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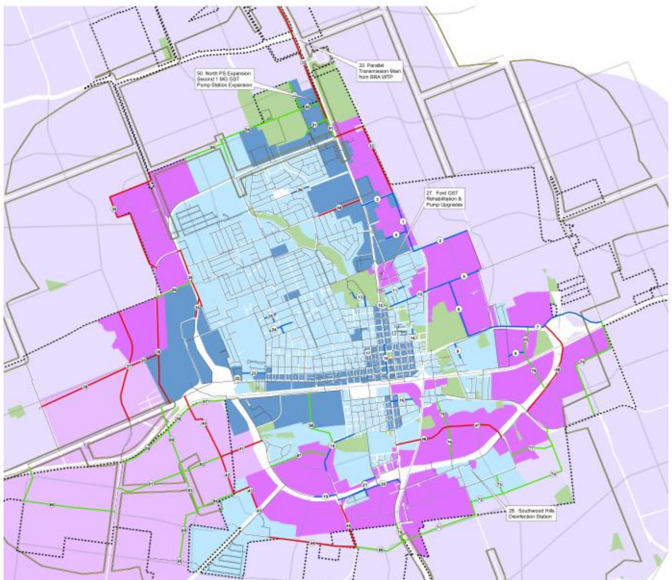

# City of Taylor Water and Wastewater Master Plan

*Taylor, Texas*
March 16, 2023

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# 1 Population and Land Use Projections

Population and land use projections provide the basis on planning future infrastructure needs. The magnitude and location of growth and specific development patterns can be hard to predict, especially beyond five years. The City of Taylor recently completed a comprehensive plan that included a projected 2040 population of 39,552. Also developed as part of the comprehensive plan were growth sectors and a future land use map. The vision of Taylor's growth plan is to encourage infill growth and increase population density within the City's existing service area.

## 1.1 Growth Sectors

The growth sectors identified in the Comprehensive Plan are shown in Figure 1-1, and include the categories listed below. These categories and descriptions are based on the currently adopted Comprehensive Plan, and are subject to change as the Comprehensive Plan is updated or amended.

- Preserved and reserved open space (such as parks and floodplains)
- Restricted growth (where increased development is discouraged)
- Controlled growth (where limited development can be supported)
- Intended growth (where development is encouraged and infrastructure exists to support development)
- Infill growth (increasing density in existing areas)

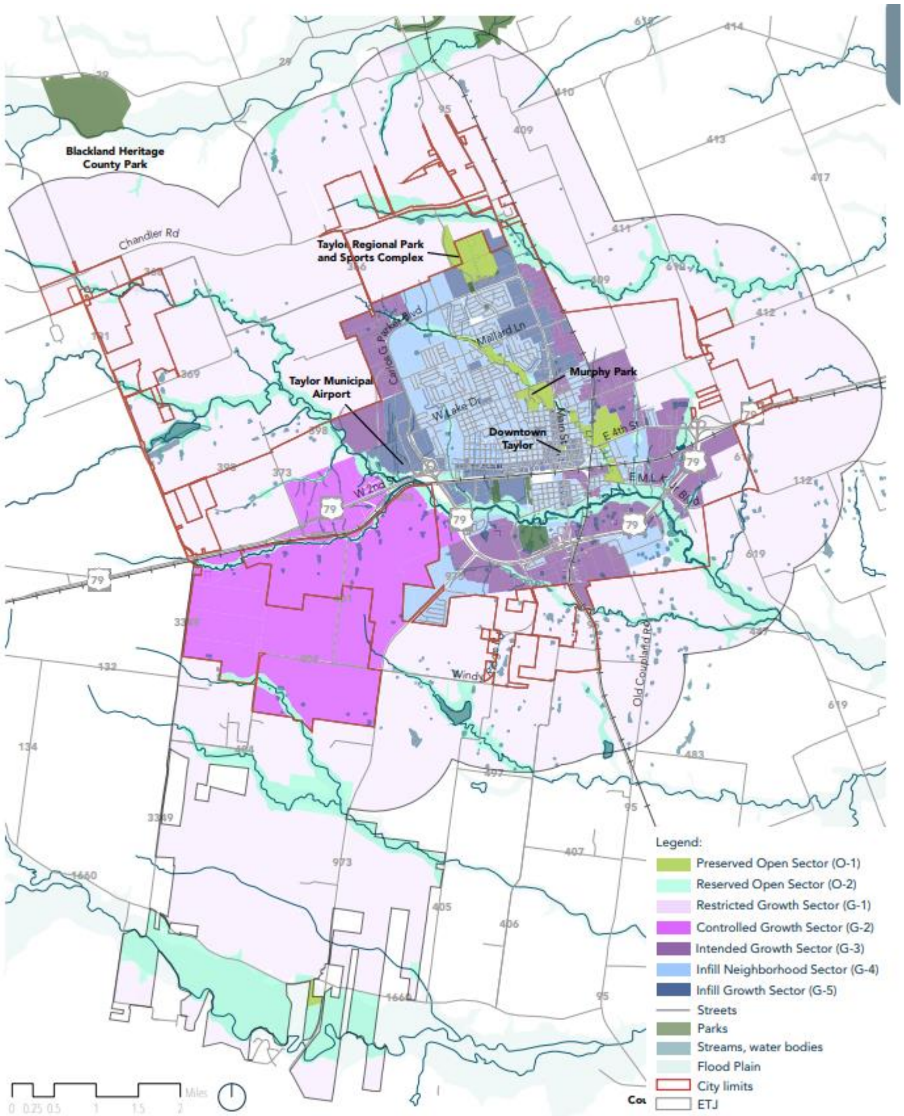



**Figure 1-1. Comprehensive Plan Growth Sectors**

## 1.2 Future Land Use and Density Assumptions

The future service area for this water and wastewater master plan was based on serving the 2040 controlled, intended and infill growth sectors. The 2040 population of 39,552 was spatially allocated to the future service area based on the comprehensive plan future land use map. The comprehensive plan future land use map is shown in Figure 1-2 and includes the following categories:

- Neighborhood infill (increasing density in existing developed areas)
- Neighborhood greenfield (new developments outside of existing developed areas)
- Employment Centers (office and industrial areas that support employment)
- Market Centers (mixed use including retail and multi-family residential)

- Civic Centers (civic destinations such as schools and libraries)

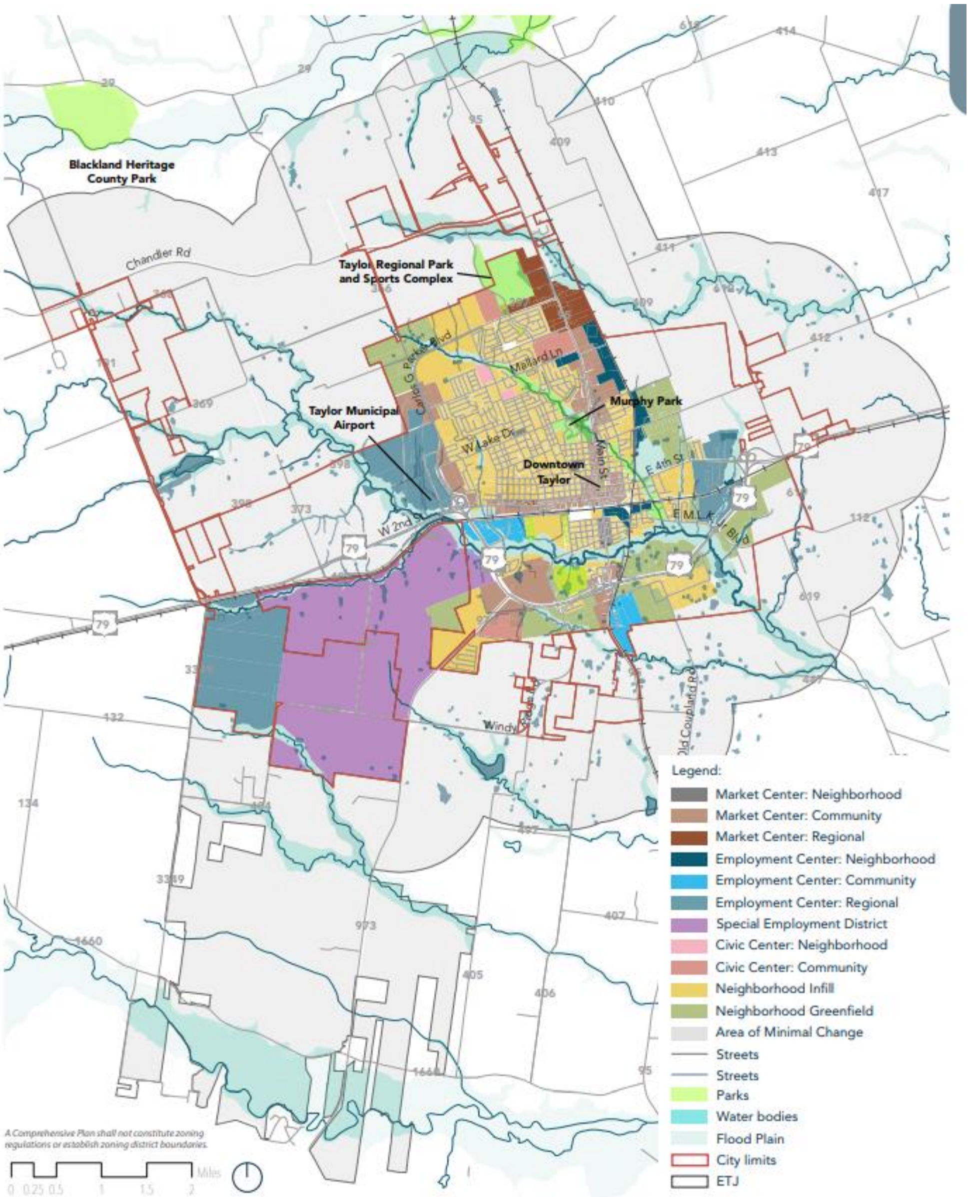



**Figure 1-2. Comprehensive Plan Future Land Use Plan**

In coordination with the comprehensive plan team, the following residential population densities were applied to the future land use categories:

- Neighborhood infill:  6.75 people per acre
- Neighborhood greenfield:  13 people per acre
- Employment Centers:  0 people per acre
  - Non-residential water demands and wastewater flows were accounted for, based on an assumption of 9 LUEs/acre and 1 LUE/2000 SF of developed space

- Market Centers:  3 people per acre
    - This equates to a multi-family residential population density of 12 people per acre, applied to 25% of the market center acreage
    - Non-residential water demands and wastewater flows were accounted for, based on an assumption of 9 LUEs/acre and 1 LUE/2000 SF of developed space for the remaining 75% of the market center acreage
- Civic Centers:  0 people per acre
    - Non-residential water demands and wastewater flows were accounted for, based on an assumption of 4.5 LUE/acre

### 1.2.1 Existing Service Area

In some portions of the existing service area, the 2040 population calculated based on these density assumptions yielded a population lower than the existing population estimated by water billing data and GIS data of residential and non-residential building footprints.  In these cases, the 2040 population was set to be equal to the current population in areas that appeared to be built-out.  This yields a 2040 population in the existing service area of approximately 28,500 people.

### 1.2.2 Future Service Area

The remaining 11,000 people to reach the 2040 population of 39,552 was distributed to the intended and controlled growth sectors.  The 2040 residential population densities assumed for these growth sectors were 13 people per acre; however, this exceeded the total 2040 population of 39,552.  A portion of these areas were reduced to a 2040 density of 10.5 people per acre.  This is not necessarily the ultimate, build-out population density. As these properties develop and homes are constructed and occupied, the density may increase after 2040.

# 2 Water System

Taylor's water distribution system consists of two pressure planes, a network of water lines, two pump stations, two ground storage tanks, four elevated storage tanks, and three pressure reducing valves (PRVs).

## 2.1 Water Supply

Taylor receives treated surface water from the Brazos River Authority (BRA), through a 27-inch transmission main from the East Williamson County Regional Water System (EWCRWS) water treatment plant (WTP) at Granger Lake.  The WTP includes a 2-million-gallon (MG) clearwell where treated water is stored.  Water from the clearwell is pumped through a high service pump station that includes four vertical turbine pumps, each with a rated capacity of 3,650 GPM at 280 feet of total dynamic head.  The pump station firm capacity (with one pump out of service) is 10,950 GPM, or 15.77 MGD.  The BRA high service pump operation is controlled based on the water level of the City's Mallard Elevated Storage Tank (EST), with the Murphy EST as a backup.

There is currently no redundancy in the single 27-inch water transmission main. BRA is working on a project to expand the capacity of the WTP and install a second transmission main along FM 1331, from the WTP and North Highway 95. This second transmission main will be dedicated to serve BRA's other customers, leaving the existing transmission main dedicated to Taylor.

### 2.1.1 Wholesale Water Customers

Taylor currently provides pass-through wholesale water to the City of Hutto, the City of Thrall, and Manville Water Supply Corporation (WSC). Taylor's 16-inch water main along Highway 79 fills a 0.5 MG ground storage tank owned by the City of Hutto, located just south of Highway 79 along FM 3349. Taylor has a contractual commitment to provide the City of Hutto with 0.3 MGD of water. Between 2019 and 2021, the actual water sold per year for each of these wholesale water customers is as follows:

**Table 2-1. Wholesale Water Customer Historical Consumption**

| Tank | 2019 (MGD) | 2020 (MGD) | 2021 (MGD) |
|---|---|---|---|
| City of Hutto | 0.24 | 0.23 | 0.19 |
| City of Thrall | 0 | 0.057 | 0.066 |
| Manville WSC[1] | 0 | .025 | 0 |

1. Formerly Noack Water Service Company.

## 2.2 Water Distribution System

The water from BRA enters directly into Taylor's lower pressure plane distribution system, and also fills the North ground storage tank. From there, the North pump station provides water to Taylor's upper pressure plane through a 24-inch diameter line along Carlos Parker, between the North pump station and the West elevated storage tank. The North pump station pump operation is based on the water level of the West tank.

Taylor's lower pressure plane is controlled by the water level of the Murphy, Mallard, and Southwood Hills ESTs, which each have an overflow water surface elevation of approximately 709 feet. Low demands in the south and southeast portion of the City create low chlorine residuals. As a temporary measure, the Southwood Hills EST was taken offline to reduce the volume of storage in the system, to reduce the age of the water distributed in this part of the city.

The pressure planes are separated by a series of closed isolation valves, as well as three PRVs. The PRVs are not oriented to allow water from the higher pressure plane to enter the lower pressure plane. Instead, the PRVs are oriented to allow water from the lower pressure plane to enter the upper pressure plane. In the event of a substantial decrease in pressure in the upper pressure plane, these PRVs would allow the entire system to be served from the BRA high service pumps. The Ford pump station is not automatically controlled, but is manually operated when necessary to circulate water in the southeast portion of the City.

Figure 2-1 illustrates a schematic of the existing system, and Figure 2-2 displays the existing distribution system and pressure plane boundary.

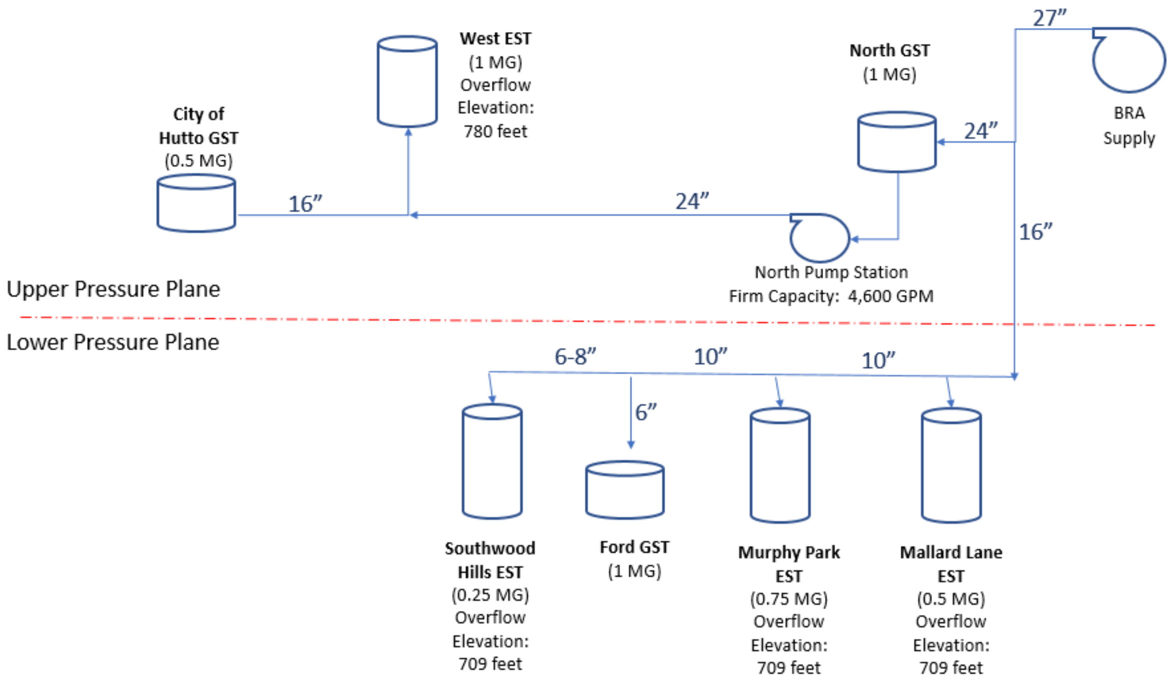



**Figure 2-1. Existing Water Distribution System Schematic**

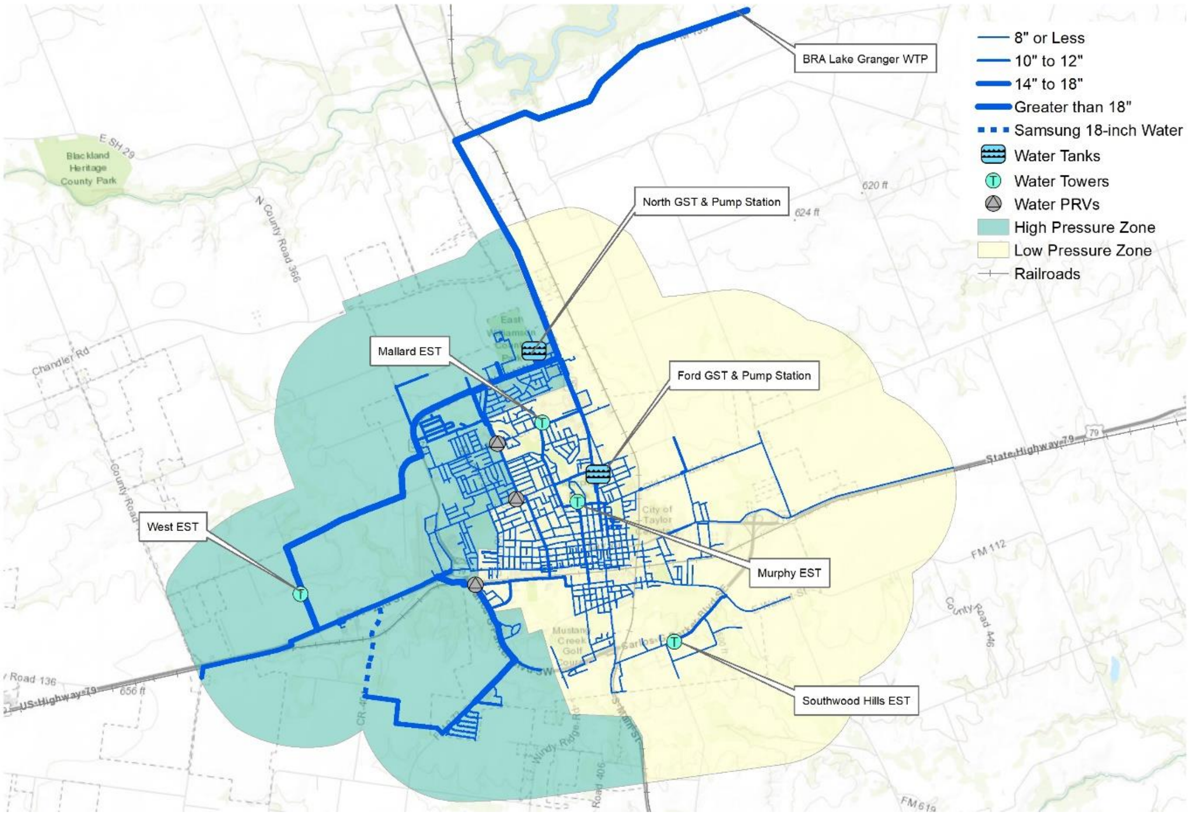



**Figure 2-2. Existing Water Distribution System Map**

# 2.3 Existing Demand Evaluation

Taylor's population in the 2020 census was 16,267. Taylor's current population is estimated at 18,000, assuming an average annual growth rate of approximately five percent since 2020. In 2021, Taylor purchased 2.62 MGD from BRA. Of that, 0.26 MGD was provided to Hutto, Thrall and Noack WSC, leaving 2.37 MGD for Taylor. Taylor water billing data indicates that in 2021, Taylor's customers consumed 1.65 MGD. Taylor's aging infrastructure is causing a high rate of water loss. Including both domestic consumption and losses, the average water consumption in Taylor in 2021 was 138 gallons/person/day. However, it should be noted that 2021 included winter storm Uri, which increased water loss. 2020 and 2019 data indicates a more typical value for Taylor is 120 gallons/person/day. Based on the current estimated population of 18,000, Taylor's current average day demand is estimated at approximately 2.2 MGD.

Historical daily and hourly demand data was not available. In these cases, TCEQ Chapter 290 requires the following peaking factors:

- Maximum day demand = 2.4 times average day demand
- Peak hour demand = 1.25 times maximum day demand (when minimum elevated storage requirements of 100 gallons per connection are met)

These peaking factors yield the following demands:

- Maximum day demand: 5.3 MGD
- Peak hour demand: 6.6 MGD (4,580 GPM)

### 2.3.1 Storage Capacity

Taylor's distribution system includes 2.5 million gallons (MG) of elevated storage and 2 MG of ground storage, as shown in Table 2-2.

**Table 2-2. Storage Volume Summary**

| Pressure Plane | Tank | Volume (MG) |
|---|---|---|
| Upper | North (Ground Storage) | 1.00 |
| | West (Elevated Storage) | 1.00 |
| Lower | Mallard (Elevated Storage) | 0.50 |
| | Murphy (Elevated Storage) | 0.75 |
| | Southwood Hills (Elevated Storage) | 0.25 |
| | Ford (Ground Storage) | 1.00 |

### 2.3.2 Booster Pumping Capacity

Taylor's distribution system includes two pump stations:

- The North pump station, which includes three pumps with a rated capacity of 2,300 gallons per minute (GPM) each.
    - A current project is underway to add two additional pumps, each with a capacity of approximately 2,300 GPM.
- The Ford pump station. The rated capacity of these pumps is unknown.

## 2.4 Existing System Analysis

The existing system was evaluated in terms of elevated and ground storage, and booster pumping capacity. The hydraulic model was utilized to evaluate existing pipe velocities, system pressures, and available fire flow.

### 2.4.1 Elevated Storage

It is assumed that peak hour demands are met by a combination of both booster pumping and elevated storage. Elevated storage is generally considered to include equalization volume that can meet peak hour demands in addition to booster pumping capacity, as well as provide emergency water for emergency fire flow conditions.

Elevated storage volume was evaluated on a basis of meeting 50% of peak hour demands, plus emergency fire flow, for a duration of four hours and is summarized in Table 2-3.

**Table 2-3. Existing Elevated Storage Evaluation**

| Pressure Plane | Total Elevated Storage Volume (MG) | Peak Hour Volume (MG)[1] | Fire Flow Volume (MG)[2] | Recommended Elevated Storage (MG) | Elevated Storage Surplus/(Deficit) (MG) |
|---|---|---|---|---|---|
| Upper | 1.00 | 0.20 | 0.60 | 0.80 | 0.2 |
| Lower | 1.50 | 0.42 | 0.60 | 1.02 | 0.48 |

1. Half of peak hour demand for a duration of four hours.
2. Maximum fire flow assumed to be 2,500 GPM for a duration of four hours.

Taylor's existing elevated storage volume is adequate to provide half of peak hour demands plus emergency fire flow of 2,500 GPM for a duration of four hours. 2,500 GPM may not be sufficient fire flow for new large, specialized industrial customers. In these cases, if a new industrial development requires additional fire flow, additional onsite fire flow storage and pumping may be required.

### 2.4.2 Ground Storage

Ground storage was evaluated on a basis of providing 8-12 hours of maximum day demand.

**Table 2-4. Existing Ground Storage Evaluation**

| Pressure Plane | Total Ground Storage Volume (MG) | 8 hours of Maximum Day Demand (MG) | 12 hours of Maximum Day Demand (MG) | Recommended Ground Storage (MG) | Ground Storage Surplus/(Deficit) (MG) |
|---|---|---|---|---|---|
| Upper | 1.00 | 0.66 | 1.00 | 1.00 | 0.0 |
| Lower | 1.00[1] | 1.34 | 2.00 | 2.00 | (1.00) |

1. This volume does not include the clearwell storage at the BRA WTP which supplements the City's ground storage in the lower pressure plane.

For the upper pressure plane, Taylor's existing 1 MG of ground storage volume at the North ground storage tank site is adequate to provide 12 hours of current maximum day demands. As growth occurs in the upper pressure plane, the existing 1 MG North GST will provide less than 12 hours of maximum day demand. A future CIP project is recommended to add additional ground storage as growth occurs in the upper pressure plane.

For the lower pressure plane, the ground storage owned by the City of Taylor is 1.0 MG, which is a deficit of 1.0 MG below the recommended 2.0 MG of storage to hold 12 hours of maximum day demand in the lower pressure plane. However, because the water Taylor purchases from BRA enters directly into Taylor's lower pressure plane, the treated water storage volume at the BRA WTP counts towards the City's system storage for the lower pressure plane.

