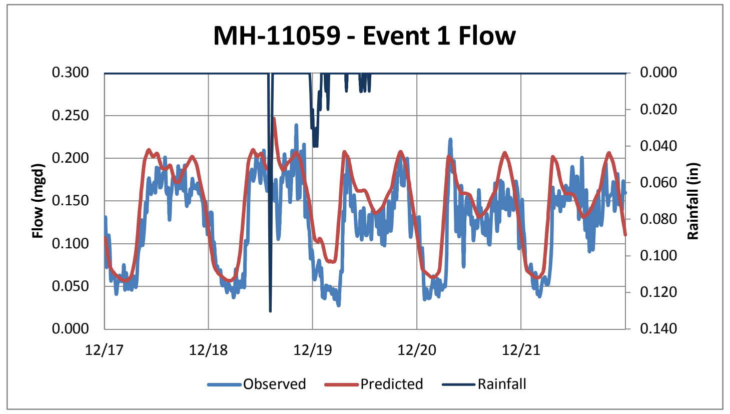

MH-1838 - Event 3 Depth
Depth (in)
Rainfall (in)
Pipe Dia (in)
Observed
Predicted
Rainfall

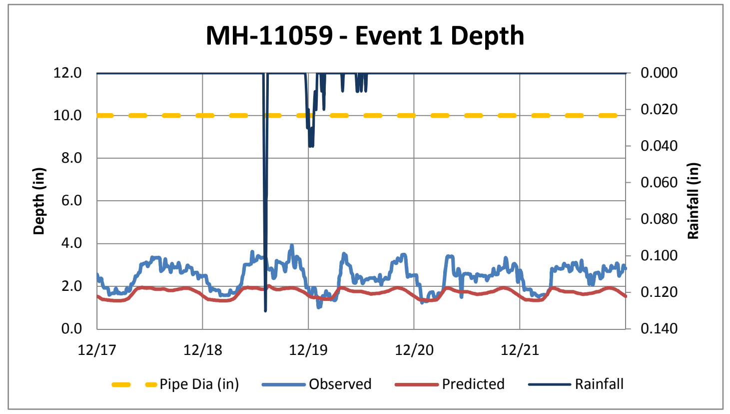

MH-1838 - Event 3 Scatter
Depth (in)
Velocity (fps)
Observed
Predicted
Pipe Dia (in)