### 2.4.3 Booster Pumping

Booster pumping capacity was evaluated on a basis of meeting 50% of peak hour demand with the largest pump out of service. A current project is under way to expand the North pump station to a firm capacity of 9,200 GPM, largely to meet the projected peak Samsung demand from the City in late 2023 and early 2024.

**Table 2-5. Existing Booster Pumping Evaluation**

| Pressure Plane[1] | Firm Capacity (GPM) | 50% pf Peak Hour Demand (GPM) | Recommended Firm Pumping Capacity (GPM) |
|---|---|---|---|
| Upper | 4,600[2] | 860 | 860 |

1. Lower pressure plane booster pumping requirements are met by the BRA WTP high service pumps.
2. Existing firm capacity, does not include expansion project currently under construction.

### 2.4.4 TCEQ Requirements

The Texas Commission on Environmental Quality (TCEQ) has requirements for public drinking water systems described in Chapter 290, Public Drinking Water Systems. These requirements include:

- 100 gallons per connection of elevated storage
- 200 gallons per connection of total storage
- Booster pumping capacity of 0.6 GPM per connection (for systems that provide more than 200 gallons/connection of elevated storage)

The City's current connection count is approximately 6,338, and meets the TCEQ requirements for storage and booster pumping, as shown in Table 2-6.

**Table 2-6. TCEQ Requirements Summary**

| Pressure Plane | Estimated Connections | Description | Required by TCEQ | City of Taylor Facilities | Surplus/ (Deficit) |
|---|---|---|---|---|---|
| Upper | 2,092 | Elevated Storage (MG) | 0.21 | 1.00 | 0.79 |
| | | Total Storage (MG) | 0.42 | 2.00 | 1.58 |
| | | Booster Pumping (GPM) | 1,255 | 4,600[1] | 3,345 |
| Lower | 4,246 | Elevated Storage (MG) | 0.42 | 1.50 | 1.08 |
| | | Total Storage (MG) | 0.85 | 2.50 | 2.15 |
| | | Booster Pumping (GPM) | 2,548 | N/A[2] | N/A |

1. Current firm capacity of the North pump station.
2. Ford pump capacity is unknown. The BRA WTP high service pumps provide booster pumping capacity to the lower pressure plane.

### 2.4.5 Hydraulic Model Development

A hydraulic model of the distribution system was developed in the Innovyze InfoWater modeling software. A hydraulic model calculates predicted system pressures, velocities and headlosses anticipated during various system demands and conditions. These values are predicted based on model parameters including system demands, pump curves, elevations, and system headloss. Headloss is a function of the projected flow rate, pipe diameter, pipe length, and roughness factor.

#### Network Development

The pipeline network utilized for the hydraulic model network was developed from the City's geographic information system (GIS) geodatabase:

- Water Lines, including attributes of diameter, material, and date acquired (where available)
- City of Taylor ground and elevated storage tanks
- PRVs, including attributes of valve size

The pipeline network was imported into the model software, and model tools were used to create nodes at endpoints and intersections of pipelines. Limited information is available on pipe materials, installation date, or condition. Generally, it is known that Taylor's distribution system is older, and on average in fair condition. Therefore, conservative roughness values were assigned to pipes, with a Hazen-Williams roughness coefficient (C-value) ranging between 100 and 120.

A terrain model was created from contour data and used to assign an elevation to each modeled node.

Taylor staff provided pump curve information for the pumps at the North pump station. BRA provided pump curve information for the Lake Granger WTP high service pumps. As-built or design drawings for tanks, as well as an inspection report, were used to determine tank floor elevations, as well as minimum and maximum tank levels, as summarized in Table 2-7.

**Table 2-7. Storage Tank Geometry**

| Tank | Diameter (feet) | Floor Elevation (feet) | Overflow Elevation (feet) |
|---|---|---|---|
| North (Ground Storage) | 80 | 616.5 | N/A |
| West (Elevated Storage) | 72 | 674 | 780 |
| Mallard Lane (Elevated Storage) | 54 | 680 | 709 |
| Murphy Park (Elevated Storage) | 60 | 674 | 709 |
| Southwood Hills (Elevated Storage) | 38 | 680 | 709 |
| Ford (Ground Storage) | 70 | 577 | N/A |

## Demand Allocation

Demands were allocated spatially to the model based on metered monthly billing data. The addresses associated with the accounts were geocoded, to create coordinates based on a street address. A demand allocator tool in the modeling software spatially assigns each meter location to the nearest model node.

## Extended Period Simulation

A 24-hour extended period simulation (EPS) was developed, to adjust demands throughout an extended time period based on user-specified diurnal curves. An EPS model allows the model to predict how the system changes during a typical day, including system pressures, pipe velocities and headlosses, tank levels and pump status. Hourly demand data was not available, so industry standard typical curves were utilized, as shown in Figure 2-3 and Figure 2-4.

The hydraulic model was not calibrated; however, fire hydrant testing data provided by Taylor staff was used to validate the model results.

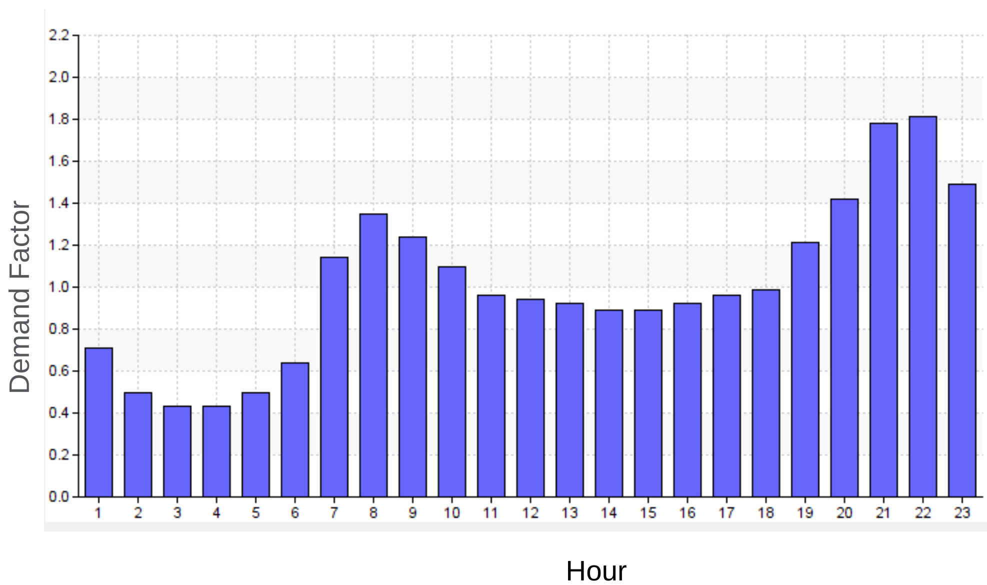



**Figure 2-3. Residential Diurnal Pattern**

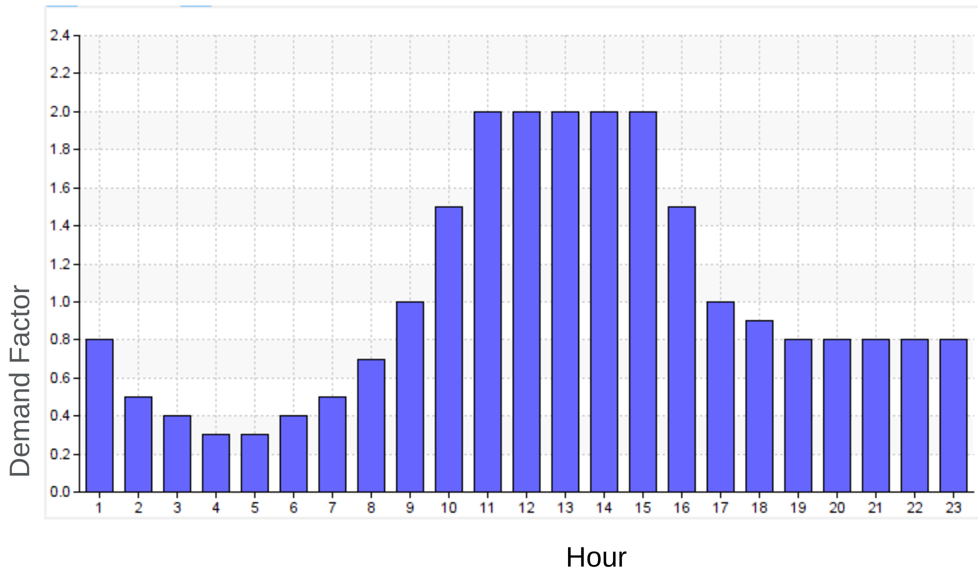



**Figure 2-4. Non-Residential Diurnal Pattern**

Base average day demands were multiplied by 2.4 to simulate maximum day demand. Theoretical peak hour demands are included as the maximum hourly demand occurring during a maximum day.

## Existing System Analysis

A hydraulic analysis was performed to evaluate system conditions during a maximum day demand scenario. Peak hour demands are represented by the time of day during a maximum day with the highest hourly peaking factor in the diurnal demand pattern. Peak hour demand represents the hour of the year with the highest overall system demand, and typically creates the lowest system pressures due to higher-than-normal headlosses.

Figure 2-5 shows the model-predicted pressure during peak hour demand conditions. TCEQ requires that water distribution systems maintain a pressure of 35 pounds per square inch (PSI). The upper pressure plane exceeds 35 PSI during peak hour demand, Areas of the lower pressure plane do drop below 35 PSI. Areas exhibiting the lowest system pressures are generally concentrated in the eastern portion of the system near Old Thorndale and FM 619, with additional lower pressures in areas at higher elevations of the lower pressure plane along North Drive, between Marshall Lane and Kent Street. The hydraulic model predicts this portion of the system is at 33.5 PSI or higher during peak hour demand. This variance of 1.5 PSI is considered within the anticipated realm of accuracy of the model.

It was noted through the EPS modeling that when the BRA high service pumps turn off, and the North GST is filled from hydraulic grade line in the lower pressure plane (determined by the overflow elevation of the Murphy or Mallard EST), the model predicts the area around the North GST is driven by the hydraulic grade line (HGL) of the North GST, causing the distribution system to experience a temporary, but significant, drop in pressure. A pressure sustaining valve at the North GST would prevent this drop in pressure when the tank is filling.

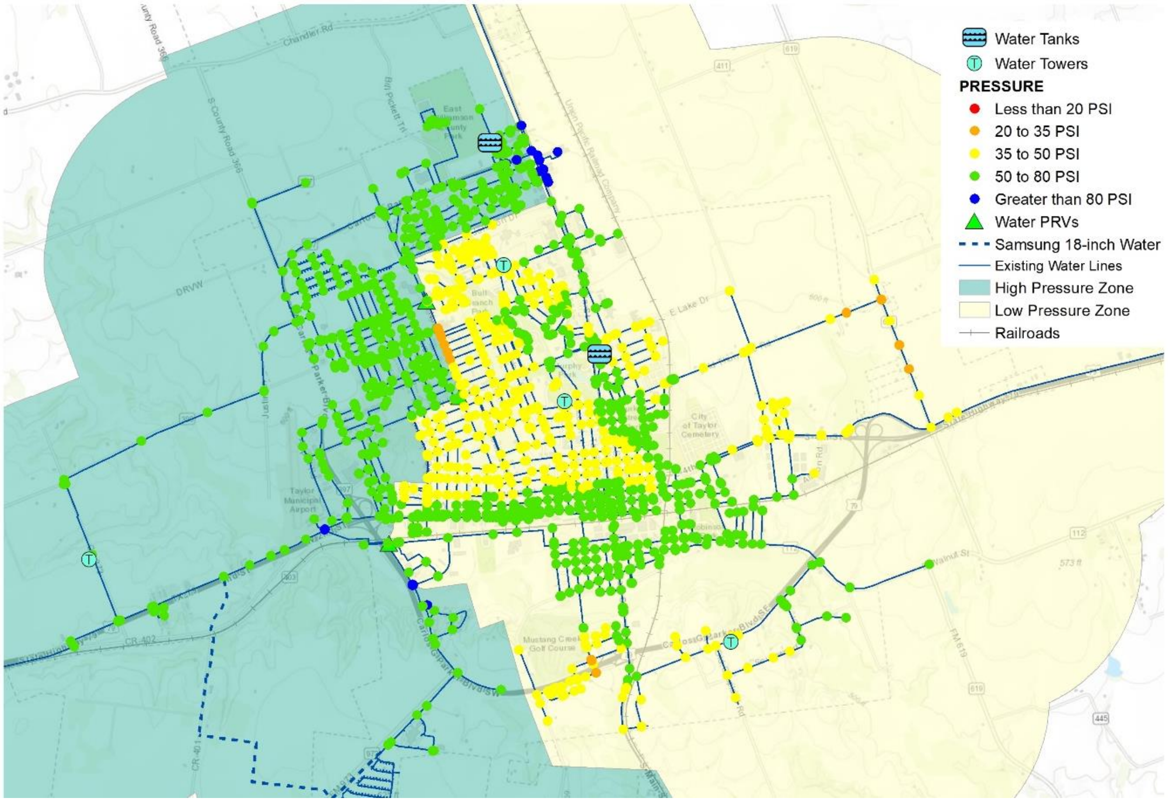



**Figure 2-5. Existing System Peak Hour Pressure**

To evaluate the system's ability to provide adequate fire suppression, a system-wide fire flow analysis was conducted. TCEQ requires that a minimum residual pressure of 20 PSI be maintained during fire flow conditions. The modeling software includes a fire flow analysis tool that will provide how much fire flow the system can provide to hydrants at a residual pressure of 20 PSI. This analysis of available fire flow was applied to a maximum demand day at model nodes near Taylor's fire hydrants.

The model indicates that the upper pressure plane can generally sustain a fire flow of at least 1,000 GPM, with some areas exceeding 3,000 GPM. The hydraulic model assumes that all system isolation valves are open; inadvertently closed valves could generate different system conditions. A valve inventory and exercising program is recommended to understand the location, status and condition of existing valves.

There are isolated areas in the central portion of Taylor, in the lower pressure plane, that have an available fire flow of less than 1,000 GPM. However, the more widespread issues with fire flow availability are focused on the east side of the City near FM 619, as well as in the south along Carlos Parker and in the Southwood Hills area. Bringing the Southwood Hills EST back online does provide some additional fire flow to this area, but there are still widespread areas the model predicts cannot achieve 1,000 GPM of fire flow, as seen in Figure 2-6.

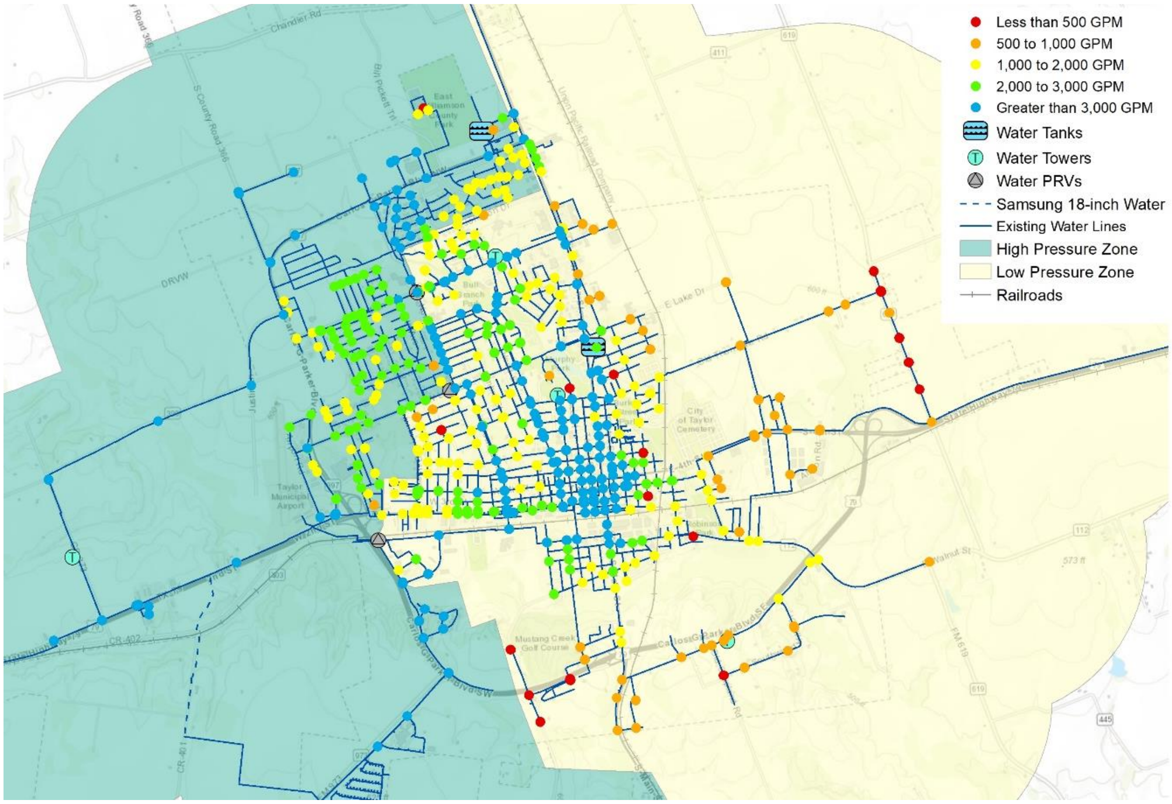



**Figure 2-6. Existing System Available Fire Flow During Maximum Day Demand**

## 2.4.6 Recommended Distribution System Improvements

The hydraulic model was used to determine the potential impact of improvements to address apparent deficiencies in the existing system. The recommended distribution system improvements can be generally summarized as:

- Replace pipes with known poor condition, requiring frequent repair
- Additional infrastructure to increase looping on the east side, reducing headlosses and providing more capacity
- Increasing looping on the south side, including a connection between pressure planes along Carlos Parker, east of FM 973. A PRV from the upper plane to the lower plane would keep the pressure planes isolated, but allow the southern portion of the city to receive flow from the upper pressure plane during emergency conditions
- Replacing select small diameter mains where the hydraulic model predicts it will have the greatest impact on available fire flow

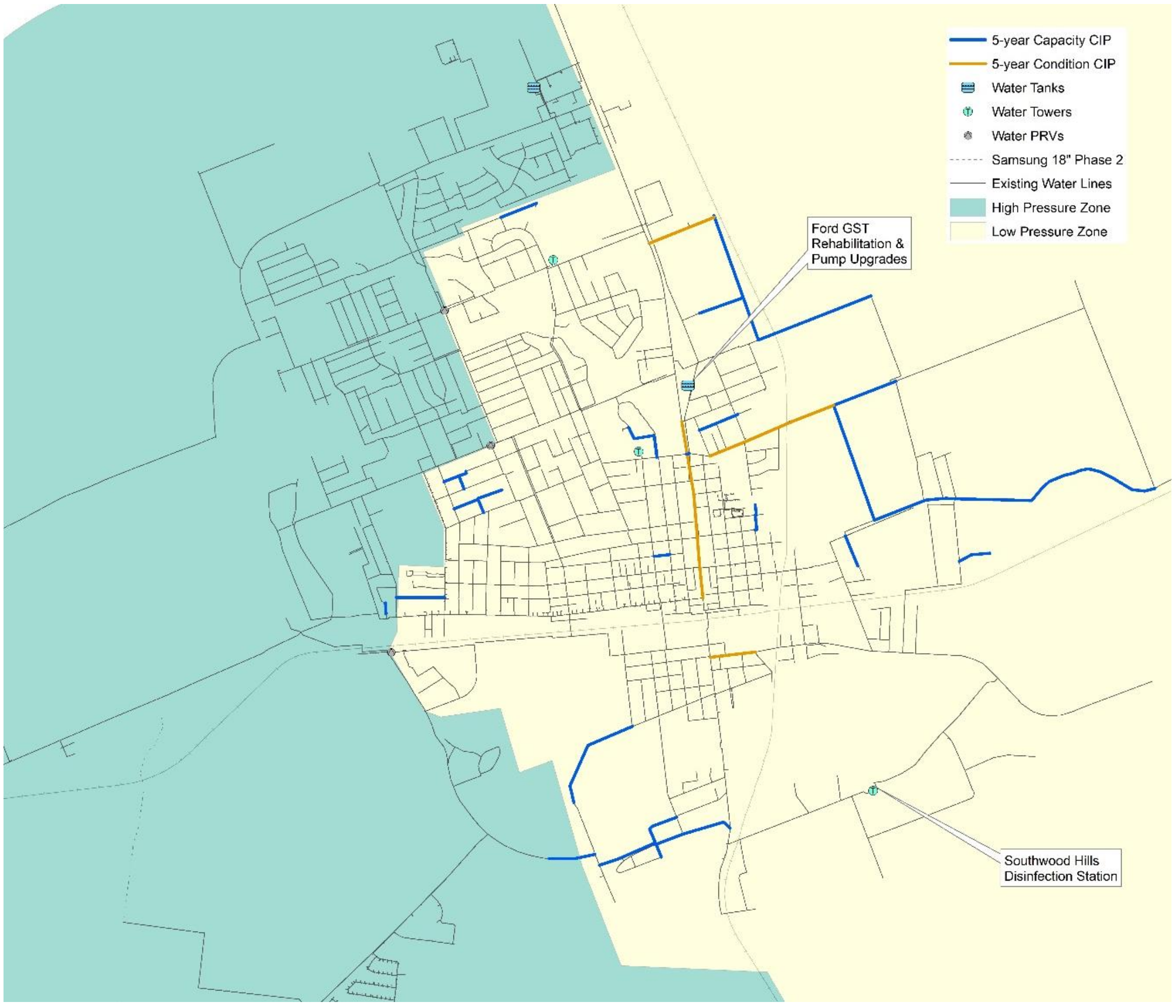



**Figure 2-7. 5-Year Capital Improvement Projects to Address Existing Water Distribution System Deficiencies**

The hydraulic model predicts that with these CIP projects in place, the water distribution system will generally provide 1,000 GPM of fire flow to all areas of Taylor, as shown in Figure 2-8.

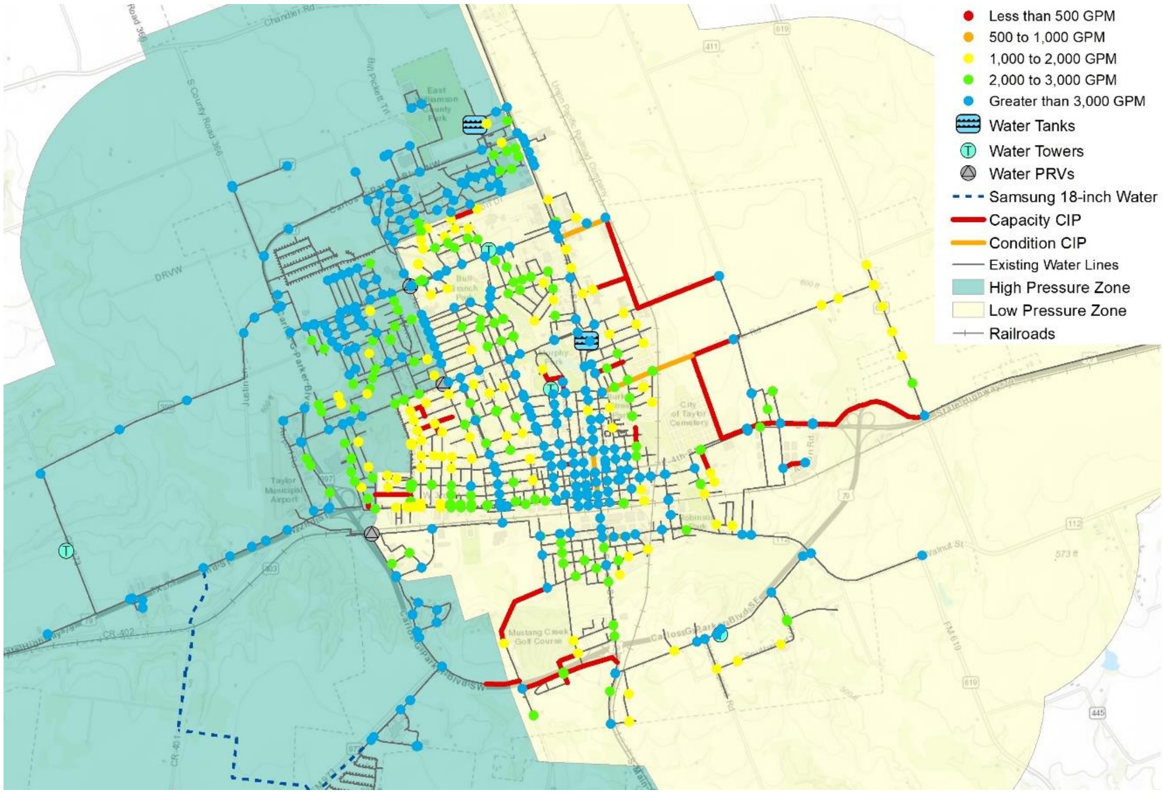



**Figure 2-8. Available Fire Flow During Maximum Day Demand, with 5-year Capital Improvement Projects to Address Existing System Deficiencies**

## 2.4.7 Water Supply Recommendations

The City of Taylor is currently dependent on the BRA East Williamson County WTP, as well as an existing 27-inch bar wrapped concrete steel cylinder transmission main for all of its water. The 27-inch transmission main was installed in 1990 and does not have a history of failures; however, it represents a single point of failure for all of Taylor's water supply. The following are recommendations to reduce the risk associated with the transmission main:

- Conduct a condition assessment of the existing 27-inch transmission main to better understand its anticipated remaining useful life.
- Coordinate with BRA on an emergency repair plan, having repair supplies and contractors on-call who could quickly repair a break and minimize downtime.

A second transmission main would provide redundancy in water transmission, but still leaves the WTP as a single point of failure in Taylor's water supply. It is recommended that Taylor coordinate with BRA to study and identify potential redundant or backup water supply alternatives to supplement the Lake Granger WTP. The alternative supply would be intended to provide average day demand for the City in the event of an emergency.

## 2.5 Water Demand Projections

Maintaining the historical average demand of 120 gallons/person/day, the anticipated demands for the 2040 projected population of 39,552 are listed below. This assumes no average increase in system efficiency. While newer infrastructure will have lower rates of water loss, newer subdivisions will more likely have automatic sprinkler systems, so the system-wide average usage is assumed to stay constant. However, in addition to reduced system loss, water conservation efforts are recommended to further reduce the City's average per capita daily water demand.

- Average day demand: 4.75 MGD
- Maximum day demand: 11.4 MGD
- Peak hour demand: 14.25 MGD

## 2.6 Future System Analysis

These future demands were loaded into the hydraulic model to evaluate where improvements may be necessary to accommodate the growth that is anticipated by 2030 and 2040. The demands were spatially allocated according to the comprehensive plan growth sectors and assumed population densities listed previously, in addition to increasing population densities in existing service areas to reflect infill growth.

The southeast portion of the City has become an area of high interest for larger, industrial customers. The water usage for this type of customer, including required fire flow, can vary widely depending on the specific industry. The existing system improvement recommendations in the 5-year CIP to improve water pressure and available fire flow to existing customers is not sufficient to accommodate the projected 10-year and 20-year growth, and additional CIP projects are recommended.

The addition of the Samsung plant on the southwest side of the City is expected to spur more industry south of 79 and west of Carlos Parker. Located in the upper pressure plane, the system is able to provide significant water to this area at adequate pressure, but new distribution infrastructure will be required to deliver water to this area.

### 2.6.1 Elevated Storage

The projected future system demands in 2030 and 2040 were compared to the existing elevated storage volumes in the lower and upper pressure planes, as shown in Table 2-8.

**Table 2-8. Future Elevated Storage Evaluation**

| Year | Pressure Plane | Existing Elevated Storage Volume (MG) | Peak Hour Volume (MG)[1] | Fire Flow Volume (MG)[2] | Minimum Elevated Storage Volume (MG) | Surplus/ (Deficit) (MG)[3] |
|---|---|---|---|---|---|---|
| 2030 | Upper | 1.00 | 0.27 | 0.60 | 0.87 | 0.13 |
| | Lower | 1.50 | 0.55 | 0.60 | 1.15 | 0.35 |
| 2040 | Upper | 1.00 | 0.40 | 0.60 | 1.00 | 0.00 |
| | Lower | 1.50 | 0.79 | 0.60 | 1.39 | 0.11 |

1. Half of peak hour demand for a duration of four hours.
2. Maximum fire flow assumed to be 2,500 GPM for a duration of four hours.
3. Existing Elevated Storage Volume minus Minimum Elevated Storage Volume.

The City's existing elevated storage is adequate to provide the minimum recommended elevated storage volume of half of projected peak hour demands until 2040, plus fire flow of 2,500 GPM, for a duration of four hours. The Samsung facility will include its own on-site fire protection system.

Faster than anticipated growth, including any new large industrial users, could accelerate the need for additional elevated storage. Large industrial facilities can vary widely on required fire flow, and can exceed what a typical water distribution system can provide and should be evaluated carefully. It is recommended that a proposed facility that will require more than 2,500 GPM of fire flow for four hours include on-site fire flow storage and booster pumping for fire suppression of their facility.

It should also be noted that the project to expand the North pump station firm capacity from 4,600 GPM to 9,200 GPM is underway and projected to be completed by summer of 2023, and can supplement fire flow in the upper pressure plane.

### 2.6.2 Ground Storage

The projected future system demands in 2030 and 2040 were compared to the existing ground storage volumes in the lower and upper pressure planes, as shown in Table 2-9.

**Table 2-9. Future Ground Storage Evaluation**

| Year | Pressure Plane | Existing Ground Storage Volume (MG) | 8 hours of Maximum Day Demand (MG)[1] | 12 hours of Maximum Day Demand (MG)[1] | Recommended Ground Storage Volume (MG) | Surplus/ (Deficit) (MG)[2] |
|---|---|---|---|---|---|---|
| 2030 | Upper | 1.00 | 0.87 | 1.31 | 2.00 | 0.69 |
| | Lower | 1.00[3] | 1.77 | 2.65 | 1.00 | (1.65) |
| 2040 | Upper | 1.00 | 1.25 | 1.88 | 2.00 | 0.12 |
| | Lower | 1.00[3] | 2.55 | 3.82 | 1.00 | (2.82) |

1. These volumes do not include any demands for the Samsung facility, which will have its own storage at the Taylor delivery point.
2. Recommended Ground Storage Volume minus volume required for 12 hours of Maximum Day Demand.
3. This volume represents only the Ford GST and do not include any storage at the BRA WTP, which supplements the City's ground storage in the lower pressure plane.

For the upper pressure plane, the City's existing 1 MG of ground storage volume at the North GST site is not adequate to provide 12 hours of projected 2030 maximum day demands. The addition of a second 1 MG ground storage tank, for a total of 2 MG of ground storage, will provide 12 hours of projected maximum day demand through 2040.

For the lower pressure plane, the City's existing 1 MG ground storage volume at the Ford GST is not adequate to provide 12 hours of projected maximum day demands in 2030 and 2040. However, because the water Taylor purchases from BRA enters directly into Taylor's lower pressure plane, the treated water storage volume at the BRA WTP counts towards the City's system storage for the lower pressure plane. Therefore, no additional ground storage is required in the City's lower pressure plane.

### 2.6.3 Booster Pumping

The projected future system demands in 2030 and 2040 were compared to the existing booster pumping firm capacity in the lower and upper pressure planes, as shown in Table 2-10.

Once the North pump station is expanded to meet the peak of Samsung's demands in late 2023 and early 2024, it will provide significant booster pumping capacity to the City once the Samsung demands decrease.

**Table 2-10. Future Booster Pumping Evaluation**

| Year | Pressure Plane[1] | Firm Capacity (GPM) | 50% pf Peak Hour Demand (GPM) | Samsung Peak Demand (GPM)[2] | Surplus/ (Deficit) (GPM)[3] |
|---|---|---|---|---|---|
| 2030 | Upper | 9,200[4] | 1,133[4] | 604 | 7,463 |
| 2040 | Upper | 9,200[4] | 1,632[4] | 604 | 6,964 |

1. Lower pressure plane booster pumping requirements are met by the BRA WTP high service pumps.
2. Projected long-term Samsung peak demand from the City in current agreement.
3. Firm Capacity minus 50% of Peak Hour Demand minus Samsung Peak Demand.
4. Includes projected firm capacity once North pump station expansion is completed.

### 2.6.4 TCEQ Requirements

The projected future system demands in 2030 and 2040 were compared to the TCEQ requirements for elevated storage, total storage, and booster pumping in the upper pressure plane, as shown in Table 2-11.

**Table 2-11. TCEQ Future Requirements Summary for the Upper Pressure Plane**

| Year | Projected Water Connections | Description | Required by TCEQ | City of Taylor Facilities | Surplus/ (Deficit) |
|---|---|---|---|---|---|
| 2030 | 3,026 | Elevated Storage (MG) | 0.30[1] | 1.00 | 0.70 |
| | | Total Storage (MG) | 0.61[2] | 3.00[4] | 2.39 |
| | | Booster Pumping (GPM) | 1,816[3] | 9,200[5] | 7,384 |
| 2040 | 4,362 | Elevated Storage (MG) | 0.44[1] | 1.00 | 0.56 |
| | | Total Storage (MG) | 0.87[2] | 3.00[4] | 2.13 |
| | | Booster Pumping (GPM) | 2,618[3] | 9,200[5] | 6,582 |

1. 100 gallons/connection.
2. 200 gallons/connection.
3. 0.6 gallons/minute/connection.
4. Includes addition of 1 MG GST at existing North GST site.
5. Includes expansion to North pump station currently under construction.

The projected future system demands in 2030 and 2040 were compared to the TCEQ requirements for elevated storage, total storage, and booster pumping in the lower pressure plane, as shown in Table 2-12.

**Table 2-12. TCEQ Future Requirements Summary for the Lower Pressure Plane**

| Year | Projected Water Connections | Description | Required by TCEQ | City of Taylor Facilities[1] | Surplus/ (Deficit) |
|---|---|---|---|---|---|
| 2030 | 6,144 | Elevated Storage (MG) | 0.61[2] | 1.50 | 0.89 |
| | | Total Storage (MG) | 1.23 [3] | 2.50[5] | 2.27 |
| | | Booster Pumping (GPM) | 3,686[4] | N/A[6] | N/A |
| 2040 | 8,857 | Elevated Storage (MG) | 0.89[2] | 1.50 | 0.61 |
| | | Total Storage (MG) | 1.77[3] | 2.50[5] | 2.13 |
| | | Booster Pumping (GPM) | 5,314[4] | N/A[6] | N/A |

1. Does not include storage or high service pumping at the BRA WTP.
2. 100 gallons/connection.
3. 200 gallons/connection.
4. 0.6 gallons/minute/connection.
5. Does not include treated water storage at the BRA WTP.
6. The lower pressure plane does have one booster pumping station at the Ford GST, of unknown capacity. Because the lower pressure plane is fed directly from the BRA WTP, the capacity of the WTP high service pumps serves as the lower pressure plane booster pumping capacity, and exceeds the capacity required by TCEQ.

## 2.7 Water System Capital Improvements Plan

Considering existing system improvements, future growth, and the recommended criteria discussed previously, water system improvements were developed to serve the anticipated growth until 2040. These improvements can generally be categorized as:

- Distribution system improvements to increase system pressures and provide adequate fire flow, particularly in southeast and southwest Taylor.
- Water supply redundancy, including a parallel transmission main from BRA and further evaluation of a redundant water supply.

The 5-year, 10-year and 20-year water system CIP projects are shown in Figure 2-9. Generally, capital improvement projects were sized to convey maximum day demand flows at a velocity of 5 feet per second or less, and convey peak hour flows at a velocity 10 feet per second or less. Taylor is currently in the process of updating the engineering manual and associated infrastructure design sizing criteria. A future update to this master plan will consider Taylor's selected design criteria and any associated changes in proposed CIP project sizing.


**FIGURE 2-9. WATER DISTRIBUTION SYSTEM 20-YEAR CAPITAL IMPROVEMENTS PLAN**

An opinion of probable construction cost (OPCC) was estimated for each project based upon conservative, planning level unit costs as well as an allowance for professional services (survey, geotechnical investigation and engineering design) and contingency for unknowns.

The OPCCs are for distribution mains and do not include distribution infrastructure within individual subdivision developments. The linear footage for each project is based on an assumed, approximate alignment that will be refined during design.

The summary of the 5-year, 10-year and 20-year Water System CIP OPCC are shown in Table 2-13, Table 2-14, and Table 2-15, respectively. The 5-year CIP includes projects that are generally recommended to address existing system issues, and the 10-year and 20-year CIP includes projects that are recommended to serve the City's projected growth. The 10-year CIP generally includes new or upsized collection mains where the more immediate growth is anticipated.

The projects are generally listed in order of priority. However, as growth patterns change, the CIP should be re-evaluated to determine if the timing for projects should be updated.

The OPCCs presented are considered Class 5 estimates as defined by the Association for the Advancement of Cost Engineering (AACE) Recommended Practice No. 18R. Class 5 estimates are provided at a project definition level of 0 to 2%, with a project accuracy range of -50% to +100%. The costs presented are in 2023 dollars and have not been escalated into the future.

**Table 2-13. Water Distribution System 5-year CIP Summary**

| CIP ID | Description | Total OPCC (2023 $) |
|---|---|---|
| 1 | Existing Transmission Main Condition Assessment & Emergency Repair Materials | $250,000 |
| 2 | Alternate Water Supply Evaluation | $200,000 |
| 3 | Valve Inventory and Exercising Program | $500,000 |
| 4 | Southwood Hills Disinfection Station | $700,000 |
| 5 | Lead Service Line Inventory & Replacements | $1,500,000 |
| 6 | Replace existing 8" along Highland Drive, due to poor condition | $1,000,000 |
| 7 | Replace existing 6" along Old Thorndale with 8", due to poor condition | $1,500,000 |
| 8 | Replace existing 8" along N. Main St, from Hosack St to 6th St, due to poor condition, with a 10" main | $1,900,000 |
| 9 | Replace existing 8" along E. MLK St., due to poor condition | $800,000 |
| 10 | New 16" line west of RR from Highland Dr to E. Lake Dr | $1,800,000 |
| 11 | New 16" line along E. Lake Dr. | $1,800,000 |
| 12 | New 8" line between Old Granger Road and RR | $600,000 |
| 13 | Upsize existing 8" line along Old Thorndale, west of Gravel Pit Road | $1,200,000 |
| 14 | New 12" line between Old Thorndale and E. Lake Dr. | $1,300,000 |
| 15 | New 12" line along Carlos Parker with PRV | $900,000 |
| 16 | Upsize existing & install new 8" line along Rice's Crossing | $1,700,000 |
| 17 | Upsize existing & install new 8" along Potomac Road | $1,100,000 |
| 18 | Upsize existing 8" along E. 4th St., from cemetery to FM 619 | $2,900,000 |
| 19 | New 8" line along W. Rio Grande St | $900,000 |
| 20 | Upsize existing 2" line to 8" along Oscar St. | $600,000 |
| 21 | New 12" connection under Main St. at West 12th St. | $400,000 |
| 22 | Upsize existing & install new 8" line near Murphy Park | $1,200,000 |
| 23 | Upsize existing 2" lines along Burkett St. | $500,000 |
| 24 | Upsize existing 2" line to 8" along W. 5th St. | $400,000 |
| 25 | Upsize existing 6" line to 8" near Allison Road | $500,000 |
| 26 | Upsize existing 2" line to 8" along Gym St. | $500,000 |
| 27 | New 8" line near Carlos Parker and 79 | $400,000 |
| 28 | Upsize existing 2" lines along W. 3rd St. | $700,000 |
| 29 | Upsize existing 2" line to 8" along Adams St. | $1,000,000 |
| 30 | Upsize existing 2" lines to 8" along Grace St. and Prather St. | $600,000 |
| 31 | Upsize existing 2" line to 8" along Johnson Dr | $600,000 |
| 32 | Ford GST rehabilitiation and pump upgrades | $800,000 |
| | Total 5-year CIP | $30,750,000 |

**Table 2-14. Water Distribution System 10-year CIP Summary**

| CIP ID | Description | Total OPCC (2023 $) |
|---|---|---|
| 33 | New 16" line along RR | $3,300,000 |
| 34 | New 12" line along Carlos Parker | $2,800,000 |
| 35 | New 8" line under RR, near Mustang Creek | $1,300,000 |
| 36 | New 8" line connecting to Carlos Parker | $1,300,000 |
| 37 | New 16" line along Mallard Lane, to EST | $1,300,000 |
| 38 | 1 MG GST at North Pump Station | $7,500,000 |
| 39 | New 12" line along Carlos Parker | $2,200,000 |
| 40 | New 18" line along Justin Lane | $3,300,000 |
| 41 | New 12" line southwest of 79 & Carlos Parker | $2,100,000 |
| 42 | New 12" line southwest of 79 & Carlos Parker | $1,300,000 |
| 43 | New 12" line southwest of 79 & Carlos Parker | $1,000,000 |
| 44 | New 12" line connecting to FM 973 waterline | $1,300,000 |
| 45 | New 18" line west of Justin Lane | $3,500,000 |
| 46 | New 18" line south of CR398 | $1,500,000 |
| 47 | New 12" line south of CR398 | $2,400,000 |
| 48 | New 12" line to serve future developments | $3,700,000 |
| 49 | Parallel Transmission Main from BRA | $24,900,000 |
| 50 | New 12" line west of Wesley Miller Lane | $2,300,000 |
| 51 | New 8" line south of Carlos Parker & Windy Ridge | $900,000 |
| | **Total 10-year CIP** | **$67,900,000** |

### Table 2-15. Water Distribution System 20-year CIP Summary

| CIP ID | Description | Total OPCC (2023 $) |
|---|---|---|
| 52 | New 24" line from Transmission Main to North GST | $1,400,000 |
| 53 | New 8" along CR398, east of Justin Lane | $900,000 |
| 54 | New 12" line along RR to serve future developments | $1,400,000 |
| 55 | New 12" line along RR to serve future developments | $1,100,000 |
| 56 | New 12" line along RR to serve future developments | $1,500,000 |
| 57 | New 12" line to serve future developments | $1,900,000 |
| 58 | New 12" line to serve future developments | $1,700,000 |
| 59 | New 12" line to serve future developments | $1,100,000 |
| 60 | New 12" line to serve future developments | $1,000,000 |
| 61 | New 12" line to serve future developments | $1,000,000 |
| 62 | New 12" line to serve future developments | $1,600,000 |
| 63 | New 12" line to serve future developments | $3,300,000 |
| 64 | New 12" line to serve future developments | $1,600,000 |
| 65 | New 8" line to serve future developments | $900,000 |
| 66 | New 12" line to serve future developments | $1,600,000 |
| 67 | New 12" line to serve future developments | $1,500,000 |
| 68 | New 8" line to serve future developments | $1,300,000 |
| 69 | New 8" line to serve future developments | $1,300,000 |
| 70 | New 12" line to serve future developments | $1,300,000 |
| 71 | New 8" line to serve future developments | $800,000 |
| 72 | New 8" line to serve future developments | $1,000,000 |
| 73 | New 8" line to serve future developments | $1,100,000 |
| 74 | New 8" line to serve future developments | $900,000 |
| 75 | New 8" line to serve future developments | $600,000 |
| 76 | New 12" line to serve future developments | $2,100,000 |
| 77 | New 12" line to serve future developments | $1,300,000 |
| 78 | New 8" line to serve future developments | $600,000 |
| 79 | New 8" line to serve future developments | $2,200,000 |
| 80 | New 8" line to serve future developments | $1,000,000 |
| 81 | New 8" line to serve future developments | $2,400,000 |
| 82 | New 16" connection from Transmission Main to North GST | $1,000,000 |
| | **Total 20-year CIP** | **$42,400,000** |

With these improvements in place, the 2040 peak hour pressures predicted by the hydraulic model are shown in Figure 2-10, and Figure 2-11 shows the model predicted available fire flow during 2040 maximum day demand conditions.

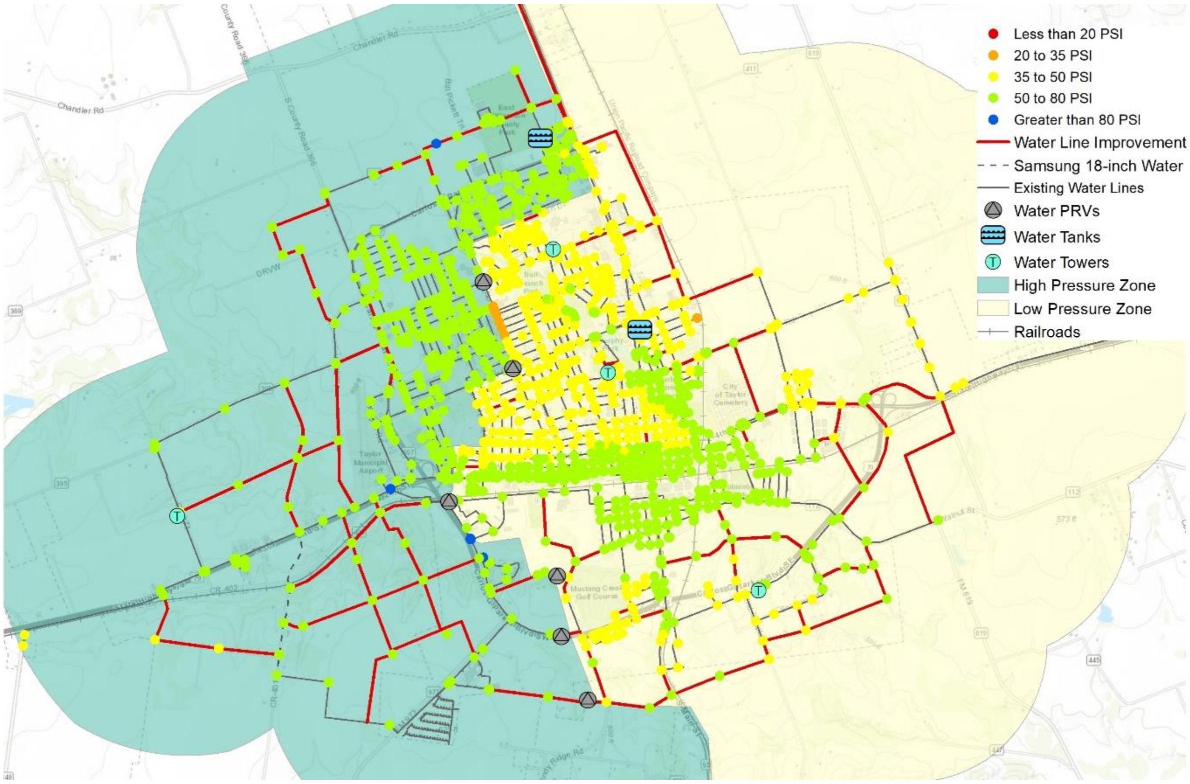



**Figure 2-10. 2040 Peak Hour Pressure with CIP Recommendations Constructed**

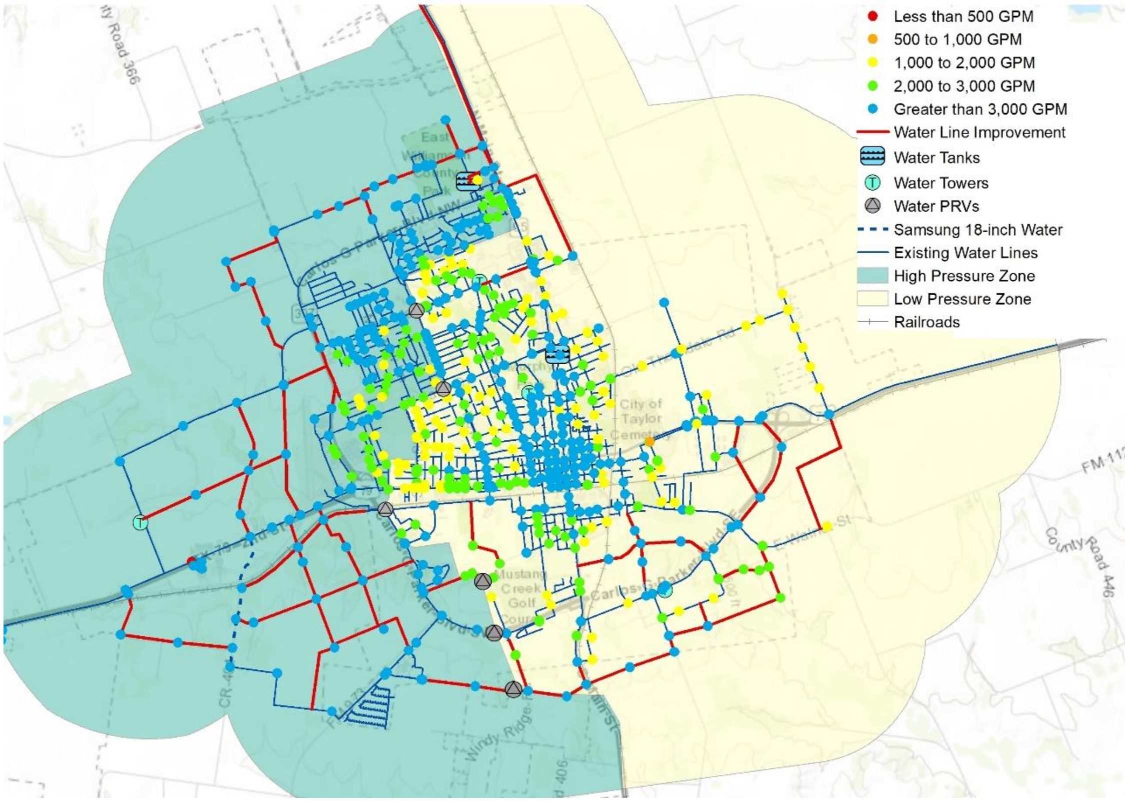



**Figure 2-11. 2040 Available Fire Flow with CIP Recommendations Constructed**

# 3 Wastewater Collection System

The City of Taylor's wastewater collection system is comprised of a gravity network generally ranging in size from four inches to 42 inches, and three lift stations with associated force mains that convey wastewater flows to the Mustang Creek wastewater treatment plant (WWTP) owned and operated by the City. Construction projects are currently underway to allow the WWTP to treat the permitted capacity of 4.0 MGD. Current average flows to the WWTP are approximately 1.25 MGD.

The wastewater collection system conveys both dry weather flows, and wet weather flows when inflow and infiltration (I&I) enter the sanitary sewer collection during rainfall events. The existing wastewater collection system is shown in Figure 3-1.


**Figure 3-1. Existing Wastewater Collection System**

## 3.1 Existing System Analysis

To gauge the existing system's capacity to convey peak wastewater flows, a hydraulic model of the wastewater collection system was developed. The hydraulic model was based primarily on Taylor's GIS data of the collection system, information from Taylor staff, flow metering data, and field site observation visits after a rainfall event. Future flow projections were included in the model to develop future wastewater system improvements.