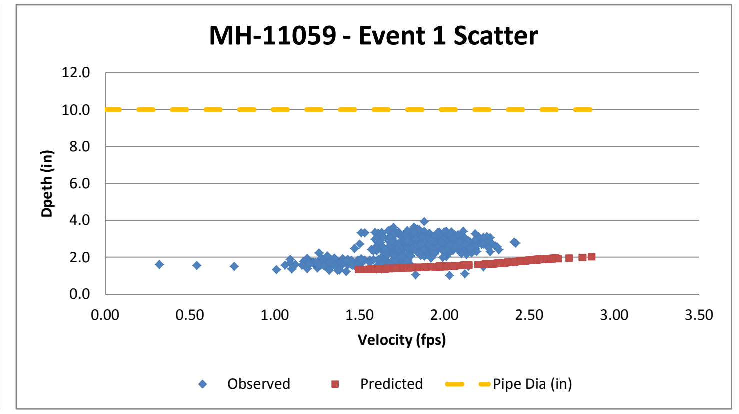

MH-1838 - Event 4 Flow
Flow (mgd)
Rainfall (in)
Observed
Predicted
Rainfall

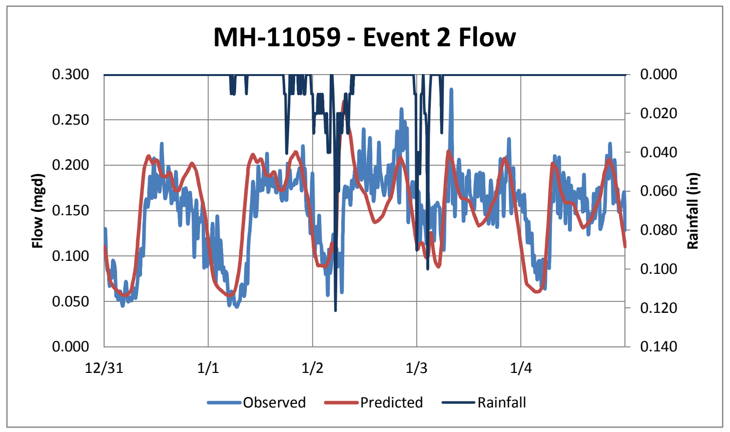

MH-1838 - Event 4 Depth
Depth (in)
Rainfall (in)
Pipe Dia (in)
Observed
Predicted
Rainfall

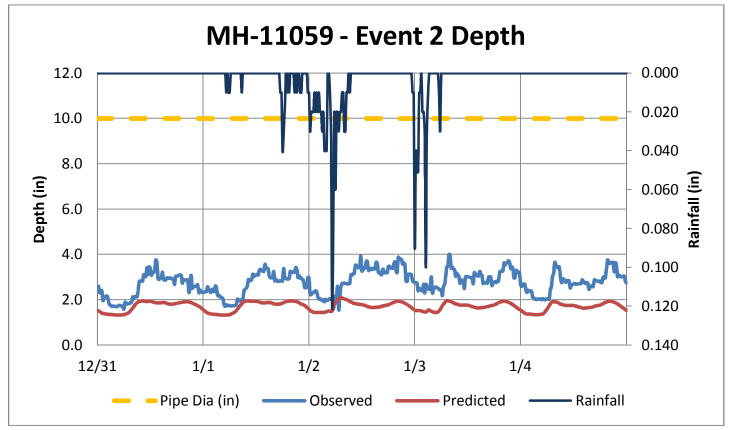

MH-1838 - Event 4 Scatter
Depth (in)
Velocity (fps)
Observed
Predicted
Pipe Dia (in)

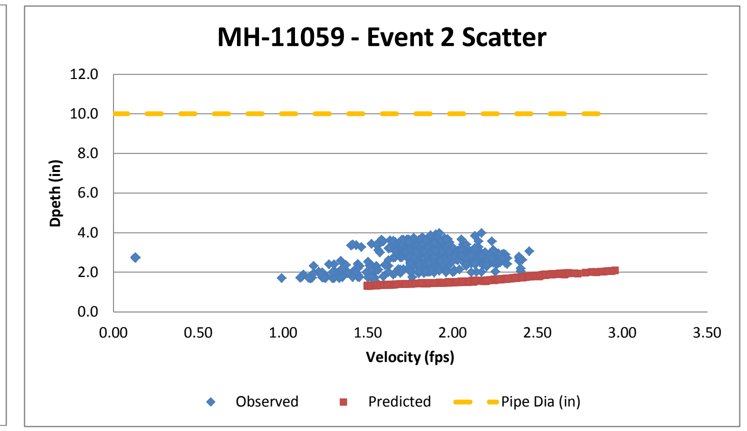

MH-11059 - Event 1 Flow
Flow (mgd)
Rainfall (in)
0.300
0.250
0.200
0.150
0.100
0.050
0.000
0.000
0.020
0.040
0.060
0.080
0.100
0.120
0.140
12/17
12/18
12/19
12/20
12/21
Observed
Predicted
Rainfall