### 3.1.1 Hydraulic Model Development

#### Network Development

The primary data sources for this project that were used to build the hydraulic model were:

- Water Consumption and Billing data for residential, industrial and commercial users from the City of Taylor
- City of Taylor GIS Data
- City of Taylor Manhole Survey Data, 2018. The survey primarily included an elevation survey of manholes on wastewater lines 12 inches in diameter or larger, where accessible.
- Revenue Requirements, Cost of Service, and Rate Design Study for Water and Sewer Service Report by Black and Veatch
- Water and Wastewater Impact Fee Update from the City of Taylor
- Hach flow metering data (collected in 2022, following the initial model network development)

The main data source used for the hydraulic model network development was the GIS database provided by the City. GIS data is typically maintained by wastewater agencies and cities as a mapping and data management tool for sewer assets, and provides good data for development of the pipe alignments, diameters and manhole locations. Where provided in the GIS database, pipe invert elevations and manhole invert and rim elevations were imported directly into the model network. Surveyed manhole elevations and pipe depths were used to estimate pipe invert elevations in the model. This assumed data was flagged as “Assumed” in the model using InfoWorks data flags.

The following describes the specific GIS data fields utilized in the hydraulic model network:

- Pipe Diameter. This was imported as the modeled pipe diameter.
- Manhole Top Elevation and Surface Elevation. Comparing the values of these fields to terrain data (LiDAR), the Top Elevation value generally appears closest to the terrain data, and was used in the model. Where the Top Elevation field was empty, the Surface Elevation field was used. This data was flagged as “GIS” in the model using InfoWorks data flags.
- Manhole Invert Elevation. This was imported to the model as chamber floor. Where this invert value was higher than an attached pipe invert, the field was changed to the default calculation, which is to match the lowest invert.
- Pipe Material. Where available, the pipe roughness coefficient was set based on the pipe material identified in GIS or during the manhole survey. The Manning’s n roughness coefficient ranges from 0.011 and 0.018 for the sanitary system. Pipe

entrance and exit losses were estimated using the software inference tool based on the angles of pipes connecting at the manhole.

### *System Connectivity*

Capturing the system connectivity in the hydraulic model is crucial to projecting wastewater flows in the system. The connectivity was checked and modified in the hydraulic model, based on the manhole survey data and profiles generated in the modeling software. The manhole survey provided the following data:

- Number of pipes connected to the manhole.
- Azimuth, pipe size, invert and material of each pipe connected to the manhole.

The azimuths and pipe data recorded in the manhole survey were plotted in GIS and compared to the GIS pipe network to evaluate and update the system connectivity. Some discrepancies remained where assumptions were required. This assumed data was flagged as “Assumed” in the model using InfoWorks data flags. Some examples include:

- Azimuth indicated a pipe in a certain direction, but no manhole was identified in that direction.
- Pipe of certain size and material was identified in a certain direction, but the next manhole in that direction identified a different size and material of pipe.

### *Network Summary*

The sanitary system can be categorized as the Mustang Creek Interceptor basin and the Bull Branch Interceptor basin. The Mustang Creek Interceptor is a 36-inch pipe that runs north of Mustang Creek, generally from Carlos Parker to E. Martin Luther King Jr. Blvd. The Bull Branch Interceptor runs along Bull Branch, which is a tributary to Mustang Creek, generally from NW Carlos Parker Blvd. to E. Martin Luther King Jr. Blvd., and ranges in size from 15-inches to 18-inches.

Initially, only pipes 10-inch in diameter and larger were included in the model. Some smaller diameter pipes were included where necessary to maintain model connectivity, or where considered hydraulically significant. Hydraulically significant areas are generally lines that provide a cross link between major lines or allow for possible overflow between lines during wet weather flow. After review of flow metering data collected after the initial model development, and reassessment of the manhole survey data in conjunction with field visits after a wet weather event, it was decided to include all pipes in the model for the purposes of loading sanitary sewer flows to the network. This decision aids in the analysis of development capacity requests that are generally located more upstream in the upper system where they would connect to 8-inch pipes. However, because these small diameter pipes were not surveyed, their slope is assumed, which has a significant impact on the model predicted available capacity and level of surcharging. Therefore, this report largely focuses on the 10-inch and larger network with regard to model results.

The final hydraulic model network and extents are shown in Figure 3-2, with the Mustang Creek basin shown in green and the Bull Branch basin shown in yellow. The green pipe segments and manholes are those included in the 10-inch and larger network.

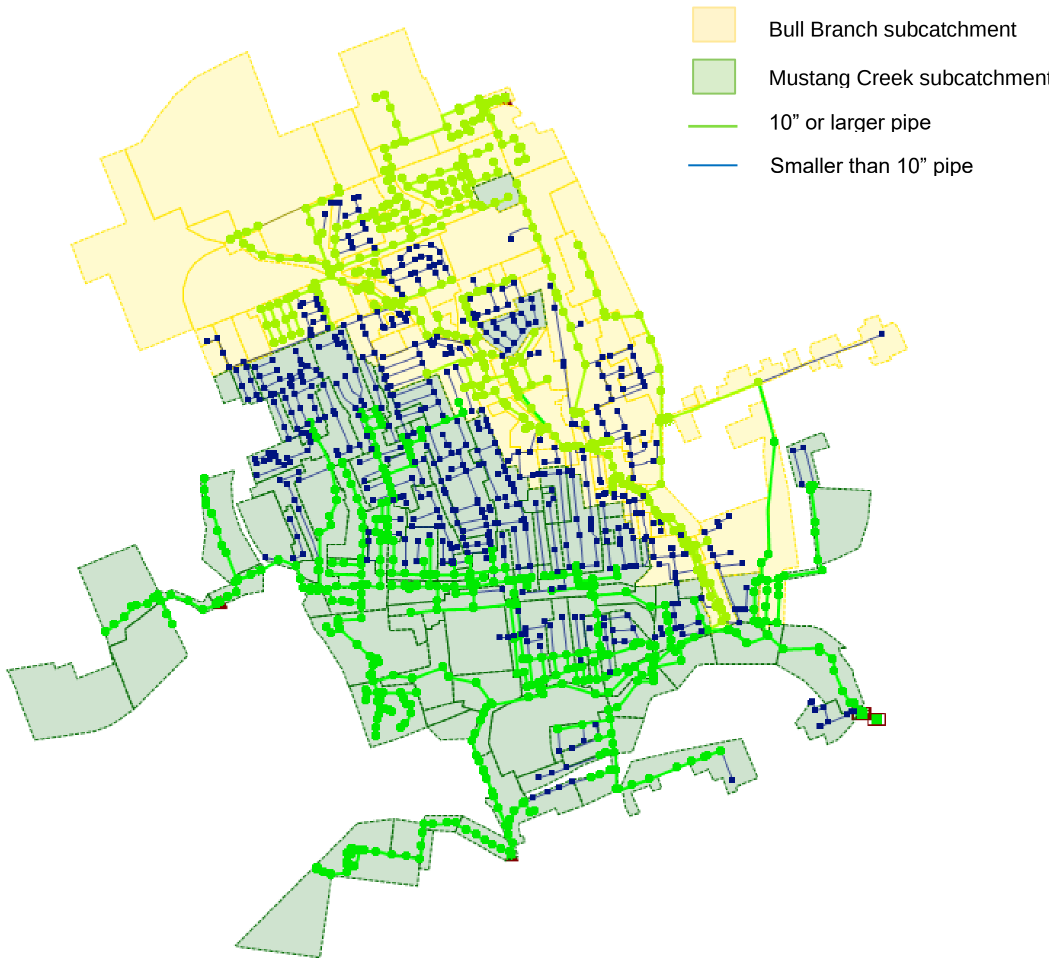



**Figure 3-2. Existing System Hydraulic Model Network**

The model components included in the entire network, as well as in the 10-inch and larger network, are summarized in Table 3-1.

**Table 3-1. Hydraulic Model Element Summary**

| Model Elements | >= 10-in | Full System |
|---|---|---|
| Manholes | 859 | 1412 |
| Pipe Segments | 881 | 1464 |
| Pipe Length (feet) | 263,893 | 475,896 |
| Lift Stations | 3 | 3 |
| Subcatchments | 169 | 169 |
| Subcatchments Total Area (acres) | 5809 | 5809 |

### *Lift Stations*

The City's three wastewater lift stations were modeled based on information provided by the City regarding pump and wet well capacity, as well as force main diameters. The lift stations below were included in the hydraulic model:

- Airport Lift Station. This lift station, located south of the Airport, is comprised of two wet wells in series. The upstream (west) wet well is six feet in diameter, approximately 30 feet deep and includes one pump. This pump pumps into the downstream (east) wet well, that is also six feet in diameter, approximately 20 feet deep and includes two pumps that discharge into a 4-inch force main. Wet well draw down testing indicates these pumps are operating at a capacity of approximately 90 GPM, which is far left of the pump's best efficiency point. Several factors limit the capacity of the pumps, including the small force main diameter and single-phase electricity to the lift station site.
- Rice's Crossing. This lift station, located south of Carlos Parker on Windy Ridge Road, includes a 10-foot diameter wet well that is approximately 20 feet deep and includes two pumps that discharge into a 10-inch force main. This lift station includes the same pumps as the downstream (east) wet well at the Airport lift station. The larger force main diameter from the Rice's Crossing lift station allows the Rice's Crossing lift station to operate closer to the pump design point of approximately 600 GPM.

### *Downstream Boundary Condition*

The wastewater treatment plant headworks was set as the system outfall in the hydraulic model, or a place where flow is allowed to leave the collection system. The outfall was not modeled as a free outfall, which would provide for unrestricted flow out of the collection system. Data on the WWTP headworks pump capacity and operation was used to simulate the water level at the WWTP headworks and the associated impacts upstream in the collection system.

## Flow Allocation

Flow rate development includes delineation of the service area sub basins, or subcatchments, that are used to load wastewater flows into the collection system model. Population within a subcatchment is the primary source of estimated dry weather flow

(DWF) sanitary sewer loading generated by that subcatchment. Factors such as subcatchment area, percent impervious cover, and soil absorption are used to simulate the wet weather flow (WWF) contribution from a subcatchment.

### *Subcatchment Delineation*

Subcatchment boundaries are generally delineated based on land use data. A subcatchment assigned as residential or commercial indicates that the land use within the boundary of that subcatchment is predominately that designation. Mixed Use is where there is a large mixed proportion of residential and commercial within the subcatchment boundary. During model calibration, this allows model parameters to be adjusted within a subcatchment area, especially for wet weather runoff parameters.

Model loading points are located where smaller, non-modeled lines connect to the collection system. These smaller, non-modeled lines were used to assist in subcatchment delineation along with other factors including land use and floodplain mapping.

### *Population*

The residential population distribution, and equivalent commercial population distribution, throughout the service area was determined for the defined loading subcatchments, primarily based on the following:

- Geolocated water consumption data
- Building footprint and type
- Parcel data
- Aerial photography

100 gallons per capita per day (GPCD) is a typical wastewater value utilized as a starting point to estimate sanitary sewer flow from residential areas.  40 GPCD is a typical wastewater value utilized as a starting point to estimate sanitary sewer flow from commercial areas.  During the dry weather calibration, the GPCD value and population of a metershed is adjusted to match observed data.

## Model Calibration

Flow metering data was collected for approximately six months in the first half of 2022, for use in calibrating the hydraulic model.  Four full (velocity and depth) flow meters were installed, supplemented by six depth-only sensors, as shown in Figure 3-3.

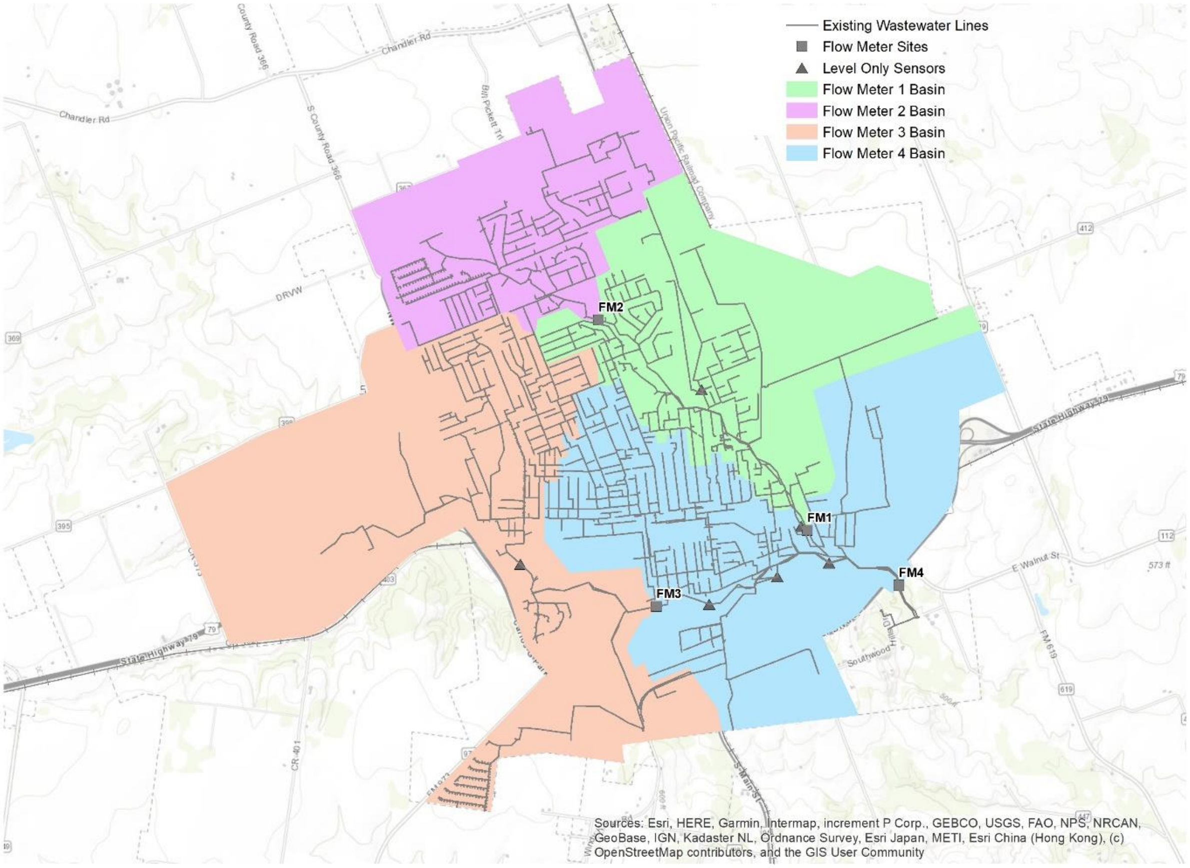



**Figure 3-3. Wastewater Flow Meter Basins**

Static model parameters adjusted during calibration include system geometry and population, simulating flows through the model network. Variable model parameters adjusted during calibration are primarily those defining where, how much and at what rate flow enters the model.

The process of dry weather model calibration generally includes:

- Adjusting GPCD values to match observed daily wastewater volumes
- Adjusting diurnal profiles to match observed daily variance in wastewater flow
- Adjusting non-residential loadings
- Adjusting baseflow (constant inflow into the system)

Once dry weather calibration is complete, the process of wet weather calibration generally includes adjusting the following parameters to match the peak flows recorded during wet weather events.

- Area contributing to runoff
- Percentage of impervious area and runoff coefficient of impervious surfaces
- Initial rainfall losses

- Runoff routing values
- Surface soil depth for pervious surfaces

A detailed description of the hydraulic model calibration is included in the City of Taylor Wastewater Flow Metering and Model Calibration technical memorandum, which is attached as Appendix A.

## Existing System Analysis

The calibrated hydraulic model was utilized to evaluate the existing system and identify any capacity deficiencies in the collection system, to establish a wastewater CIP. The CIP is primarily focused on conveying peak wet weather flows to reduce surcharging and sanitary sewer overflows predicted by the hydraulic model. CIP alternatives considered in the hydraulic model include upsizing existing infrastructure, installing new infrastructure, reducing peak wet weather flows, eliminating lift stations, and repairing poor condition mains. These alternatives were evaluated to determine the model-predicted impact of these improvements on system performance.

A 5-year, 6-hour wet weather assessment storm was applied to the calibrated hydraulic model. A hyetograph of the modeled assessment storm rainfall intensity and duration is shown in Figure 3-4. This storm is used commonly throughout Texas.

An assessment storm is considered the rainfall event that is used to define the sanitary sewer collection system's level of performance during such a rainfall event, commonly referred to as level of service. Typical levels of service range from no sanitary overflows during the rainfall event, to no pipe surcharging during the rainfall event. The desired level of service has a significant impact on the associated capital cost. The assessment storm and level of service criteria are different from the design criteria of a new pipe; existing infrastructure can be allowed to surcharge before a capital project is triggered.

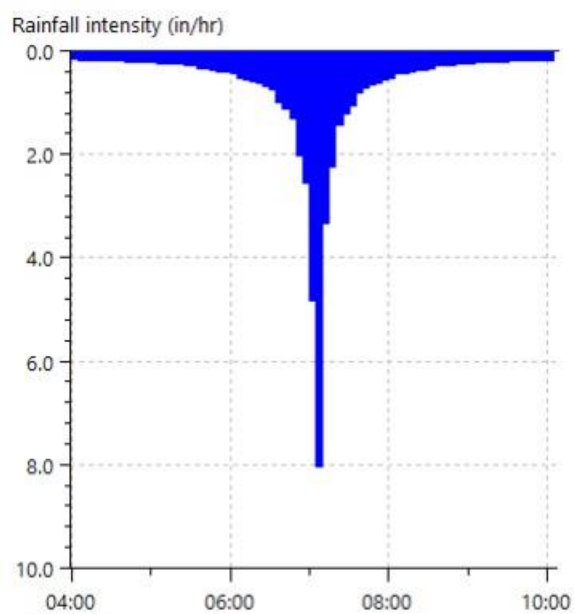



**Figure 3-4. 5-year, 6-hour Assessment Storm Hyetograph**

To understand the system response to a storm event of this magnitude, the model determines the peak flow that enters the collection system. The 5-year, 6-hour storm is applied to the system to coincide with the time of day the system is also conveying peak dry weather flows. This predicts the theoretical flow condition that would be most taxing on the collection system, and conveying these flows without any sanitary sewer overflows was established as the minimum desired level of service for the collection system.

Figure 3-5 displays a color-coded map representing the level of surcharging and sanitary sewer overflows predicted by the wastewater collection system during the 5-year, 6-hour assessment storm event. Surcharging can occur when a pipe diameter is too small to convey the upstream peak flows, or when a downstream system capacity constriction causes backwater. The main area of concern in the existing collection system identified through the hydraulic modeling is in the Bull Branch interceptor. This interceptor ranges in size from 15-inches to 18-inches, including some intermediate bottlenecks where the diameter decreases, and conveys flow from a significant portion of the City. The hydraulic model indicates that the Bull Branch interceptor is significantly surcharged during rainfall events.

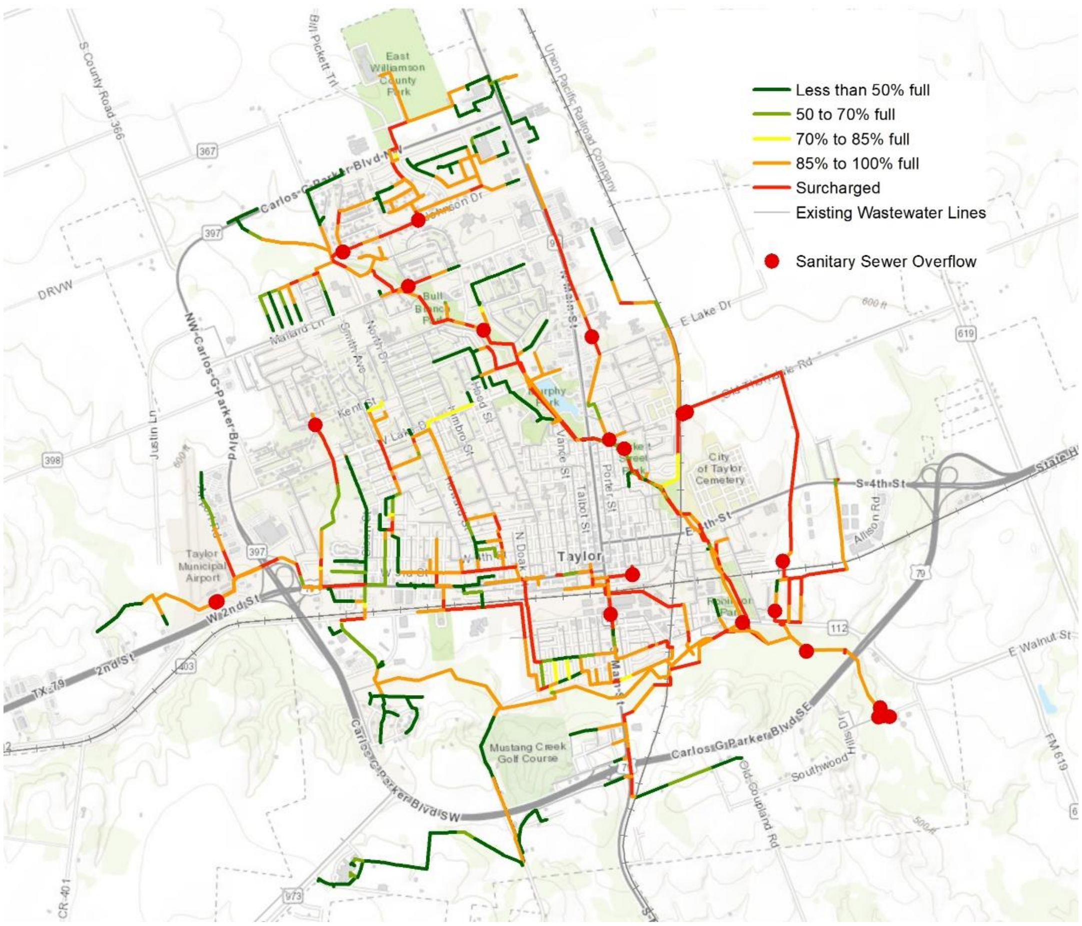



**Figure 3-5. Model-Predicted Level of Surcharging in Existing System During a 5-year, 6-hour Storm**

The hydraulic model was used to determine the potential impact of improvements to address apparent deficiencies in the existing system. The hydraulic model was also utilized to evaluate the existing collection system under future 2040 flows, to size the CIP to convey future flows. The 2040 wastewater flow projections and resultant CIP are discussed further in the following sections.

## 3.2 Wastewater Flow Projections

The anticipated wastewater flows to the WWTP through the 2040 projected population of 39,552 are listed below. These do include the contractual wastewater flows from Samsung through 2026, but no other additional flows from a similarly large user or future Samsung phases.

**Table 3-2. Wastewater Flow Projections**

| Year | Projected Population | Average Dry Weather Flow (MGD) | Peak Wet Weather Flow (MGD) |
|---|---|---|---|
| 2030 | 25,400 | 2.3 | 20 |
| 2040 | 39,552 | 3.5 | 29 |

These projections indicate the City's 4.0 MGD WWTP will be adequate to serve the annual average dry weather flow from the City's population through 2040. In Chapter 305.126 of the Texas Administrative Code, in what is commonly known as the 75/90 rule, TCEQ requires that a WWTP permit holder begin planning an expansion to a WWTP when the annual average daily flow to a WWTP reaches 75% of the permitted capacity for three months in a row. Similarly, the rule requires that the permit holder begin construction on this expansion when the annual average daily flow to a WWTP reaches 90% of the permitted capacity for three months in a row.

Based on the comprehensive plan projections and future land use, it is anticipated that by 2040, the City should be under way in actively designing a wastewater treatment plant expansion, and begin construction shortly after 2040. It is anticipated that the City's existing WWTP site will provide adequate space for a 1-2 MGD expansion; further expansions would likely require additional land.

## 3.3 Future System Analysis

These future flows were loaded into the hydraulic model to evaluate where improvements in the existing system may be necessary to accommodate the growth that is anticipated by 2040. The flows were spatially allocated according to the comprehensive plan growth sectors and assumed population densities listed previously, in addition to increasing population densities in existing service areas to reflect infill growth.

The hydraulic model was utilized to identify what new wastewater projects would be required to accommodate the projected growth in areas not currently served with wastewater.

## 3.4 Wastewater System Capital Improvements Plan

Considering existing system improvements, future growth, and the selected level of service discussed previously, wastewater system improvements were developed to serve the anticipated growth until 2040. These improvements can generally be categorized as:

- Sanitary sewer upsizing, particularly in the Bull Branch interceptor.
- Condition assessment and inflow reduction.
- Installation of new wastewater gravity mains to serve growth.

The wastewater system capital improvements plan is shown in Figure 3-6. Generally, capital improvement projects were sized to fully convey peak wet weather flows from a 5-year, 6-hour storm event with no inflow reduction. The hydraulic model indicates that significant reductions in inflow and infiltration could potentially reduce the diameter and/or limits of the recommended projects. A potential inflow reduction strategy and associated modifications to the existing system capital improvements plan are described in the City of Taylor Wastewater Flow Metering and Model Calibration technical memorandum, which is attached as Appendix A.

In addition, Taylor is currently in the process of updating the engineering manual and associated infrastructure design sizing criteria. A future update to this master plan will consider Taylor's selected design criteria and any associated changes in proposed CIP project sizing.


FIGURE 3-6. WASTEWATER COLLECTION SYSTEM 20-YEAR CAPITAL IMPROVEMENTS PLAN

An opinion of probable construction cost (OPCC) was estimated for each project based upon conservative, planning level unit costs as well as an allowance for professional services (survey, geotechnical investigation and engineering design) and contingency for unknowns.

The OPCCs are for collection mains and do not include collection infrastructure within individual subdivision developments. The linear footage for each project is based on an assumed, approximate alignment that will be refined during design. The total length and/or diameter of some projects could potentially be reduced with successful inflow reduction. This is discussed further in the City of Taylor Wastewater Flow Metering and Model Calibration technical memorandum, which is attached as Appendix A

The summary of the Wastewater System 5-year CIP, 10-year CIP and 20-year CIP OPCCs are shown in Table 3-3, Table 3-4 and Table 3-5, respectively. Generally, the projects are listed in order of priority.