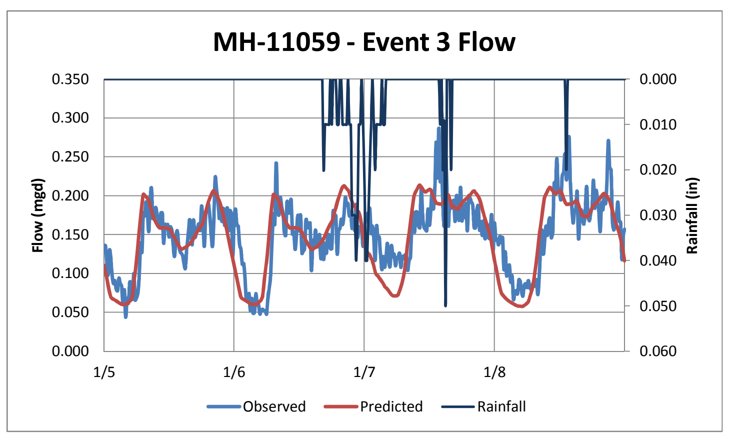

MH-11059 - Event 1 Depth
Depth (in)
Rainfall (in)
12.0
10.0
8.0
6.0
4.0
2.0
0.0
0.000
0.020
0.040
0.060
0.080
0.100
0.120
0.140
12/17
12/18
12/19
12/20
12/21
Pipe Dia (in)
Observed
Predicted
Rainfall

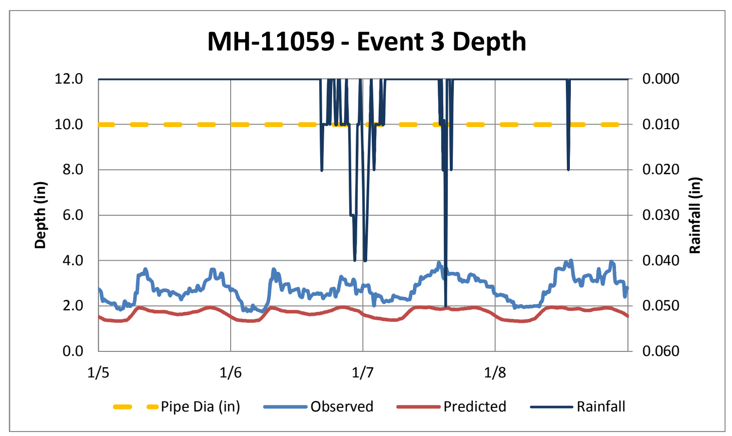

MH-11059 - Event 1 Scatter
Dpeth (in)
Velocity (fps)
12.0
10.0
8.0
6.0
4.0
2.0
0.0
0.00
0.50
1.00
1.50
2.00
2.50
3.00
3.50
Observed
Predicted
Pipe Dia (in)

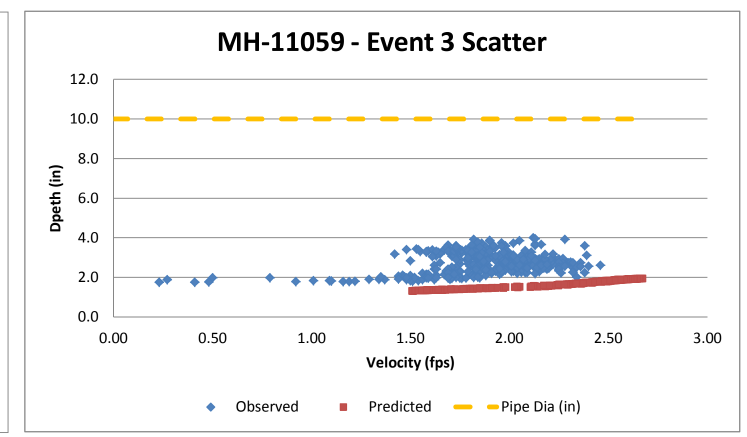

MH-11059 - Event 2 Flow
Flow (mgd)
Rainfall (in)
0.300
0.250
0.200
0.150
0.100
0.050
0.000
0.000
0.020
0.040
0.060
0.080
0.100
0.120
0.140
12/31
1/1
1/2
1/3
1/4
Observed
Predicted
Rainfall

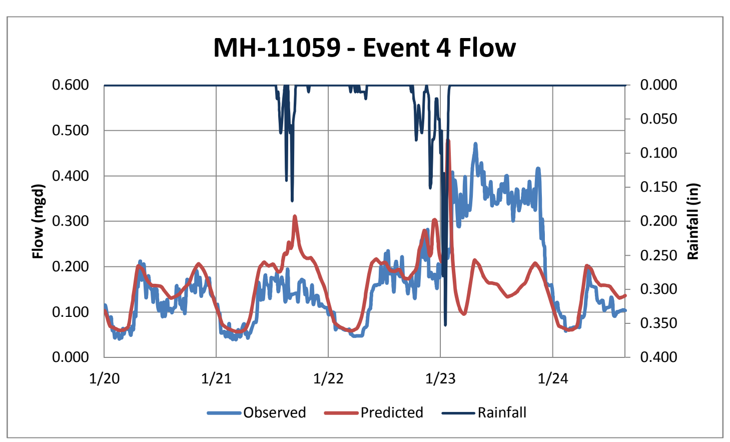

MH-11059 - Event 2 Depth
Depth (in)
Rainfall (in)
12.0
10.0
8.0
6.0
4.0
2.0
0.0
0.000
0.020
0.040
0.060
0.080
0.100
0.120
0.140
12/31
1/1
1/2
1/3
1/4
Pipe Dia (in)
Observed
Predicted
Rainfall

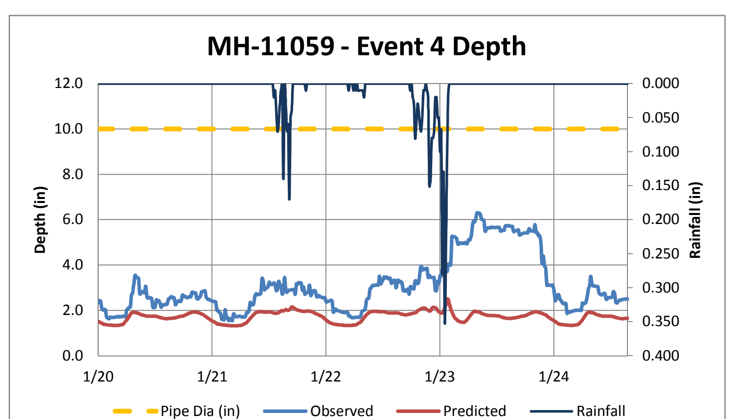

MH-11059 - Event 2 Scatter
Dpeth (in)
Velocity (fps)
12.0
10.0
8.0
6.0
4.0
2.0
0.0
0.00
0.50
1.00
1.50
2.00
2.50
3.00
3.50
Observed
Predicted
Pipe Dia (in)

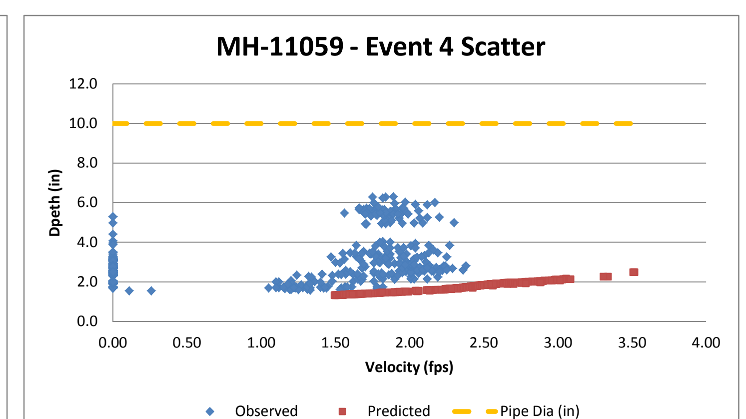

MH-11059 - Event 3 Flow
Flow (mgd)
Rainfall (in)
0.350
0.300
0.250
0.200
0.150
0.100
0.050
0.000
0.000
0.010
0.020
0.030
0.040
0.050
0.060
1/5
1/6
1/7
1/8
Observed
Predicted
Rainfall