The 5-year CIP includes projects that are generally recommended to address existing system issues, and the 10-year and 20-year CIP include projects that are recommended to serve the City's projected growth. The initial I&I investigation and reduction program shown in the 5-year CIP includes more extensive field monitoring and testing, as well as additional hydraulic modeling and remediation measures to reduce I&I. In addition, including an annual operating cost to continue I&I reduction efforts, such as the City's ongoing smoke testing program, is also recommended. This could be in the range of $100,000 to $200,000 per year, with the scope of these efforts determined through data gathered during the initial detailed investigation.

The 10-year CIP generally includes new or upsized mains where the more immediate growth is anticipated. However, as growth patterns change, the CIP should be re-evaluated to determine if the timing for projects should be updated. In addition, the 10-year CIP projects are generally located near flow meter locations. While the flow metering provides greater confidence in the hydraulic model results of the entire collection system, it is most accurate near the monitoring locations. In addition to monitoring growth patterns, monitoring flows in locations near the 20-year CIP projects would allow the model accuracy to be improved in these locations, to re-evaluate their priority.

**Table 3-3. Wastewater Collection System 5-year CIP Summary**

| CIP ID | Description | Total OPCC (2023 $) |
|---|---|---|
| 1 | Upsize existing 15" and 18" along Bull Branch to 24", from Main St. to E 7th St. | $4,900,000 |
| 2 | Upsize existing 15" and 18" along Bull Branch to 30", from E 7th St. to Robinson Park | $2,800,000 |
| 3 | Upsize existing 12" and 15" along Bull Branch to 18", From W. Lake Dr. to W. 12th St. | $1,700,000 |
| 4 | I&I Investigation and Reduction & Condition Repairs | $2,000,000 |
| | **Total 5-year CIP** | **$11,400,000** |

**Table 3-4. Wastewater Collection System 10-year CIP Summary**

| CIP ID | Description | Total OPCC (2023 $) |
|---|---|---|
| 5 | Decommission Airport Lift Station & install new gravity 24" main | $7,700,000 |
| 6 | Upsize interceptor to WWTP to 48" | $8,800,000 |
| 7 | New 10" gravity main along Justin Lane | $1,900,000 |
| 8 | Upsize gravity main along Gravel Pit Road to 12" | $1,200,000 |
| 9 | New 12"-18" gravity main along Gravel Pit Road | $3,900,000 |
| 10 | Upsize upstream portion of Bull Branch interceptor to 18" | $3,500,000 |
| 11 | Upsize main along T.H. Johnson to 18" | $2,000,000 |
| 12 | Upsize gravity main to Rice's Crossing Lift Station to 18" | $2,000,000 |
| 13 | Upsize force main from Rice's Crossing Lift Station to 12" | $1,200,000 |
| 14 | Expand Rice's Crossing Lift Station | $3,500,000 |
| 15 | New 10" gravity main to serve development in West Taylor | $1,500,000 |
| 16 | New 10" gravity main to serve development in Southwest Taylor | $1,300,000 |
| 17 | Upsize gravity main near E. Rio Grande St. to 18" | $3,700,000 |
| 18 | Upsize gravity main near S. Doak St. to 15" | $3,500,000 |
| 19 | New 8"-12" gravity main near Mariposa St. & E. 4th St. | $2,400,000 |
| 20 | New 10" gravity main to serve development in Southeast Taylor | $1,800,000 |
| 21 | New 10" gravity main to serve development in Southwest Taylor | $1,100,000 |
| | **Total 10-year CIP** | **$51,000,000** |

**Table 3-5. Wastewater Collection System 20-year CIP Summary**

| CIP ID | Description | Total OPCC (2023 $) |
|---|---|---|
| 22 | New 10" gravity main to serve development in northwest Taylor | $1,500,000 |
| 23 | New 12" gravity main to serve development in west Taylor | $1,600,000 |
| 24 | New 10" gravity main to serve development in southwest Taylor | $1,200,000 |
| 25 | New 10" gravity main to serve development in west Taylor | $900,000 |
| 26 | New 10" gravity main to serve development in northwest Taylor | $1,100,000 |
| 27 | Upsize gravity main near RR & Old Thorndale to 12"-15" | $2,000,000 |
| 28 | Upsize gravity main to 18" | $1,900,000 |
| 29 | Upsize gravity main near W. Rio Grande St. to 18"-24" | $2,900,000 |
| 30 | Upsize gravity main near S. Edmond St. to 24" | $2,700,000 |
| 31 | Upsize gravity mains along Mills, Edmond and W. 3rd St. to 12"-24" | $9,800,000 |
| 32 | Upsize gravity main along Sloan St. and W. 2nd St. to 15"-18" | $5,000,000 |
| 33 | Upsize gravity main along Airport Rd. to 10" | $1,100,000 |
| 34 | Upsize gravity main along south of Justin Lane to 18" | $600,000 |
| 35 | New 10" gravity main to serve development in north Taylor | $800,000 |
| 36 | New 10" gravity main to serve development in northwest Taylor | $1,100,000 |
| 37 | New 10" gravity main to serve development in northwest Taylor | $1,600,000 |
| 38 | Upsize gravity main near FM 112 to 24" | $3,400,000 |
| 39 | Upsize gravity main near E. 2nd St. to 15" | $2,200,000 |
| 40 | Upsize gravity main near W. 2nd St. to 18"-24" | $2,600,000 |
| 41 | Upsize gravity main south of City Cemetery to 18" | $900,000 |
| 42 | Upsize gravity main near Royal St. to 12" | $1,100,000 |
| 43 | Upsize gravity main to 10" | $700,000 |
| 44 | New 10" gravity main to serve development in Southeast Taylor | $3,800,000 |
| 45 | New 10" gravity main to serve development in Southeast Taylor | $1,100,000 |
| 46 | New 12" gravity main to serve development in Southeast Taylor | $1,100,000 |
| 47 | New 10" gravity main to serve development in Southeast Taylor | $1,100,000 |
| 48 | New 10" gravity main to serve development in south Taylor | $1,100,000 |
| 49 | New 10" gravity main to serve development in northeast Taylor | $800,000 |
| 50 | New 10" gravity main to serve development in northeast Taylor | $400,000 |
| 51 | Upsize gravity main near Howard St. to 15" | $2,100,000 |
| 52 | Upsize gravity main near Old Granger Rd. to 10" | $2,100,000 |
| 53 | Wastewater Treatment Plant Expansion Design | $2,000,000 |
| | **Total 20-year CIP** | **$62,300,000** |

The OPCCs presented are considered Class 5 estimates as defined by the Association for the Advancement of Cost Engineering (AACE) Recommended Practice No. 18R. Class 5 estimates are provided at a project definition level of 0 to 2%, with a project accuracy range of -50% to +100%. The costs presented are in 2023 dollars and have not been escalated into the future.

With these improvements in place, the level of surcharging predicted by the hydraulic during 2040 peak wet weather flows are shown in Figure 3-7. With these improvements, no sanitary sewer overflows are predicted. The level of surcharging in the interceptor to the WWTP is highly dependent on the operation of the WWTP influent pumps, and the amount of backwater created. The results in Figure 3-7 are based on the current WWTP pumps and operation. Modifications to the pump operation could reduce the backwater and resultant surcharging in the interceptor.

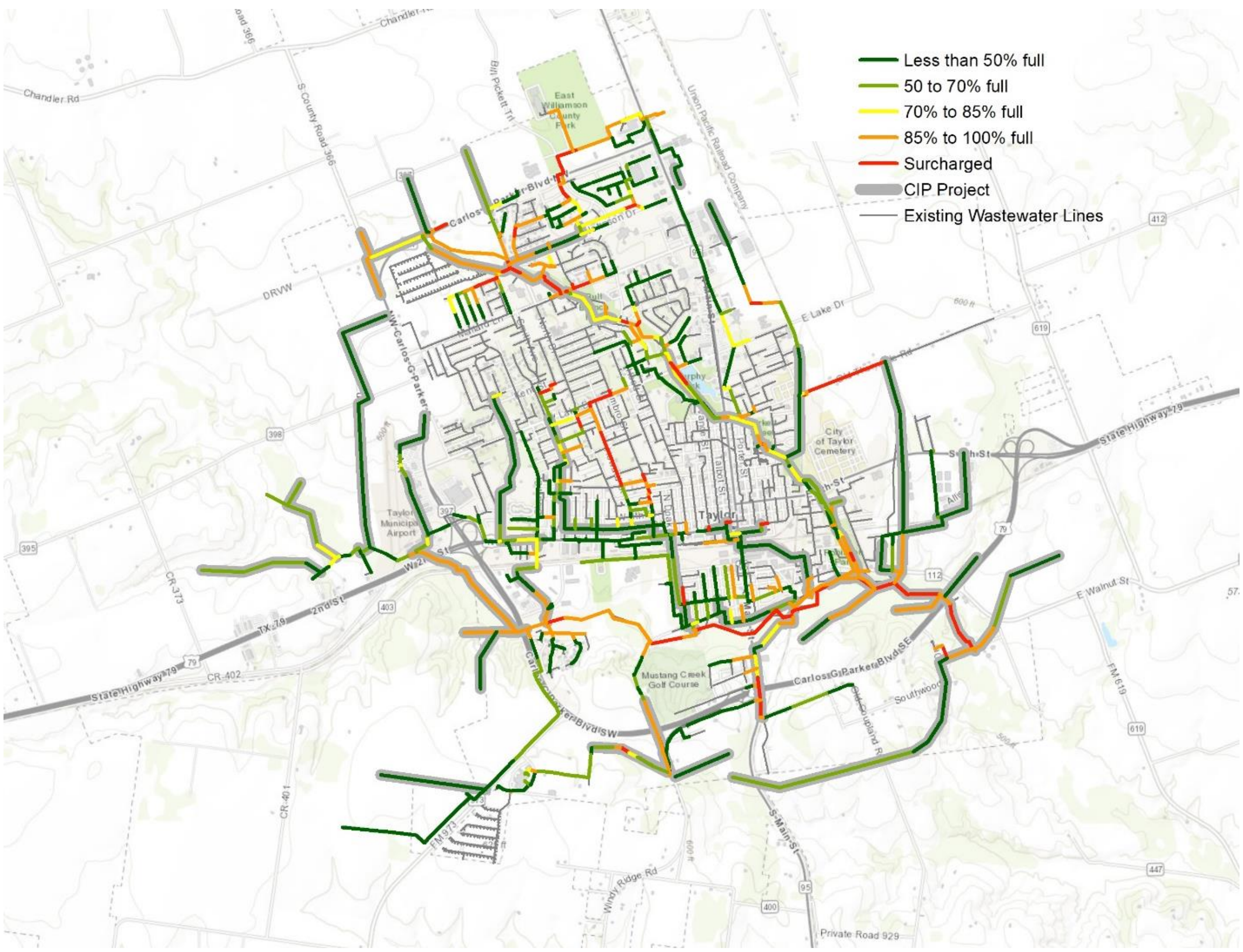



**Figure 3-7. Model-Predicted Level of Surcharging in Future System During a 5-year, 6-hour Storm**

## 3.5 Wastewater Operation and Maintenance Recommendations

In addition to the CIP recommended to improve the wastewater system performance, continued enhancements to the ongoing operation and maintenance of the collection system can further improve performance.

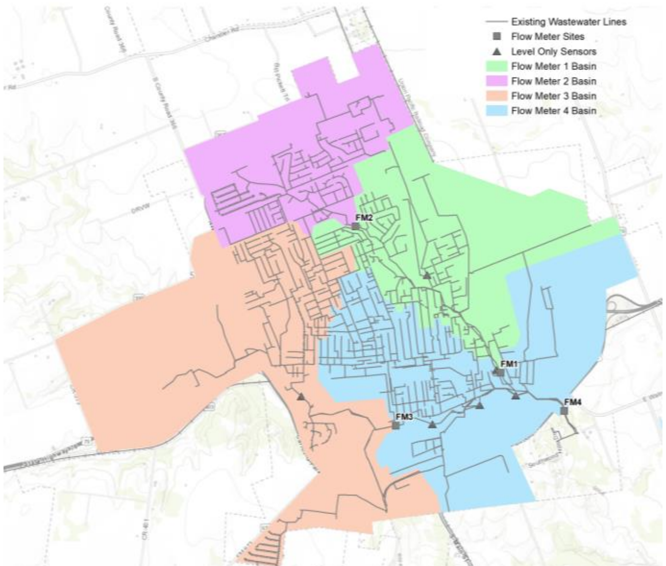
Categorizing the collection system into smaller, distinct, geographic maintenance areas would allow for future development of a Capacity Management, Operation and Maintenance (CMOM) program. These maintenance areas could be prioritized according to likelihood of failure (LoF), based on available data such as pipe age and material, and any prior inspections. They can also be prioritized according to consequence of failure (CoF), based on location and potential impacts to the environment and the public.

This LoF and CoF prioritization would help identify portions of the system with the highest risk associated with a failure, that can be used to prioritize where further field inspection and cleaning efforts would be most beneficial to improve system performance. Many of these condition assessment efforts would also dovetail with ongoing inflow & infiltration reduction efforts.

Potential recommendations to assess and improve the condition and performance of the wastewater system include:

- Conducting visual field inspections of manholes in creek beds, especially after a major rain event, to observe and repair any signs of damage from flooding.
- Conducting CCTV inspections with crews certified in pipe and manhole inspections from the National Association of Sewer Service Companies (NASSCO). Planning for ongoing pipe and manhole repair and rehabilitation based on the CCTV inspection findings.
- Continuing to perform annual smoke testing to identify pipe defects.
- Standardizing documentation of sanitary sewer cleaning frequency and cleaning findings, to optimize planned cleaning schedules.
- Append manhole survey to continue to obtain information on pipe invert elevation, pipe material and condition to continually update the City’s GIS data and hydraulic model.

# Appendix A.




# City of Taylor Wastewater Flow Metering and Model Calibration

*Taylor, Texas*
March 16, 2023


Heather Lindner
3.16.2023

Prepared by HDR Engineering, Inc.
Texas Registered Engineering Firm No. F-754

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# Tables



# Figures

## Appendices

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# 1 Executive Summary

A hydraulic model of the City of Taylor's wastewater collection system was developed based on the City's GIS information on pipe location and diameter, as well as an elevation survey of selected manholes on larger diameter mains. The hydraulic model indicated potential capacity constraints, especially along the Bull Branch interceptor, that could potentially create sanitary sewer overflows during rain events.

To verify these conditions along Bull Branch, as well as update and calibrate the City's hydraulic model, the City worked with HDR and Hach Flow to install a series of rain gauges, flow meters, and level only sensors in selected areas of the wastewater collection system, as shown in Figure 1-1.

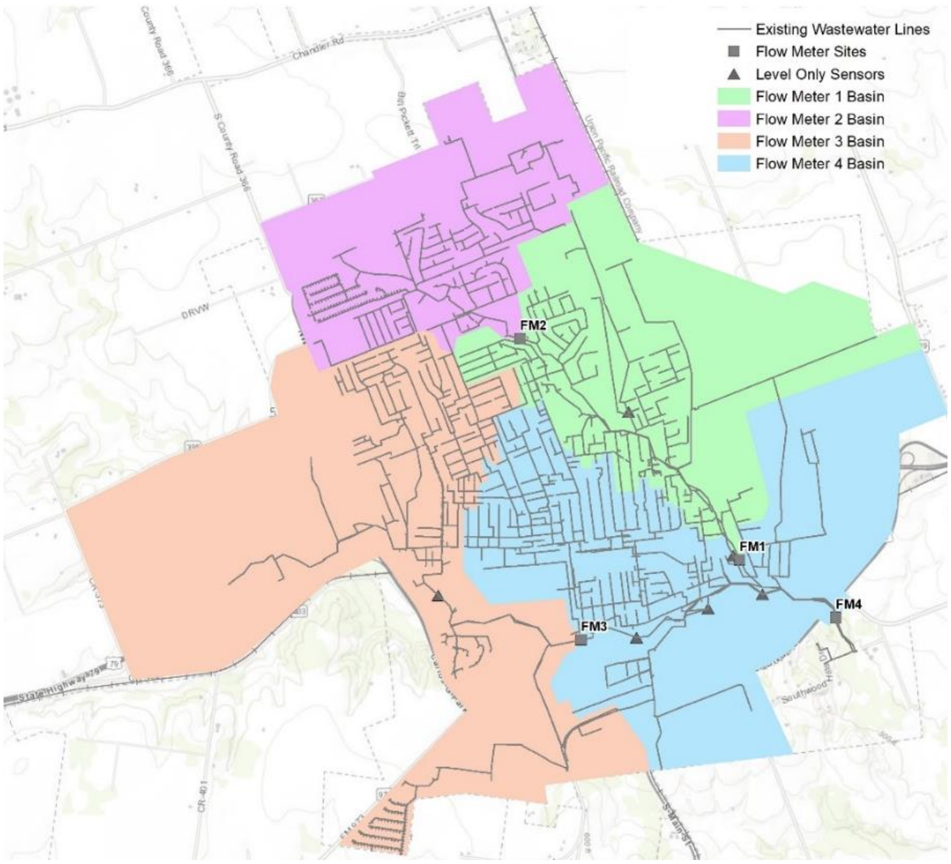



**Figure 1-1. Flow Meter Locations**

The flow metering data was used to adjust parameters in the hydraulic model to better simulate observed flows. For dry weather flows, this involves adjusting populations, gallons per person per day, and the diurnal patterns that show the variation in flows over a typical day. For wet weather flows, this involves adjusting parameters that allow rainfall derived inflow and infiltration (RDII) into the sanitary sewer system. The dry weather calibration involved numerous challenges with a complex network of parallel pipes, suspected cross connections, conflicting data, and flow meter data quality. Overall, the model is well calibrated and errs on the conservative side for system capacity analysis and capital improvement planning. The results of the adjustments made during the model calibration are detailed in Section 4 of this report.

## 1.1 Rainfall Derived Inflow and Infiltration Investigation and Reduction

The flow meter data indicates the presence of RDII into Taylor's wastewater collection system. RDII is an increase in sanitary sewer flows that occur during and after a rainfall event:

- Inflow is generally from point sources, such as manhole covers or damaged manhole walls above grade, that creates a fast, sharp increase in peak flows that quickly recedes after the rainfall event.
- Infiltration is generally from pipe or manhole defects and cracks, that creates a slow increase in sanitary sewer flow that can remain elevated for days following a rainfall event.

RDII, and particularly inflow, can take up a majority of the sanitary sewer capacity, and is therefore often attributed as a major cause of capacity-driven sanitary sewer overflows (SSO).

The calibrated hydraulic model indicates the Bull Branch interceptor is significantly surcharged and possibly overflowing during rainfall events. The flow meters in the Bull Branch basin indicate rates of high RDII, and that inflow specifically is contributing to the increase in peak flows. If the interceptor capacity is increased by installing larger pipes, without any reduction in RDII, the larger pipes will fill up with RDII and not actually increase capacity for conveying wastewater. This requires not only larger, cost-prohibitive infrastructure to convey both the wastewater and RDII, but can also have a significant impact to the downstream WWTP. Increased flows to the WWTP are costly to treat, can overwhelm the hydraulic capacity of the WWTP and can make treatment less effective. Therefore, the recommended solution is a combination of reducing inflow and increasing interceptor capacity.

For inflow reduction to be most effective, an investigation to identify the sources of inflow is recommended. A potential strategy to further investigate the sources of RDII, and particularly inflow, is shown in Figure 1-2. An RDII study and reduction effort, including some associated pipe and manhole condition repair, is a project in the City's water and wastewater master plan, budgeted at a total of $2,000,000. Of this $2,000,000, the investment in an RDII study and reduction effort is estimated at up to $1,500,000.

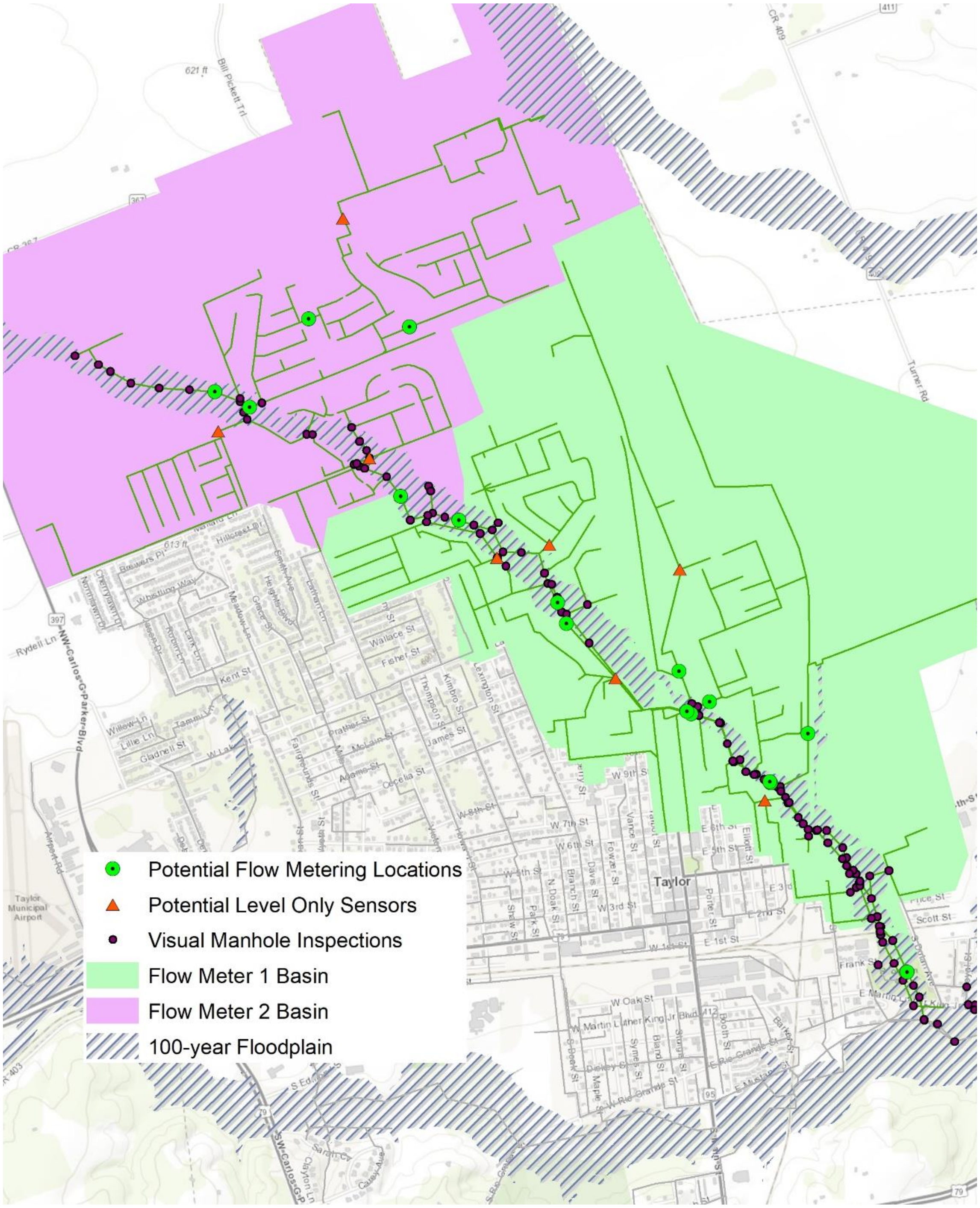



**Figure 1-2. Possible Inflow Investigation Strategy**

The sources and locations of inflow discovered during the investigation will provide information on the most effective remediation method to reduce inflow. However, potential strategies to reduce inflow include:

- Replacing manhole covers that become submerged with more watertight manholes, or elevating manholes

- Visually inspecting the manholes along interceptor frequently, especially manholes adjacent to the creek, for missing covers or creek inflow and repairing defects
- Eliminating any inadvertent connections with storm sewers or downspouts discovered through dye testing or smoke testing
- Public education regarding open sanitary sewer cleanout caps

The level of inflow reduction achieved will have an impact on the required capital improvement project to increase pipe capacity. Section 5 of this report includes maps that show the model predicted improvement in system performance with various levels of inflow reduction, as well as what capital improvements would be required if the targeted inflow reduction is achieved.

If 30% inflow reduction can be achieved with a $1,500,000 investment, the resultant capital improvement projects to address existing system capacity constraints are estimated to cost approximately $4,000,000 to design and construct. Without any inflow reduction, this cost was estimated at approximately $9,400,000. Therefore, it is estimated that the inflow reduction efforts could result in a savings of nearly $4,000,000 in capital improvement projects.

To serve future growth, in addition to the cost of inflow reduction, approximately $9,100,000 of capital improvement projects were identified. $6,800,000 of these costs are additional projects required to serve growth, and $1,200,000 is attributed to oversizing to serve growth, making these costs impact fee eligible.

## 1.2 Next Steps

The first recommended next step is further investigation to find the source of the RDII and reduce inflow. This will not only provide information on how to best reduce the amount of RDII in the sanitary sewer system, but also valuable information that can also be used to improve the model calibration and refine the capital improvement projects. Therefore, it is recommended to proceed with the RDII investigation prior to constructing any capital improvement projects.

The RDII investigation and reduction efforts will both demonstrate the City is actively addressing the Bull Branch interceptor capacity issue, while also allowing for a refined capital improvement plan to ensure budgeted funds are allocated where they are most needed.

Preliminary design efforts can also begin on upsizing approximately 2,550 linear feet of the most downstream section of the Bull Branch interceptor to 24 inches in diameter. This would include a topographic survey, geotechnical investigation, and definition of horizontal and vertical pipe alignments. If the results of the RDII investigation and inflow reduction efforts and subsequent model update, change the recommended pipe diameter, this can be adjusted prior to final design and construction.

The City has budgeted approximately $3,300,000 to improve the Bull Branch interceptor. It is recommended that these funds be used to investigate and reduce inflow, with the remaining funds used to proceed with increasing this diameter of the downstream portion of the interceptor.

# 2 Introduction

A hydraulic model of the City of Taylor's wastewater collection system was developed based on the City's GIS information on pipe location and diameter, as well as an elevation survey of selected manholes on larger diameter mains. The hydraulic model indicated potential capacity constraints, especially along the Bull Branch interceptor, that could potentially create sanitary sewer overflows during rain events. To verify these conditions along Bull Branch, as well as update and calibrate the City's hydraulic model, the City worked with HDR and Hach Flow to install a series of rain gauges, flow meters, and level only sensors in selected areas of the wastewater collection system.

The flow metering data and the updated hydraulic model were used to investigate potential areas and the scale of inflow and infiltration into the collection system. The model results were used to estimate system capacity and identify potential capacity constraints. The existing system flows and projected 2040 flows, both for dry weather and wet weather conditions, were simulated throughout the collection system to identify infrastructure improvements that may be required to meet flow projections. These improvements were summarized in a wastewater capital improvements plan as part of the City's water and wastewater master plan.

The purpose of this report is to summarize the flow metering data and document the update and calibration of the hydraulic model used to prepare the City's wastewater CIP recommendations. The initial model development, flow allocation and system analysis are documented in the City's Water and Wastewater Master Plan technical memorandum.

# 3 Data Collection

Four flow meters (measuring both flow depth and velocity), six meters measuring only flow depth, and one rain gauge were installed in the City of Taylor's collection system for approximately six months, from January to July of 2022. The rainfall and flow meter data collected were evaluated and determined to be sufficient for model calibration.

The flow meter data does indicate the presence of rainfall derived inflow and infiltration (RDII) into Taylor's wastewater collection system. RDII is an increase in sanitary sewer flows that occur during and after a rainfall event:

- Inflow is generally from point sources, such as manholes, that creates a fast, sharp increase in peak flows that quickly recedes after the rainfall event.
- Infiltration is generally from pipe or manhole cracks, that creates a slow increase in sanitary sewer flow that can remain elevated for days following a rainfall event.

RDII, and particularly inflow, is often attributed as a major cause of capacity-driven sanitary sewer overflows (SSO). Additional flow meters are suggested as a result of these analyses for further, more detailed RDII analysis that could pin-point the sources of RDII and evaluate potential reduction strategies. It is often more cost effective to reduce inflow than infiltration, and also has a greater impact on peak flow reduction.  This RDII study and reduction plan is a budgeted project in the City's water and wastewater master plan.

## 3.1 Rain Gauge Location

One rain gauge was installed near the Public Works Department, as shown in Figure 3-1.

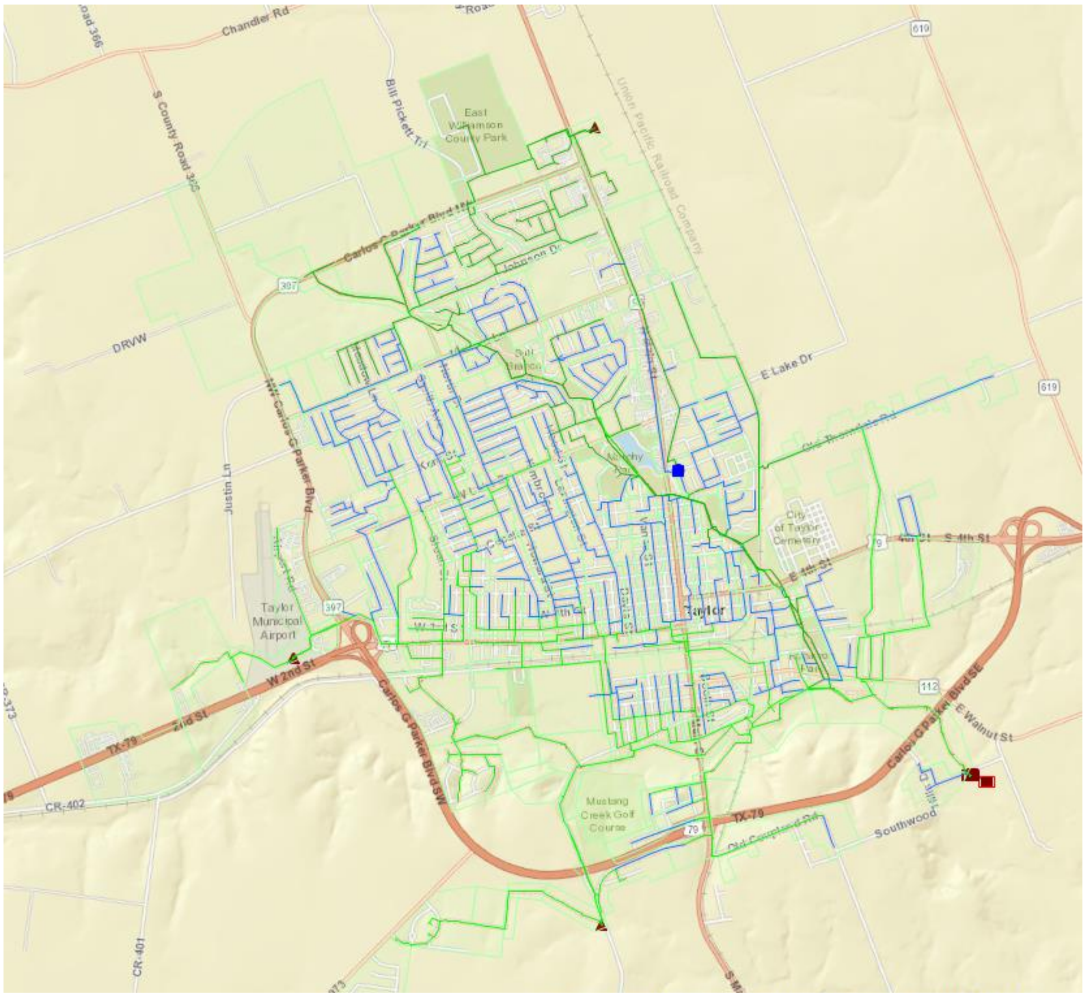



**Figure 3-1.  Rain Gauge Location**

The quality of the rainfall data does have some influence on the results, as storms are not spatially uniform across the project basin. Rain gauge data was supplemented with radar data to understand how rainfall events may have varied across the collection system basin.

## 3.2 Flow Metering Locations

Four full flow meters measuring both flow depth and velocity, as well as six additional meters measuring only flow depth, were installed at the locations shown in Figure 3-2.

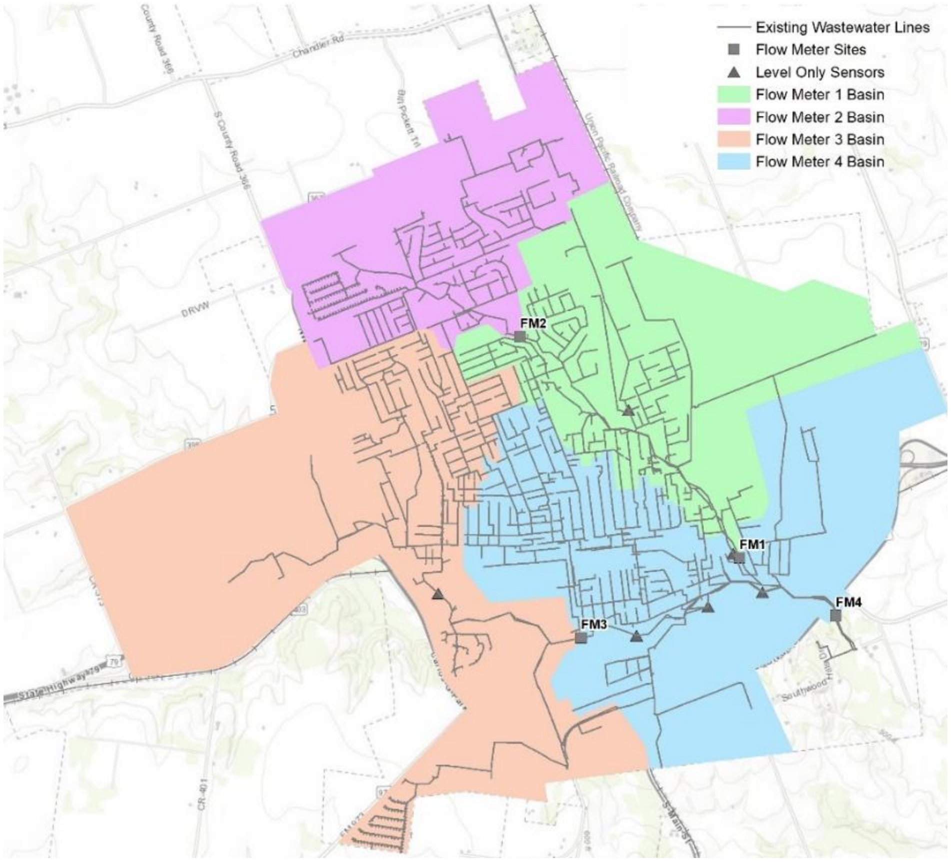



**Figure 3-2. Flow Meter Locations**

## 3.3 Model Network Updates

The partial manhole survey data that was conducted before the initial hydraulic model was developed included azimuths of the pipes entering survey manholes. This information, along with careful review of aerial photography, the flow metering data provided information that was used to update the network of gravity mains in the Bull Branch basin. The downstream portion of the Bull Branch interceptor includes parallel lines with some cross connections between the lines. Some assumptions were made to fill in gaps where uncertainty remains between GIS, the manhole survey, and flow metering data. These assumptions on the network are shown in more detail in Appendix A. Additional field investigations to locate sources of inflow, described further in this report, would also provide information that can reduce remaining uncertainty in the model network.

# 4 Hydraulic Model Calibration

The model calibration process creates a representative indication of system performance and begins with dry weather calibration. The original development of the hydraulic model network and flow allocation is described in more detail in the City of Taylor Water and Wastewater Master Plan Technical Memorandum.

This technical memorandum describes the process to update and calibrate the model based on the observed flow metering data. The model was calibrated against the observed flow data collected at all flow metering locations, during dry and wet weather conditions.

## 4.1 Dry Weather Flow

Dry weather flow (DWF) represents the flow in the system during dry periods, when the majority of the flow contribution is from the system's customers. The DWF at a given time is generally determined by the following formula:

$$Q_{DWF} = (Population * GPCD) * Diurnal\ Profile\ Factor + Baseflow$$

The population was determined as described in Section 3 of the Water and Wastewater Master Plan Report. The remaining three parameters are the factors that are adjusted during DWF calibration:

- GPCD (average customer loading rate, in gallons per capita per day)
- Trade Flow (commercial and industrial loading)
- Diurnal Profile Factor (normalized diurnal loading variation)
- Baseflow (constant inflow into the system)

To determine the dry weather diurnal profiles, flow meter data was analyzed for each metershed. These profiles were normalized to make them dimensionless and were then used to represent the flow peaking factors for each area. A typical diurnal profile for residential and commercial was created from the metershed profiles for each metershed. The profiles were assigned to each subcatchment based on the subcatchment land use and the area's general flow meter characteristics. A subcatchment's land use was designated as residential if the majority of the parcels within the subcatchment are residential. A subcatchment's land use was designated as commercial if the majority of the parcels within the subcatchment are residential.

During the dry weather calibration process, the diurnal profiles in each metershed were adjusted in two ways:

- Adjusting the shape and timing of peaks, to account for attenuation of peak flows in the metersheds further downstream
- Adjusting the diurnal factor during peak flow to match observed peak flows

The dimensionless diurnal profiles were applied to average day flows to mimic daily flow variations observed in the sewer system. The initial average daily flow loading in GPCD was determined for each diurnal profile, and then adjusted to match the flow meter data over the dry weather calibration period. The residential and commercial diurnal profiles used, for both weekday and weekend conditions, are shown in Figure 4-1. The top row represents the residential diurnal pattern, and the bottom row represents the commercial diurnal pattern.

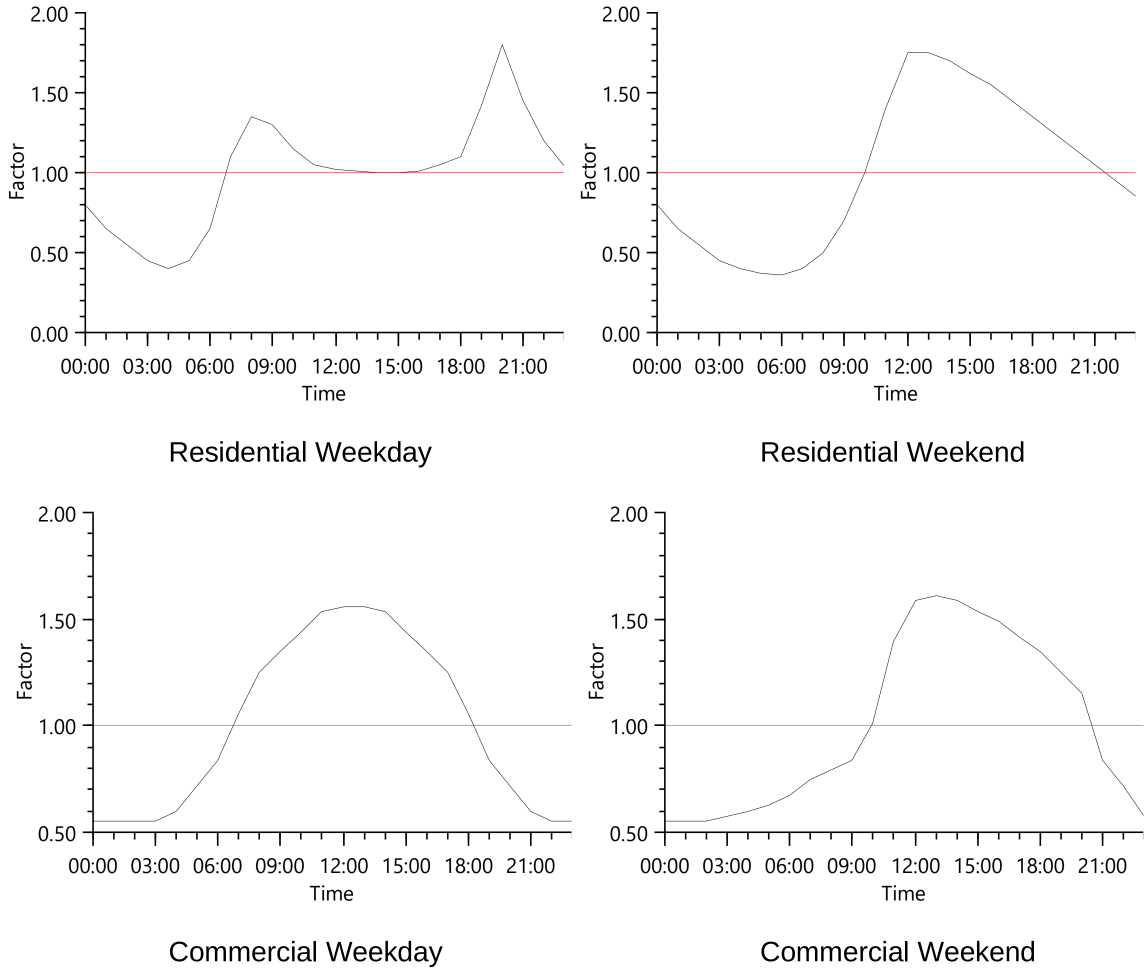



**Figure 4-1. Modeled Diurnal Profiles**

## 4.1.1 Dry Weather Flow Calibration Goals

To consider a model calibrated to DWF, the following calibration guidelines were considered. These tolerances are industry standards, based on guidelines from the Chartered Institution of Water and Environmental Management (CIWEM), formerly known as the Wastewater Planning Users Group (WaPUG).

- The model-predicted and observed DWF hydrographs should be similar in shape and magnitude.
- The difference between the model-predicted and observed peak flow should be within ±10%.

- The difference between the model-predicted and observed total volume of flow should be within ±10%.

A typical calibrated hydraulic model is expected to represent DWF for an average dry day across a large range of dates. Because observed days typically vary from the average day, even a well-calibrated model may not accurately represent every dry day in the observed period.

March 13-19, 2022 was chosen as the dry weather flow calibration period, as it was a consistently dry period with good flow meter data capture. The final GPCDs following the dry weather calibration process are shown in Figure 4-2.

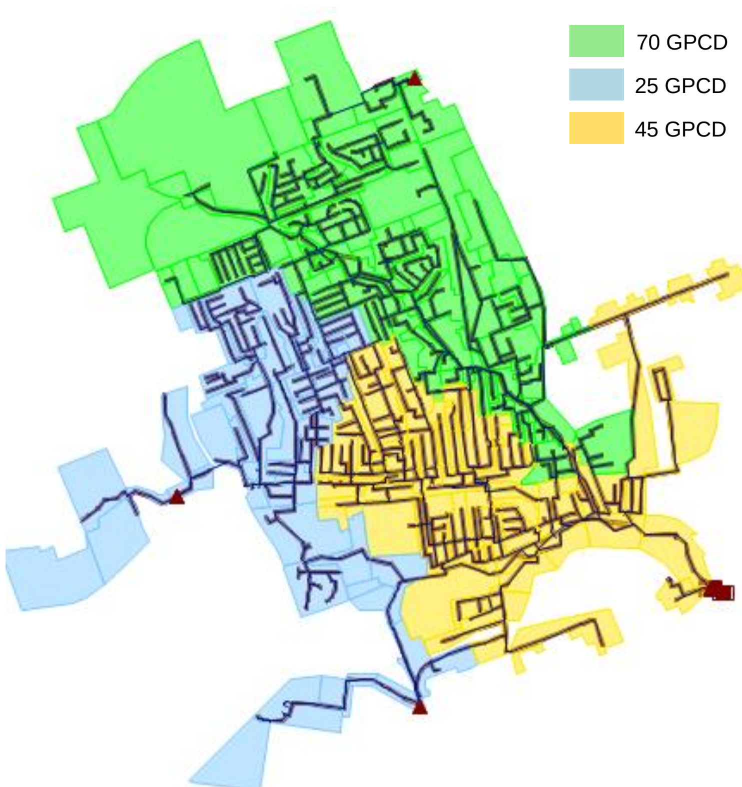



**Figure 4-2. Calibrated GPCDs**

A summary of the DWF calibration and statistics are shown in Table 4-1 and Table 4-2. The observed peak flow values are based on an average of the maximum flow during the dry weather weeks of January 26-31, March 11-21 and May 11-21. The tables are followed by an explanation of the meters that are not calibrated to dry weather calibration tolerance goals.

### Table 4-1. Dry Weather Calibration Summary (Peak Flow and Volume)

| | Peak Flow (MGD) | | | Volume (MGD) | | |
|---|---|---|---|---|---|---|
| **Flow Meter (FM) ID** | **Observed** | **Modeled** | **Difference (Modeled - Observed)** | **Observed** | **Modeled** | **Difference (Observed – Modeled)** |
| FM1 | 1.19 | 1.33 | +4.6% | 0.76 | 0.81 | +6.5% |
| FM2 | 0.71 | 0.67 | +11.6% | 0.41 | 0.40 | +1.6% |
| FM3 | 0.37 | 0.78 | +113.2%[1] | 0.21 | 0.23 | +9.1% |
| FM4 | 1.88 | 2.34 | +24.3%[2] | 1.23 | 1.42 | +15.6%[2] |

Notes:

1. The flow meter appears to be attenuating the variation in flow from the upstream lift stations (Airport and Rice's Crossing). At these low dry weather flows, the model is showing a higher variation in peak flow when the upstream lift stations are discharging.
2. The flow meter is influenced by backwater from the WWTP headworks operation. The dry weather model utilized the more conservative potential operation of the WWTP headworks.

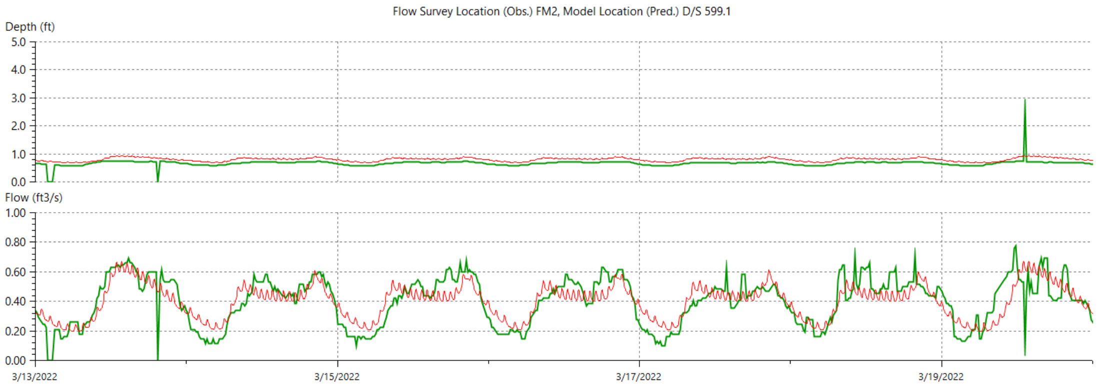
### Table 4-2. Dry Weather Calibration Summary (Flow Depth)

| | Flow Depth (feet) | | |
|---|---|---|---|
| **Flow Meter (FM) or Level Meter (LM) ID** | **Observed** | **Modeled** | **Difference (Modeled - Observed)** |
| FM1 | 0.92 | 0.60 | -0.32 |
| FM2 | 0.73 | 0.94 | +0.21 |
| FM3 | 0.30 | 0.38 | +0.08 |
| FM4 | 0.56 | 0.81 | +0.25 |
| LM1 | 0.27 | 0.20 | -0.07 |
| LM2 | 0.36 | 0.39 | +0.03 |
| LM3[1] | 1.16 | 0.26 | -0.90 |
| LM4[2] | 0.60 | 0.80 | +0.21 |
| LM5[3] | 0.39 | 0.51 | +0.13 |
| LM6[4] | 0.41 | 0.57 | +0.16 |

Notes:

1. Level meter readings are erratic. A manual estimate of the maximum depth was taken.
2. The maximum level recorded was about 7.8 to 8.0 feet, likely due to the depth at which the sensor was mounted in the manhole.
3. The maximum level recorded was about 6.9 to 7.1 feet, likely due to the depth at which the sensor was mounted in the manhole.
4. The maximum level recorded was about 7.8 feet, likely due to the depth at which the sensor was mounted in the manhole.

Calibration plots were developed to show the full comparison between model and observed dry weather flows to demonstrate how the model is predicting the observed flows. A sample DWF calibration plot for Flow Meter 2 is shown in Figure 4-3, which compares the observed data (green) to model predicted data (orange). Appendix B includes the DWF calibration plots for all metersheds. The representative dry weather time period selected for the plots was March 13-19, 2022.




| | Depth | | Flow | | |
|---|---|---|---|---|---|
| | Min (ft) | Max (ft) | Min (MGD) | Max (MGD) | Volume (US Mgal) |
| Observed | 0.000 | 2.696 | 0.000 | 0.776 | 2.809 |
| ...l GMI (Guage only) Full>DWF | 0.682 | 0.938 | 0.197 | 0.672 | 2.806 |

**Figure 4-3. Sample DWF Calibration Plot (Flow Meter 2)**

## 4.2 Wet Weather Flow

Wet weather flow (WWF) represents the additional flow in the sewer system caused by rainfall. Modeling WWF involves using a hydrology method to transform rainfall into system inflow. The hydrology method selected for this model is the Wallingford Variable Percentage Runoff method. This method uses three types of surfaces – fast impervious, slow impervious and pervious – to represent the RDII. This method takes into account the changing antecedent moisture conditions of the subcatchments and modifies the rainfall contribution at pervious surfaces accordingly. As the catchment gets wetter, more flow is contributed to the system as the capacity of the pervious surface to soak up the water diminishes.

The surface model parameters modified for WWF calibration include:

- Contributing Area
- Percentage of Impervious Area
- Land Use Surface Type
- Initial Rainfall Losses
- Runoff Routing Value
- Runoff Coefficient for Impervious Surfaces
- Surface Soil Depth for Pervious Surfaces

To account for inflow and infiltration into the system, the selected RDII method combines three unit hydrographs into a composite unit hydrograph to define the amount of runoff and travel time to get into the system. Each of the three unit hydrographs is defined by parameters that were adjusted during model calibration until observed and model-predicted flows matched within a desired tolerance. The three hydrographs correspond to the following major components:

- Fast response: stormwater that has a direct path to get into the system over impervious surfaces, such as roads and driveways. This hydrograph creates the initial peak observed in the flow meter data.
- Medium response: stormwater that has an indirect path to get into the system such as parking lots and roofs and have to travel overland to a sewer access point. These flows do not arrive as quickly as the fast response sources and are somewhat attenuated.
- Slow response: stormwater that has an indirect path to get into the system, usually across permeable areas increasing the time of concentration. Flows from these sources can take several hours or more to reach the sanitary sewer system after the rainfall event subsides and are typically significantly attenuated.

Total subcatchment areas are variable in size, land use and percentage utilized or developed. To account for the inherent differences in contributing area, the WWF contributing area was set to include a 50-foot buffer along every pipe in the GIS. This buffered area was then used with the impervious cover to estimate the percentage of fast and slow impervious area, with the remainder of the buffered area set to pervious area. The percentage of fast and slow impervious areas within the buffered area were primarily determined based on aerial photography, to estimate the amount of paved area.  In this way, the contributing area was standardized, effectively removing it as a calibration parameter. The resulting primary variables used were the runoff coefficients for each area, which enables a comparison of runoff coefficients across subcatchments.

The RDII volume was estimated using the combined runoff from the three surfaces. With volume determined, the range of the routing values for the surface parameters was developed to match peaks and then best capture the tail of the RDII response. Initial loss and the above parameters were modified to produce the desired response in the system for each calibration and then verified using the verification event.