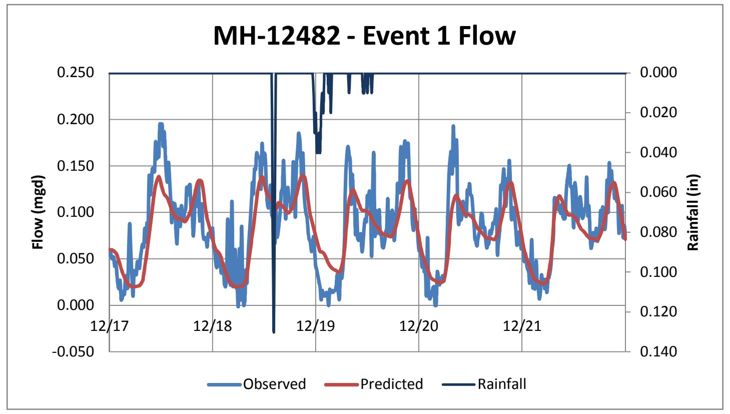

MH-11059 - Event 3 Depth
Depth (in)
Rainfall (in)
12.0
10.0
8.0
6.0
4.0
2.0
0.0
0.000
0.010
0.020
0.030
0.040
0.050
0.060
1/5
1/6
1/7
1/8
Pipe Dia (in)
Observed
Predicted
Rainfall

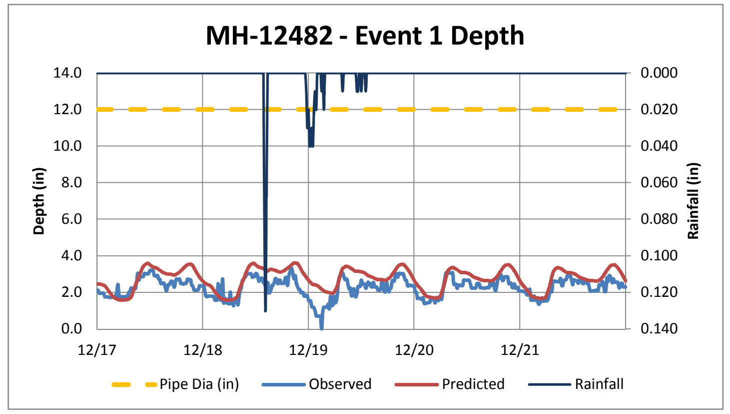

MH-11059 - Event 3 Scatter
Dpeth (in)
Velocity (fps)
12.0
10.0
8.0
6.0
4.0
2.0
0.0
0.00
0.50
1.00
1.50
2.00
2.50
3.00
Observed
Predicted
Pipe Dia (in)

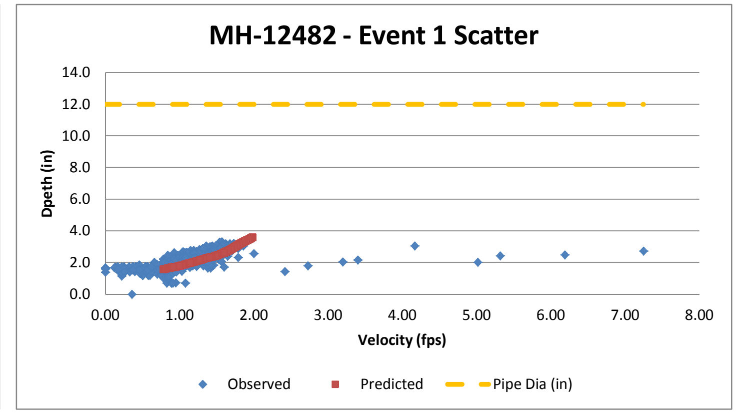

MH-11059 - Event 4 Flow
Flow (mgd)
Rainfall (in)
0.600
0.500
0.400
0.300
0.200
0.100
0.000
0.000
0.050
0.100
0.150
0.200
0.250
0.300
0.350
0.400
1/20
1/21
1/22
1/23
1/24
Observed
Predicted
Rainfall

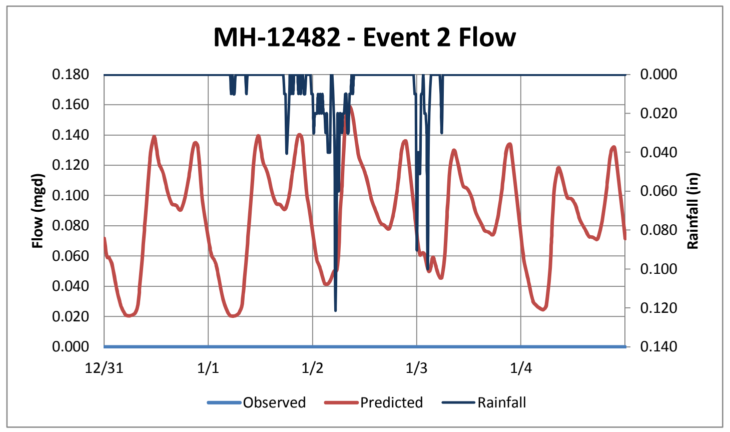

MH-11059 - Event 4 Depth
Depth (in)
Rainfall (in)
12.0
10.0
8.0
6.0
4.0
2.0
0.0
0.000
0.050
0.100
0.150
0.200
0.250
0.300
0.350
0.400
1/20
1/21
1/22
1/23
1/24
Pipe Dia (in)
Observed
Predicted
Rainfall

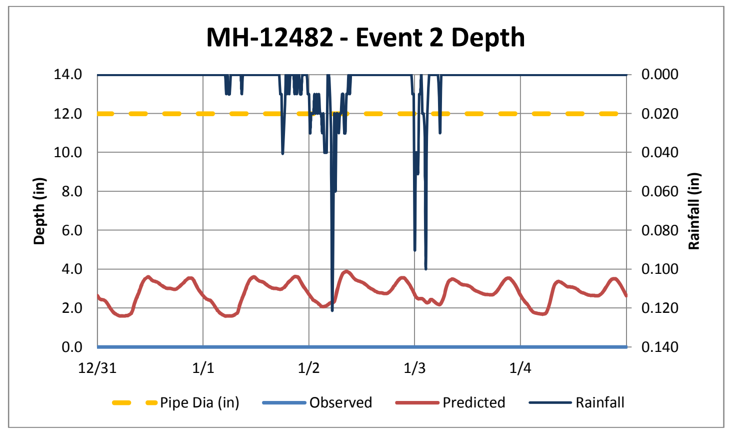

MH-11059 - Event 4 Scatter
Dpeth (in)
Velocity (fps)
12.0
10.0
8.0
6.0
4.0
2.0
0.0
0.00
0.50
1.00
1.50
2.00
2.50
3.00
3.50
4.00
Observed
Predicted
Pipe Dia (in)