### 4.2.1 Wet Weather Flow Calibration Goals

Overall, the goal of WWF calibration is to have confidence that the resulting model will represent the observed data over a large range of events. This target is not always fully achievable because a single RDII response cannot always represent the variety of observed storms and system conditions. Therefore, to calibrate the model for WWF, the following calibration guidelines were used.  These tolerances are industry standards, based on guidelines from the Chartered Institution of Water and Environmental Management (CIWEM), formerly known as the Wastewater Planning Users Group (WaPUG).

- The model-predicted and observed WWF hydrographs should be similar in shape and magnitude.
- The difference between the model-predicted and observed peak flow rate should be within -15% and +25%
- The difference between the model-predicted and observed volume of flow should be within -10% and +20%

- The difference between the model-predicted and observed depth should be within ±10% or ±0.33 ft if not surcharged or -0.33 ft to +1.64 ft if surcharging.

If a metershed approximates these tolerances for two out of the three calibration and verification events, it is considered well calibrated. This is because data is much more limited for wet weather, and meters often drop out or provide data that is not of ideal quality.

### 4.2.2 Wet Weather Flow Calibration Events

Wet weather flow calibration focuses on rainfall events and the RDII these exert on the sanitary system. While a number of wet weather events occurred during the flow monitoring period, the two rainfall events used for calibration occurred from January 31, 2022 to February 2, 2022. The rainfall depth from the January 31 event was approximately 2.4 inches, with an intensity of approximately one inch per hour. The rainfall depth from the February 2 event was approximately 3.0 inches, with an intensity of approximately 1.25 inch per hour. These were two individual storms, but the second one occurred only two days after the first. This created wetter antecedent moisture conditions during the second event, which increased both inflow and infiltration, as well as producing a longer infiltration. It can be challenging to replicate different antecedent moisture conditions in a wet weather calibration. The model’s groundwater infiltration module was utilized to help model that observed impact from two back to back rain events.

Table 4-3 and Table 4-4 present the January 31, 2022 WWF calibration statistics for the calibration and verification rainfall events, for the peak flow and total volume conditions, as well as flow depth.

**Table 4-3. WWF Calibration Summary (Peak Flow and Volume) for January 31, 2022 Event**

| | Peak Flow (MGD) | | | Volume (MGD) | | |
|---|---|---|---|---|---|---|
| **Flow Meter (FM) ID** | **Observed** | **Modeled** | **Difference (Modeled - Observed)** | **Observed** | **Modeled** | **Difference (Modeled - Observed)** |
| FM1 | 3.56 | 3.70 | +4.1% | 3.36 | 3.28 | -2.6% |
| FM2 | 1.49 | 1.35 | -7.5% | 1.45 | 1.39 | -4.2% |
| FM3 | 3.98 | 1.83 | -54.0%[1] | 1.06 | 1.04 | -1.4% |
| FM4 | 10.24 | 8.81 | -14.0% | 7.08 | 8.51 | +20.2% |

Notes:

1. The flow meter appears to be picking up intermittent backwater conditions which is released in a flush. This higher depth is generating a higher calculated flow, causing erratic readings and indicating an artificially high peak flow.

**Table 4-4. Wet Weather Calibration Summary (Flow Depth) for January 31, 2022 Event**

| | Flow Depth (feet) | | |
|---|---|---|---|
| **Flow Meter (FM) or Level Meter (LM) ID** | **Observed** | **Modeled** | **Difference (Modeled - Observed)** |
| FM1 | 4.91 | 1.53 | -3.38 |
| FM2 | 1.03 | 2.25 | +1.22 |
| FM3 | 3.03[1] | 0.94 | -2.10 |
| FM4 | 19.93 | 15.86 | -4.07[2] |
| LM1 | 0.33 | 0.30 | -0.02 |
| LM2 | 0.38 | 0.59 | +0.21 |
| LM3[3] | 0.41 | 0.39 | -0.02 |
| LM4[4] | 7.86 | 12.54 | +4.68 |
| LM5[5] | 6.91 | 7.53 | +0.63 |
| LM6[6] | 6.42 | 4.57 | -1.84 |

Notes:

1. The flow meter appears to be picking up intermittent backwater conditions which is released in a flush. This higher depth is generating a higher calculated flow, causing erratic readings and indicating an artificially high peak flow.
2. The flow meter is influenced by backwater from the WWTP headworks operation. Backwater from the WWTP is not being fully represented in the model for this event.
3. Level readings are erratic. A manual estimate of the maximum depth taken.
4. The maximum level recorded was about 7.8 to 8.0 feet, likely due to the depth at which the sensor was mounted in the manhole, and not capturing the true level of surcharging.
5. The maximum level recorded was about 6.9 to 7.1 feet, likely due to the depth at which the sensor was mounted in the manhole.
6. The maximum level recorded was about 7.8 feet, likely due to the depth at which the sensor was mounted in the manhole.

Table 4-5 and Table 4-6 present the February 2, 2022 WWF calibration statistics for the calibration and verification rainfall events, for the peak flow and total volume conditions, as well as flow depth.

**Table 4-5. WWF Calibration Summary (Peak Flow and Volume) for February 2, 2022 Event**

| | Peak Flow (MGD) | | | Volume (MGD) | | |
|---|---|---|---|---|---|---|
| **Flow Meter (FM) ID** | **Observed** | **Modeled** | **Difference (Modeled – Observed)** | **Observed** | **Modeled** | **Difference (Modeled – Observed)** |
| FM1 | 3.96 | 5.32 | +34.3%[1] | 5.22 | 5.38 | +3.0% |
| FM2 | 2.15 | 2.26 | +4.9% | 1.92 | 2.17 | +13.1% |
| FM3 | 6.11 | 3.01 | -50.7%[2] | 2.63 | 2.06 | -21.7% |
| FM4 | 13.99 | 14.06 | +0.4% | 12.58 | 14.13 | +12.3% |

Notes:

1. The flow meter data indicates there may be something in the pipes downstream of the meter, reducing velocities and restricting the calculated flow to about 4 MGD. This could be a blockage of some kind, or a difference in the downstream modeled pipe alignment. Field survey for the area was inconclusive.
2. There is suspected to be a large inflow point to the sanitary sewer from the creek upstream of the flow meter, generating large spikes in the observed peak flow.

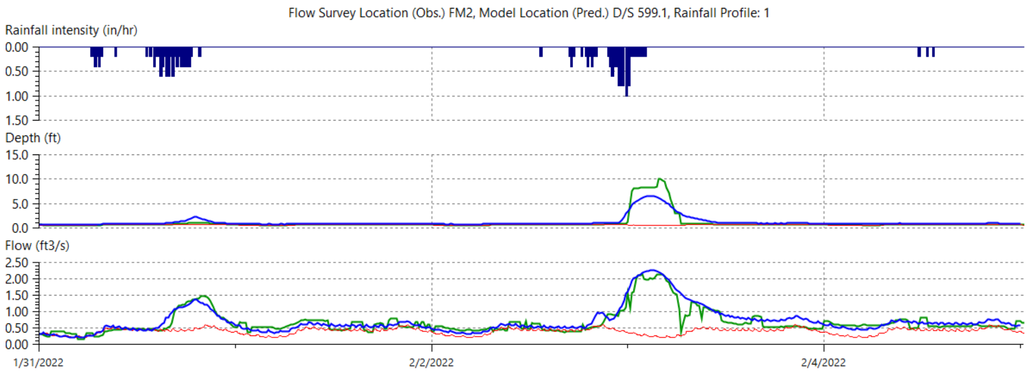
**Table 4-6. WWF Calibration Summary (Flow Depth) for February 2, 2022 Event**

| | Flow Depth (feet) | | |
|---|---|---|---|
| **Flow Meter (FM) or Level Meter (LM) ID** | **Observed** | **Modeled** | **Difference (Modeled – Observed)** |
| FM1 | 5.22 | 5.75 | +0.53 |
| FM2 | 10.10 | 6.53 | -3.57 |
| FM3 | 8.12 | 7.91 | -0.21 |
| FM4 | 19.92 | 19.82 | -0.10 |
| LM1 | 0.46 | 0.37 | -0.10 |
| LM2 | 0.39 | 5.23 | +4.85 |
| LM3[1] | 0.70 | 0.51 | -0.19 |
| LM4[2] | 8.08 | 17.81 | +9.73 |
| LM5[3] | 7.06 | 13.89 | +6.84 |
| LM6[4] | 7.72 | 11.23 | +3.51 |

Notes:

1. Level readings are erratic. A manual estimate of the maximum depth taken.
2. The maximum level recorded was about 7.8 to 8.0 feet, likely due to the depth at which the sensor was mounted in the manhole, and not capturing the true level of surcharging.
3. The maximum level recorded was about 6.9 to 7.1 feet, likely due to the depth at which the sensor was mounted in the manhole, and not capturing the true level of surcharging.
4. The maximum level recorded was about 7.8 feet, likely due to the depth at which the sensor was mounted in the manhole, and not capturing the true level of surcharging.

WWF calibration plots were developed to show the full comparison between model and observed wet weather flows to demonstrate how the model is predicting the observed flows. A sample WWF calibration plot for Flow Meter 2 is shown in Figure 4-3, which compares the observed data (green) to model predicted wet weather flows (blue) and dry weather flows (red). Rainfall is shown as dark blue lines at the top of the plots. Appendix C includes the WWF calibration plots for all metersheds.




| | Rainfall | | | Depth | | Flow | | |
|---|---|---|---|---|---|---|---|---|
| | Depth (in) | Peak (in/hr) | Average (in/hr) | Min (ft) | Max (ft) | Min (MGD) | Max (MGD) | Volume (US Mgal) |
| Rain | 5.150 | 1.000 | 0.043 | | | | | |
| Observed | | | | 0.529 | 10.100 | 0.145 | 2.149 | 3.374 |
| ...age only)>DWF | | | | 0.520 | 0.751 | 0.207 | 0.605 | 2.011 |
| ...riod Meter Data | | | | 0.694 | 6.531 | 0.211 | 2.255 | 3.551 |

**Figure 4-4. Sample WWF Calibration Plot (Flow Meter 2)**

The model can be used for further insight into system RDII by analyzing the WWF calibration parameters. These values should be reviewed with caution as they are somewhat subjective to the modeler's application of the calibration parameters in the model. Calibration can be achieved with a range of parameters; however, the number of wet weather events to which the model has been calibrated usually lowers this variability in the parameters and care has been taken to have this model to function across a broad range of events.

# 5 Conclusions and Recommended Next Steps

The main area of concern in the existing collection system identified through the flow metering and hydraulic modeling is in the Bull Branch interceptor. This interceptor ranges in size from 15-inches to 18-inches, including some intermediate bottlenecks where the diameter decreases as flow proceeds downstream. The interceptor conveys flow from a significant portion of the City, and the calibrated hydraulic model indicates it is significantly surcharged and possibly overflowing during rainfall events.

The flow metering data and model calibration indicate areas within the Bull Branch basin are exhibiting rates of high rainfall derived inflow and infiltration (RDII) entering the City's wastewater collection system. The metering data indicates that inflow specifically is contributing to the increase in peak flows. As seen in Figure 5-1 and Figure 5-2, the total flow in the wastewater lines increases sharply when rainfall begins and drops sharply after rainfall ends, which is indicative of inflow from point sources, such as manholes. The total flows do stay somewhat elevated after rainfall ends, which indicates that pipes and manhole structures in poor condition may also be causing some infiltration. However, inflow is what increases peak flows and takes up the majority of the pipe capacity.

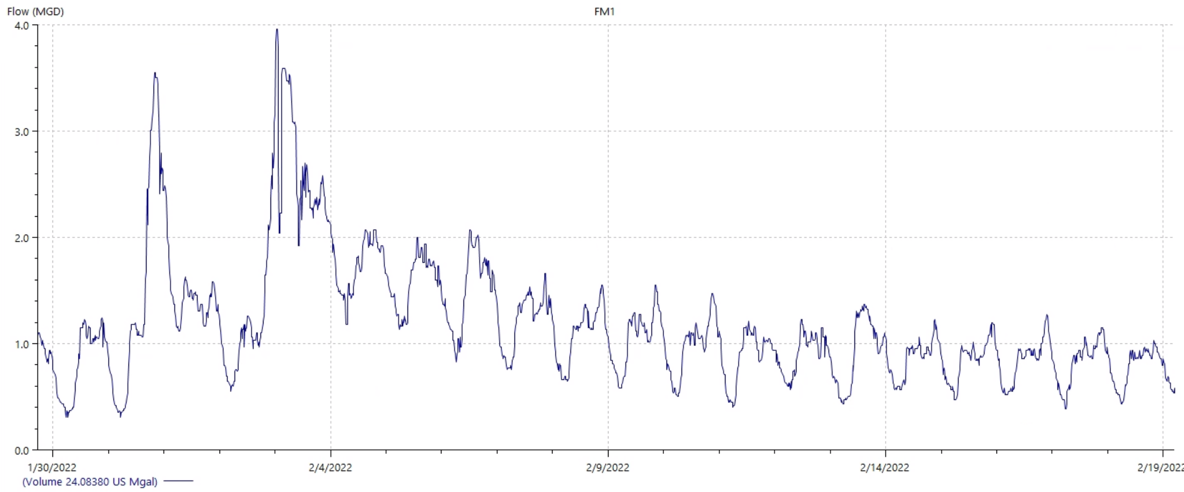



**Figure 5-1. Flow Data Collected at Flow Meter 1 During Wet Weather Events**

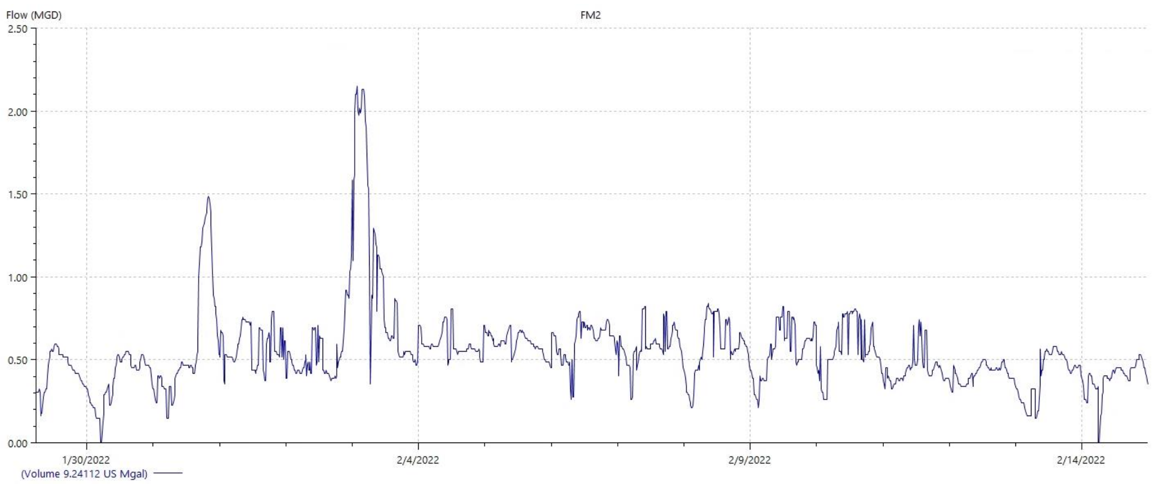



**Figure 5-2. Flow Data Collected at Flow Meter 2 During Wet Weather Events**

## 5.1 RDII Reduction

If the interceptor capacity is increased by installing larger pipes, without any reduction in RDII, the larger pipes will fill up with RDII and not actually increase capacity for conveying wastewater. This requires not only larger, cost-prohibitive infrastructure to convey both the wastewater and RDII, but can also have a large impact to the downstream WWTP. Increased flows to the WWTP are costly to treat, can overwhelm the hydraulic capacity of the WWTP and can make treatment less effective. Therefore, the recommended solution is a combination of reducing inflow and increasing interceptor capacity. For inflow reduction to be most effective, an investigation to identify the sources of inflow is recommended.

### 5.1.1 RDII Investigation

An investigation to locate potential sources of RDII would likely include a combination of the following strategies. Some strategies are focused on finding inflow sources, while others are focused on infiltration. A reduction in inflow will have the greatest impact on a reduction in sanitary sewer peak flow, and is often the higher priority from a wastewater capacity standpoint.

- Additional flow monitoring and level only manhole sensors to further pinpoint the source of the RDII already identified in meters FM1 and FM2.
- Visual inspection of approximately 150 manhole covers in the Bull Branch floodplain, and any that become submerged during any localized street flooding. Manhole covers and manhole walls above grade can leak substantial amounts of storm water into the sanitary sewer, depending on the type, or can often be missing. Deteriorated or damaged manhole walls that sit above grade can also allow direct storm water inflow into the system. This strategy would primarily locate inflow.
- Dye testing for any inadvertent cross connections with storm sewers at locations where storm sewers run parallel to, or above, sanitary sewers. This would involve flooding a storm sewer inlet with water dyed with a non-toxic solution, and observing the downstream sanitary sewer flow for presence of the dye. This strategy would primarily locate inflow.
- Continuing the City's ongoing smoke testing program to locate sanitary sewer defects, open cleanouts, or inadvertent cross connections. Non-toxic smoke is blown into the sanitary sewer system, and smoke will escape the system and appear above ground at potential defect or cross connection locations. This strategy would primarily locate infiltration.

Figure 5-3 illustrates a potential strategy to begin a further investigation into the sources of inflow in the Bull Branch basin. Exact monitoring locations would be further evaluated to identify locations with optimal hydraulic conditions to capture data. Additional flow meters would allow the collection system to be further divided into smaller basins, to observe which smaller basins exhibit signs of RDII. Data from additional level only sensors would further supplement this information in locations where hydraulic conditions are not favorable to in-pipe flow metering (such as at bends or pipe intersections). This information could pinpoint where additional strategies such as smoke testing or dye testing are warranted.

There are approximately 150 manholes located within the 100-year floodplain boundary along Bull Branch. Visual inspection of these manholes would provide information on the manhole cover type and how much it may potentially leak, manholes with missing manhole covers, or manholes that may have significant wall damage that allows substantial water from the creek to enter the manhole.

Data from additional metering and visual field inspections will yield valuable information with which to further improve the hydraulic model in areas where some uncertainty remains. This would provide additional confidence to decisions regarding where to upsize the Bull Branch interceptor with capital improvement projects, and to what size, to ensure budgeted funds are allocated where they are most needed.

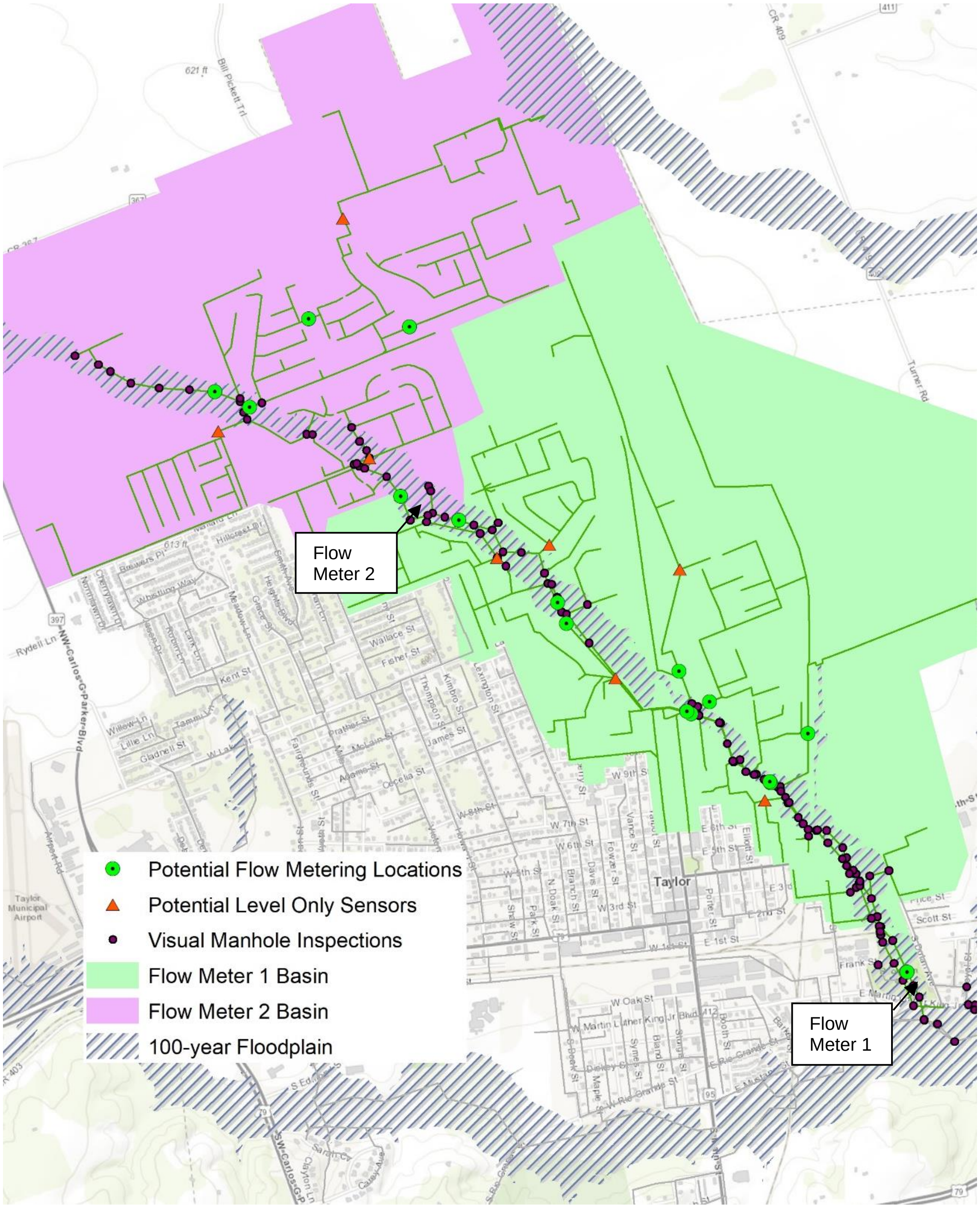



**Figure 5-3. Possible Inflow Investigation Strategy**

## 5.1.2 Inflow Reduction

The calibrated hydraulic model was used to simulate reductions in inflow. During wet weather calibration, certain parameters are established for each metershed that dictate how much RDII is generated in the metershed, and how much of that RDII enters the collection system. To simulate a reduction in inflow, the area contributing inflow to the collection system is reduced by the amount of desired inflow reduction. For example, to simulate a 10% reduction in inflow, the contributing area is reduced by 10%. This effort did focus on reducing inflow, and not infiltration, as a reduction in inflow has the greatest impact in reducing overall peak flow in the sanitary sewer.

This method was utilized to gauge the impact of various levels of inflow reduction on the interceptor performance, without any upsizing. The levels of inflow considered ranged from 5 to 35%. This value represents an average reduction over the metershed, meaning some smaller isolated areas within the metershed will be higher or lower. Therefore, a reduction greater than 35% was not evaluated, as it is unrealistic that adequate inflow reduction could occur in a metershed to achieve that average.

The following maps illustrate the improvement in system performance during a 5-year, 6-hour storm with 10%, 20% and 30% levels of inflow reduction in the existing system. These maps do not include any capital improvement projects to upsize the diameter of the Bull Branch interceptor, only inflow reduction.