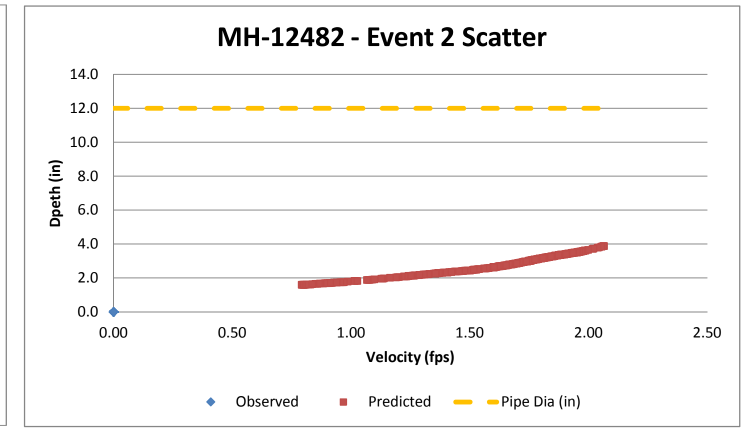

MH-12482 - Event 1 Flow
Flow (mgd)
Rainfall (in)
Observed
Predicted
Rainfall

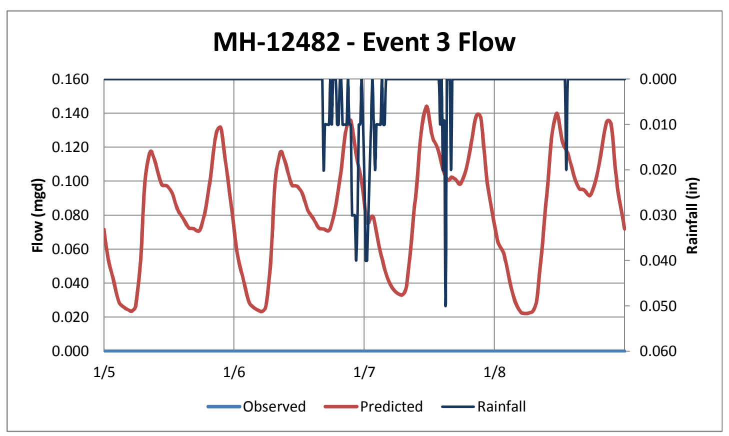

MH-12482 - Event 1 Depth
Depth (in)
Rainfall (in)
Pipe Dia (in)
Observed
Predicted
Rainfall

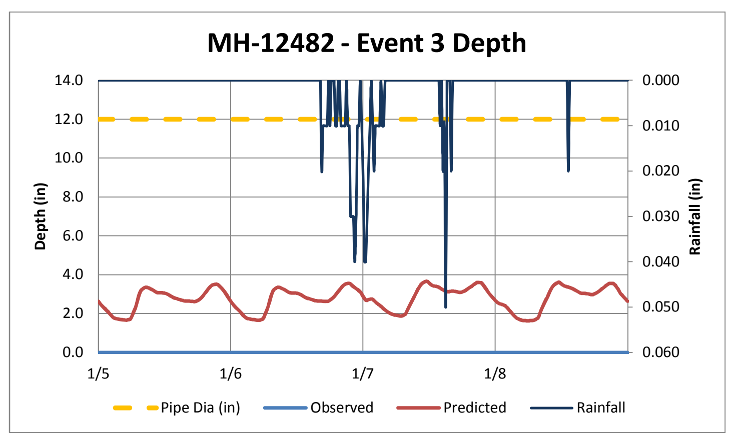

MH-12482 - Event 1 Scatter
Depth (in)
Velocity (fps)
Observed
Predicted
Pipe Dia (in)

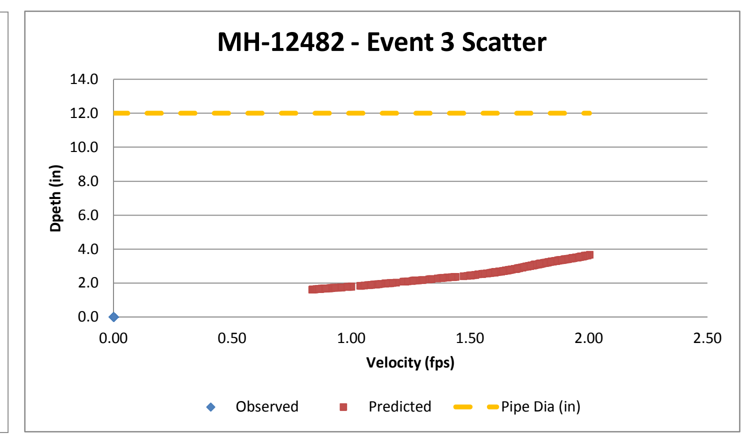

MH-12482 - Event 2 Flow
Flow (mgd)
Rainfall (in)
Observed
Predicted
Rainfall

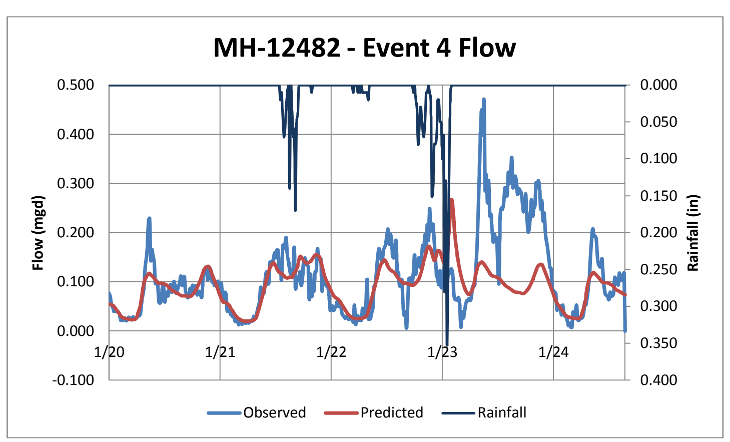

MH-12482 - Event 2 Depth
Depth (in)
Rainfall (in)
Pipe Dia (in)
Observed
Predicted
Rainfall

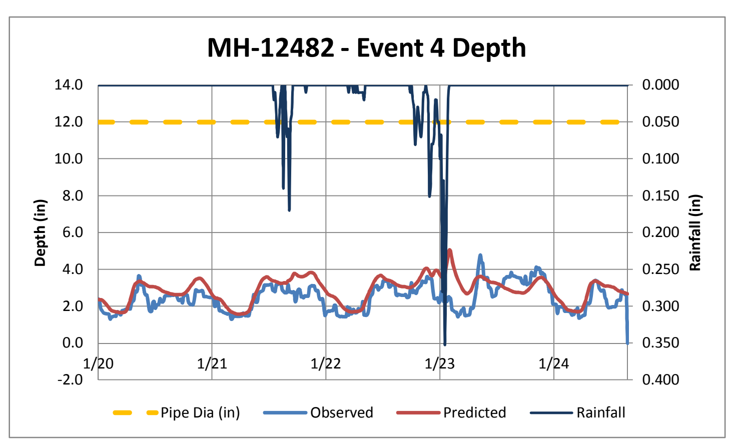

MH-12482 - Event 2 Scatter
Depth (in)
Velocity (fps)
Observed
Predicted
Pipe Dia (in)

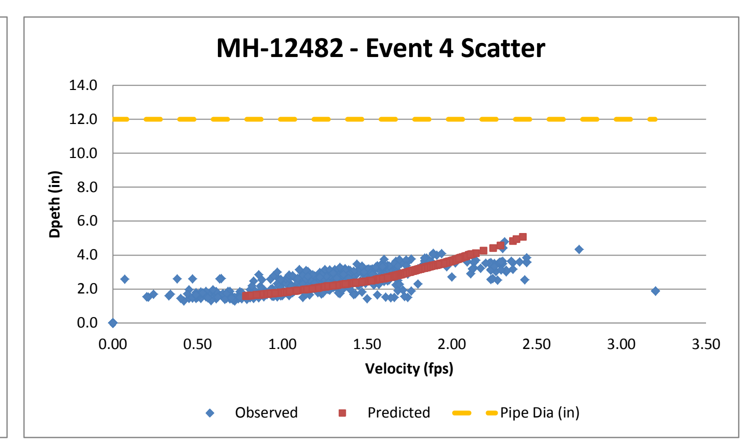

MH-12482 - Event 3 Flow
Flow (mgd)
Rainfall (in)
Observed
Predicted
Rainfall


MH-12482 - Event 3 Depth
Depth (in)
Rainfall (in)
Pipe Dia (in)
Observed
Predicted
Rainfall


MH-12482 - Event 3 Scatter
Depth (in)
Velocity (fps)
Observed
Predicted
Pipe Dia (in)


MH-12482 - Event 4 Flow
Flow (mgd)
Rainfall (in)
Observed
Predicted
Rainfall


MH-12482 - Event 4 Depth
Depth (in)
Rainfall (in)
Pipe Dia (in)
Observed
Predicted
Rainfall


MH-12482 - Event 4 Scatter
Depth (in)
Velocity (fps)
Observed
Predicted
Pipe Dia (in)