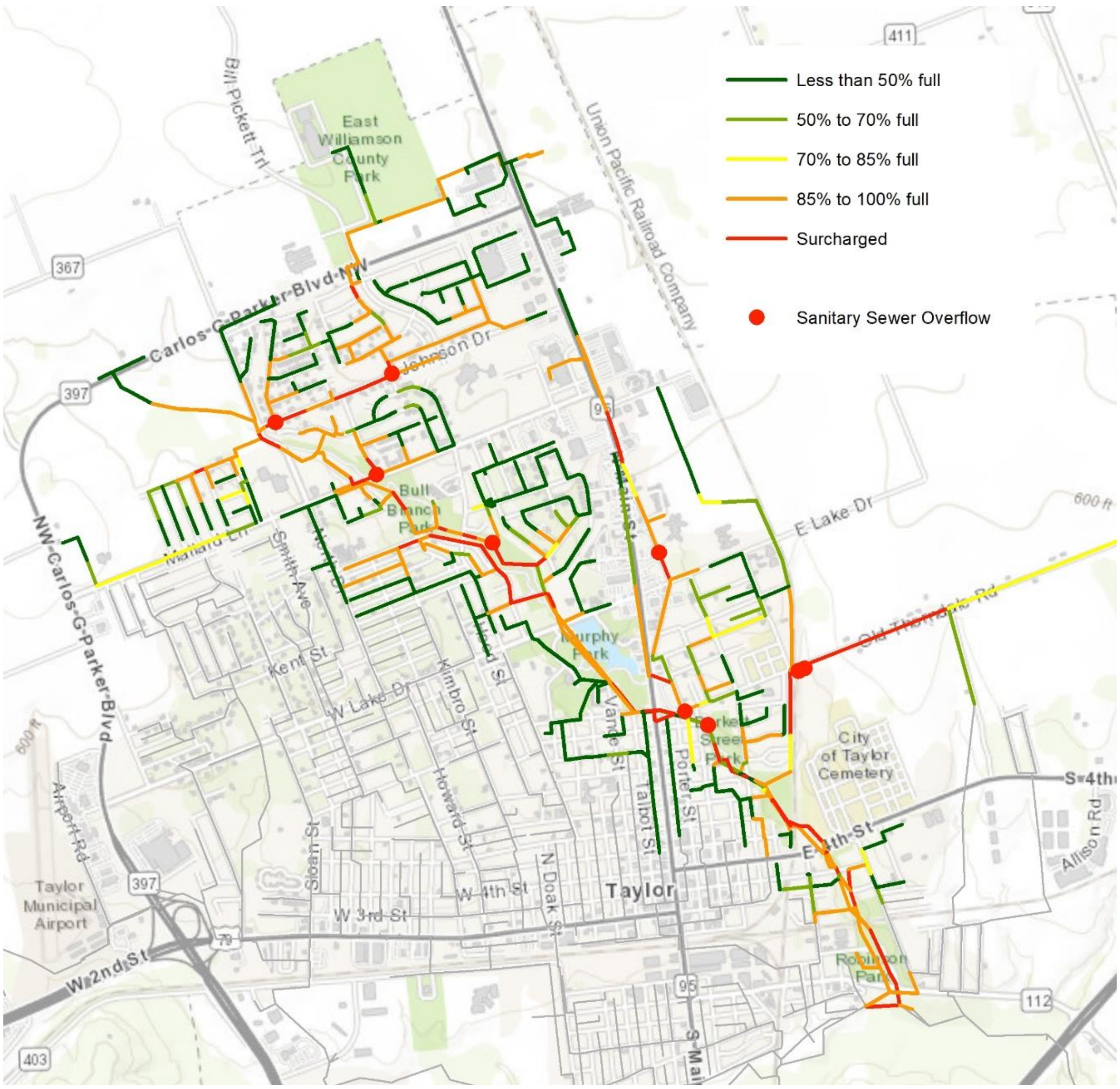



Figure 5-4. Existing System Performance with no Inflow Reduction

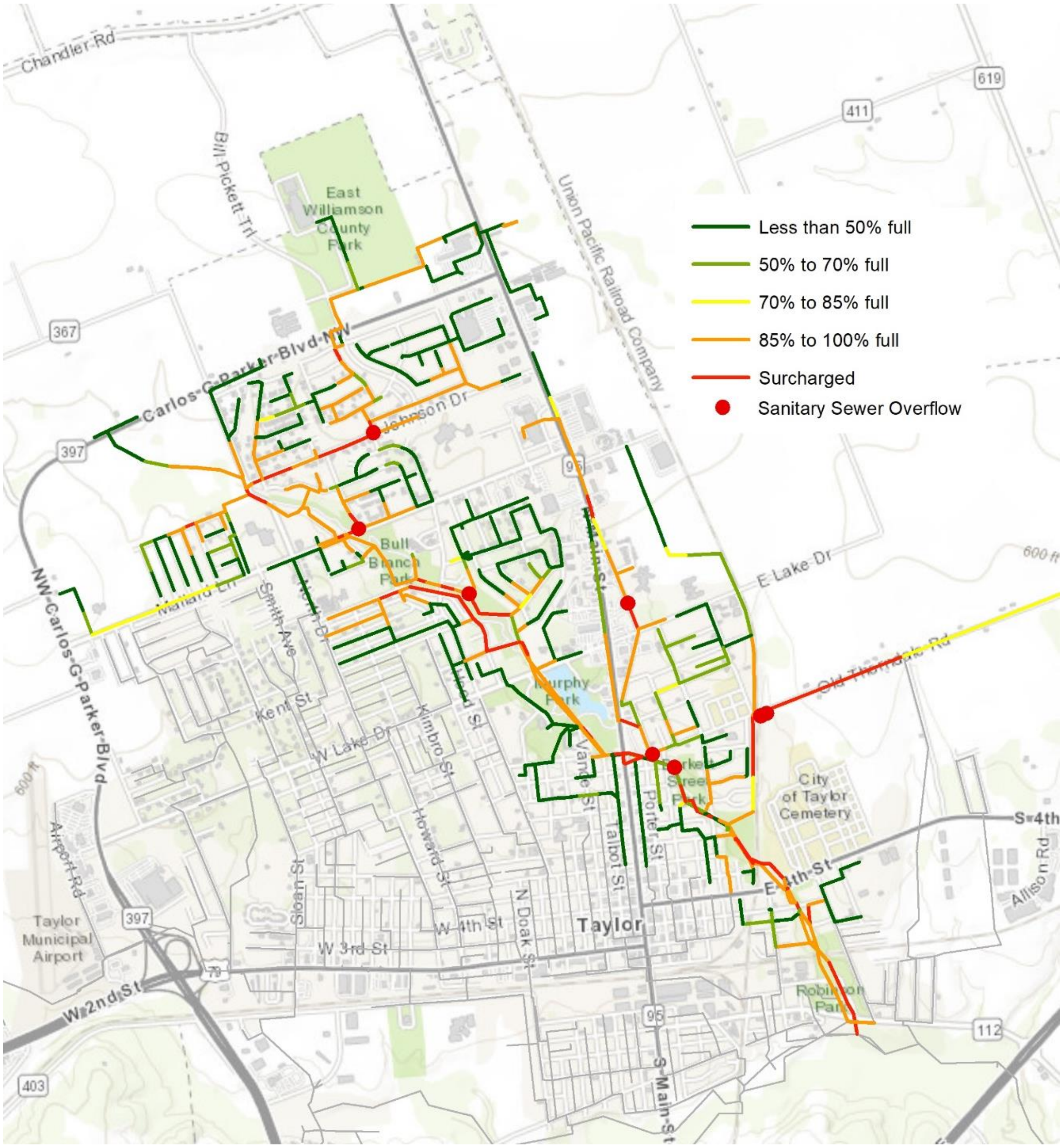



**Figure 5-5. Existing System Performance with 10% Inflow Reduction**

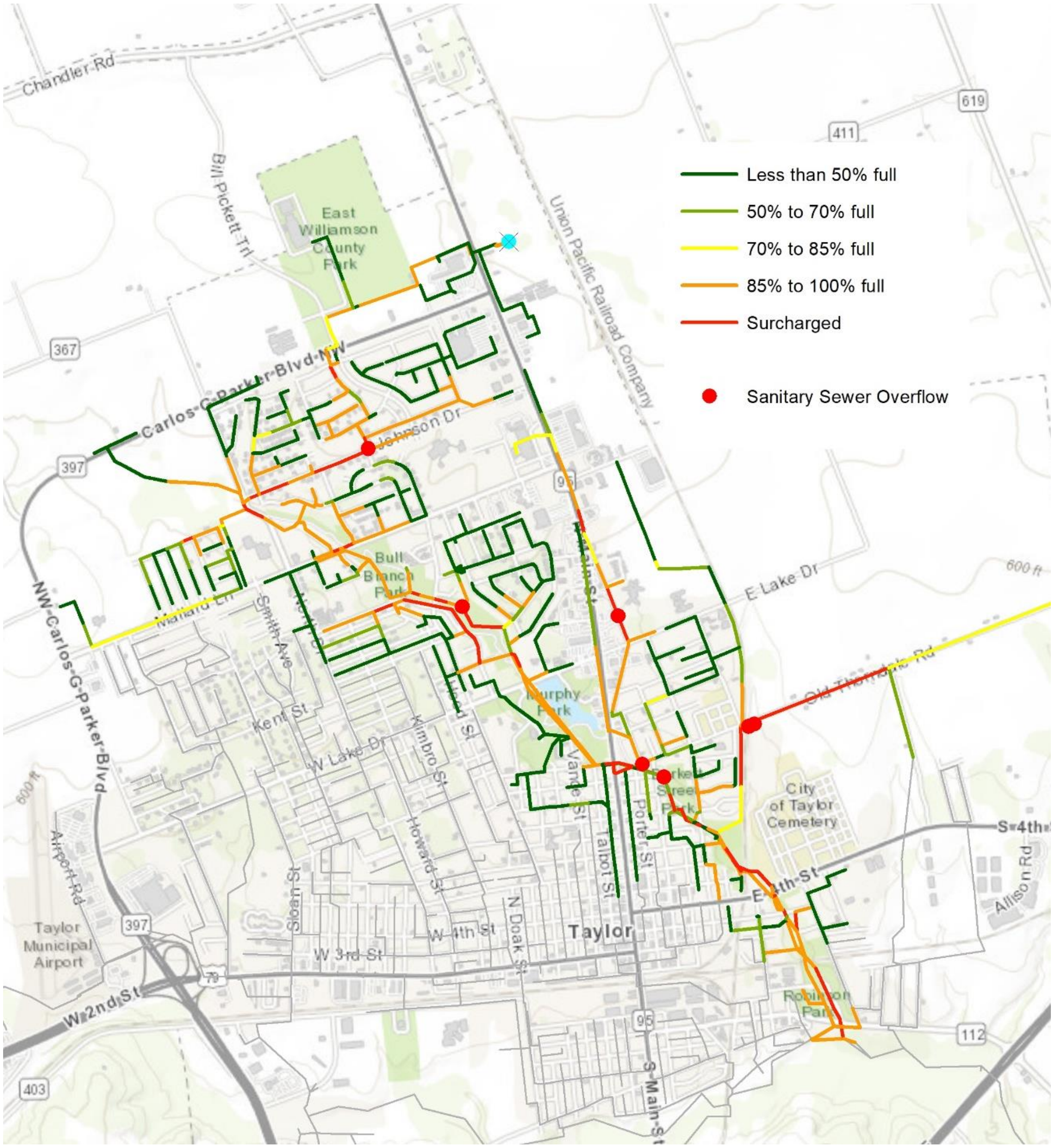



**Figure 5-6. Existing System Performance with 20% Inflow Reduction**

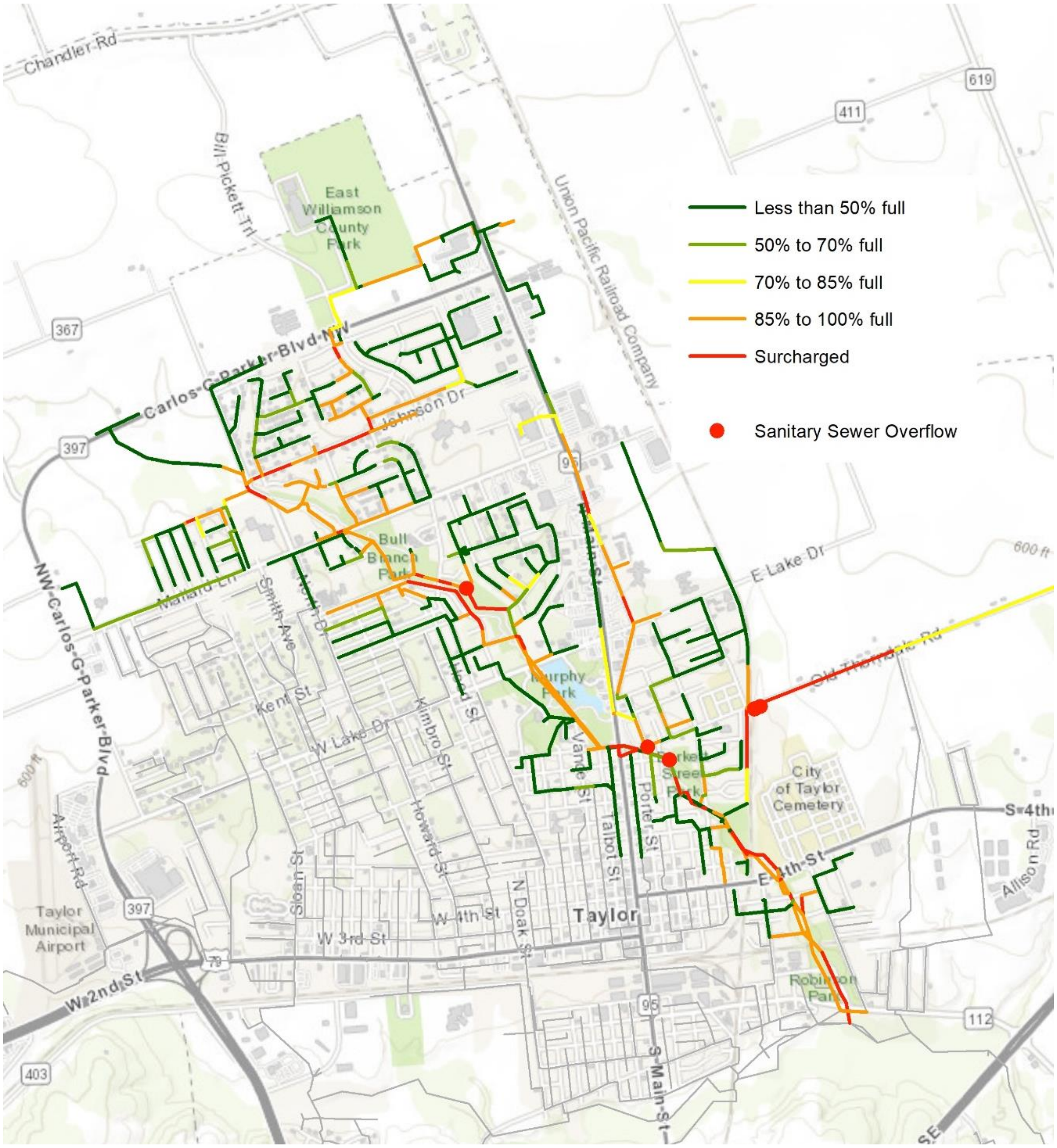



**Figure 5-7. Existing System Performance with 30% Inflow Reduction**

As seen in Figure 5-7, if 30% reduction in inflow can be achieved, with no other improvements to the interceptor capacity, the model predicted SSOs would be reduced to three locations along the Bull Branch interceptor, as well as a location along Old Thorndale Road.

In addition to 30% inflow reduction, to achieve a level of service that would eliminate all SSOs during a 5-year, 6-hour storm, capital improvements to upsize select portions of the Bull Branch interceptor were modeled. These improvements are shown in Figure 5-8.

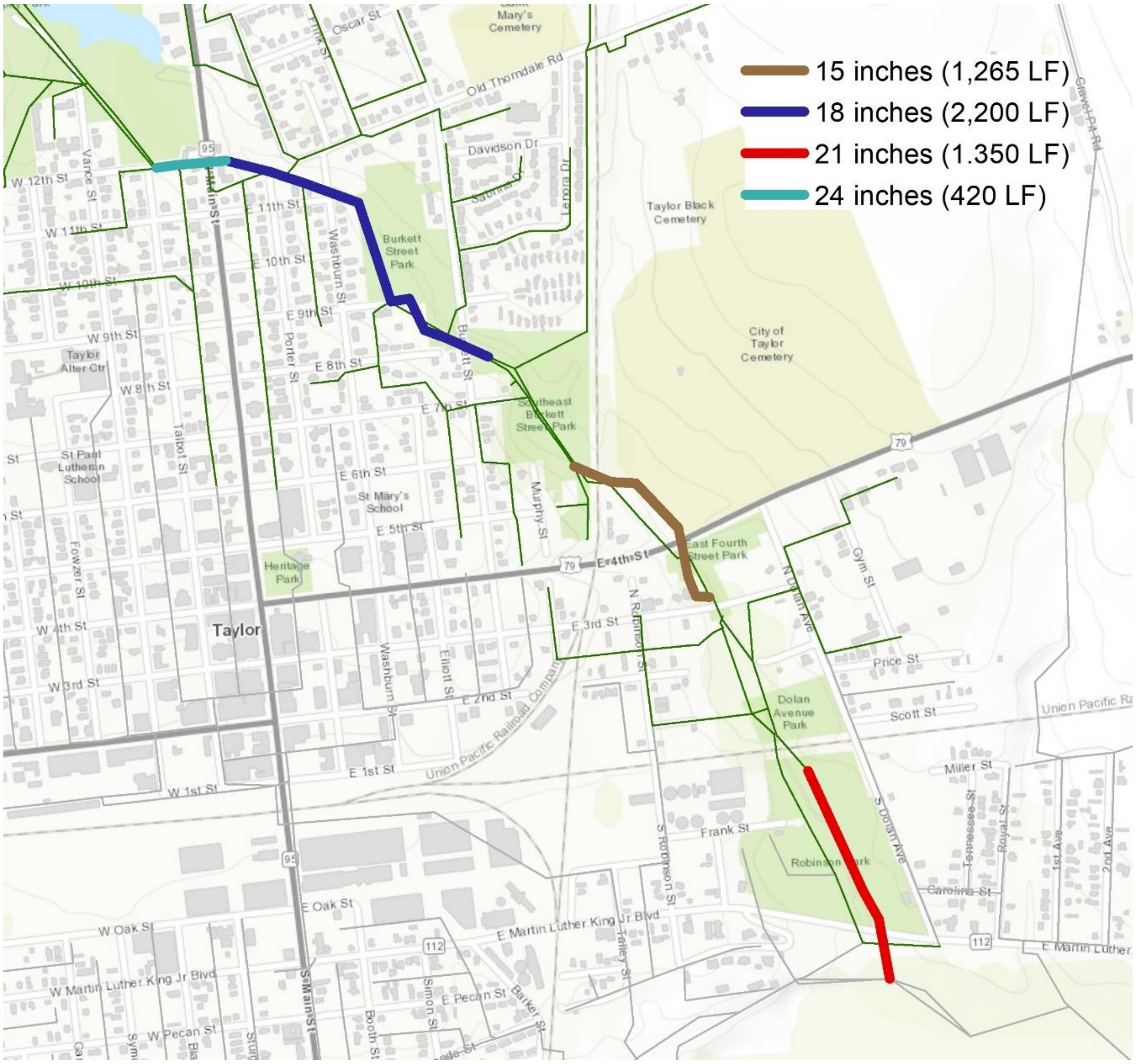



**Figure 5-8. Capital Improvement Project Recommendations, in addition to 30% Inflow Reduction**

Design and construction of the CIP projects shown in Figure 5-8 are estimated to cost approximately $4,000,000 as shown in Table 5-1.This is in addition to approximately $1,500,00 anticipated for inflow reduction investigation and reduction, for a total of $5,500,000. Without any inflow reduction, the master plan identified $9,400,000 in existing system capital improvements along the Bull Branch interceptor. Therefore, it is estimated that if successful, the inflow reduction effort could potentially save up to $4,000,000.

**Table 5-1. Opinion of Probable Construction Costs for Capital Improvement Project Recommendations (Existing System)**

| Pipe Diameter | Length (LF) | OPCC (2023 $) |
|---|---|---|
| **15** | 1,265 | $800,000 |
| **18** | 2,200 | $1,600,000 |
| **21** | 1,350 | $1,100,000 |
| **24** | 420 | $500,000 |
| **Total** | | **$4,000,000** |

The sources and locations of inflow discovered during the investigation will provide information on the most effective remediation method. However, strategies to reduce inflow by 30% include:

- Replacing manhole covers that become submerged with more watertight manholes, or elevating manholes, if visual inspections indicate manhole covers are a significant source of inflow
- Visually inspecting the manholes along interceptor frequently, especially manholes adjacent to the creek, for missing covers or creek inflow and repairing defects
- Eliminating any inadvertent connections with storm sewers or downspouts discovered through dye testing or smoke testing
- Coordination with the public information office to educate the public about open sanitary sewer cleanout caps, through a social media campaign or door hangers

The calibrated hydraulic model indicates that the capital improvements shown in Figure 5-8, in addition to an average 30% RDII reduction, would be sufficient to eliminate SSOs during a 5-year, 6-hour storm with the existing system flows. However, any capital improvement project should be sized to address not only current system flows, but future growth as well. The projected 2040 dry weather flows developed in the master plan, in coordination with the comprehensive plan population projections, were added to the hydraulic model to determine if any additional improvements would be required for the projected 2040 sanitary sewer loading. These improvements are shown in Figure 5-9. The future flows primarily require additional upstream projects, as well as increasing the pipe diameter at the bottom of the interceptor from 21 inches to 24 inches.

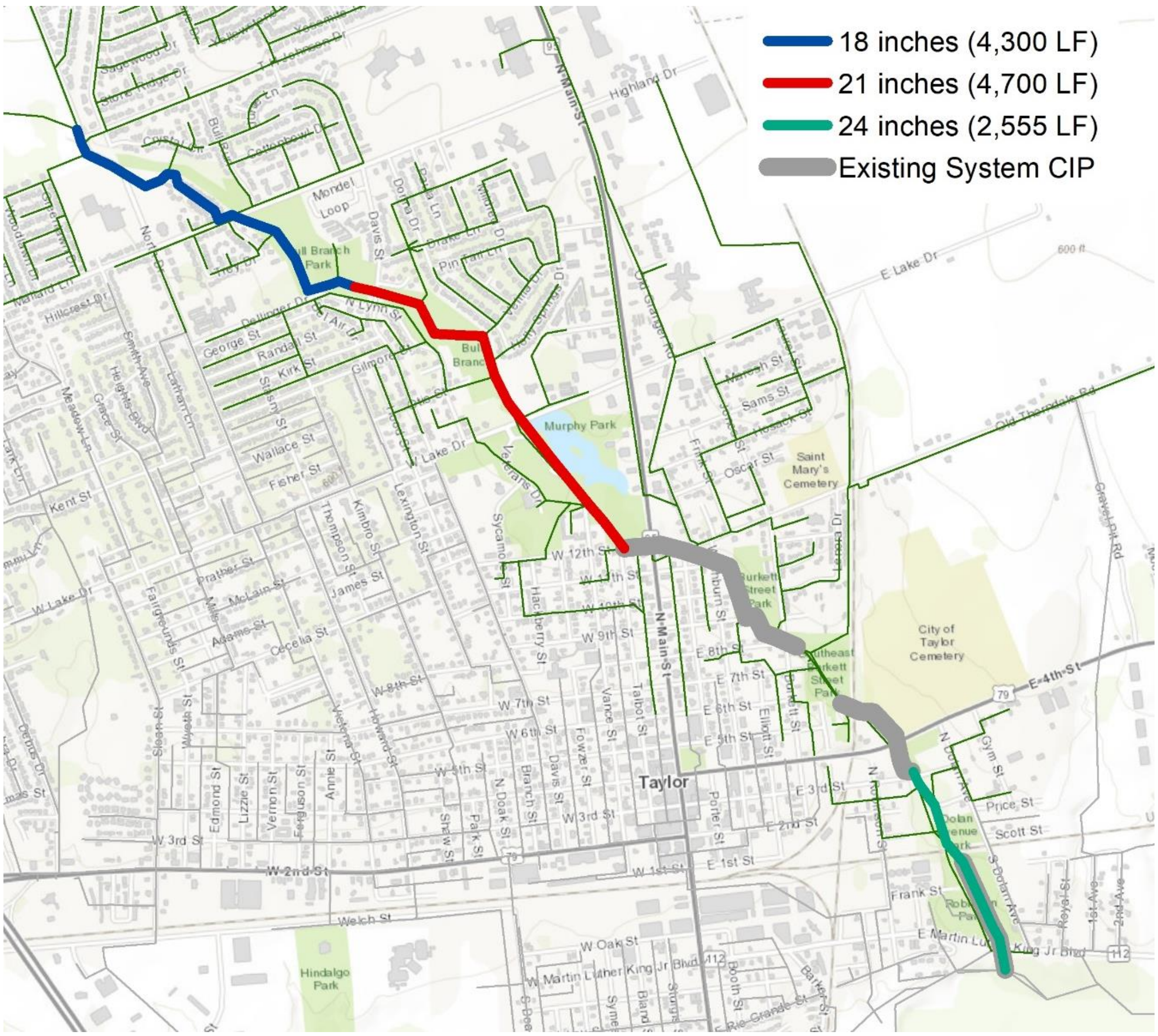



**Figure 5-9. Capital Improvement Project Recommendations, in addition to 30% Inflow Reduction, and Projected 2040 Sanitary Sewer Loading**

Design and construction of the CIP projects shown in Figure 5-8 are estimated to cost approximately $9,100,000, as shown in Table 5-2.

**Table 5-2. Opinion of Probable Construction Costs for Capital Improvement Project Recommendations (with Future Growth)**

| Pipe Diameter | Length (LF) | OPCC (2023 $) |
|---|---|---|
| 18 | 4,300 | $3,000,000 |
| 21 | 4,700 | $3,800,000 |
| 24 | 2,555 | $2,300,000 |
| **Total** | | **$9,100,000** |

$6,800,000 of that $9,100,000 are additional projects needed to serve growth, and are eligible for funding through impact fees. The $2,300,000 for the 24-inch line at the bottom includes some oversizing for growth. A shorter and smaller segment would be required to convey existing flows (1,350 linear feet of 21-inch pipe, estimated at $1,100,000). Therefore, this oversizing of $1,200,000 could be attributed to growth and eligible for funding through impact fees.

## 5.2 Summary and Conclusions

Prior to calibration, the hydraulic model indicated potential capacity constraints and SSOs along the Bull Branch interceptor. The flow metering and subsequent model calibration confirm these constraints. The dry weather and wet weather calibration involved numerous challenges with a complex network of parallel pipes, suspected cross connections, conflicting data, and flow meter data quality. Overall, the model is well calibrated and errs on the conservative side where optimal calibration tolerances could not be achieved with standard calibration methods. The model is well calibrated and errs on the conservative side for system capacity analysis and capital improvement planning.

The flow metering also confirms the presence of RDII, and particularly inflow, in the Bull Branch interceptor. Further investigation to find the source of the RDII will not only provide information on how to best reduce the amount of RDII in the sanitary sewer system, but also valuable information on the system pipe network and flows that can also be used to reduce uncertainty in the model and improve the calibration. Therefore, it is recommended to proceed with the RDII investigation prior to constructing any capital improvement projects. The RDII investigation and reduction efforts will both demonstrate the City is actively addressing the Bull Branch interceptor capacity issue, while also allowing for a refined capital improvement plan to ensure budgeted funds are allocated where they are most needed.

Preliminary design efforts can also begin on upsizing the 2,555 linear feet of the Bull Branch interceptor to 24 inches in diameter, as shown in Figure 5-9. This would include a topographic survey, geotechnical investigation, and definition of horizontal and vertical pipe alignments. If the results of the RDII investigation and inflow reduction efforts and subsequent model update, change the recommended pipe diameter, this can be adjusted prior to final design and construction.

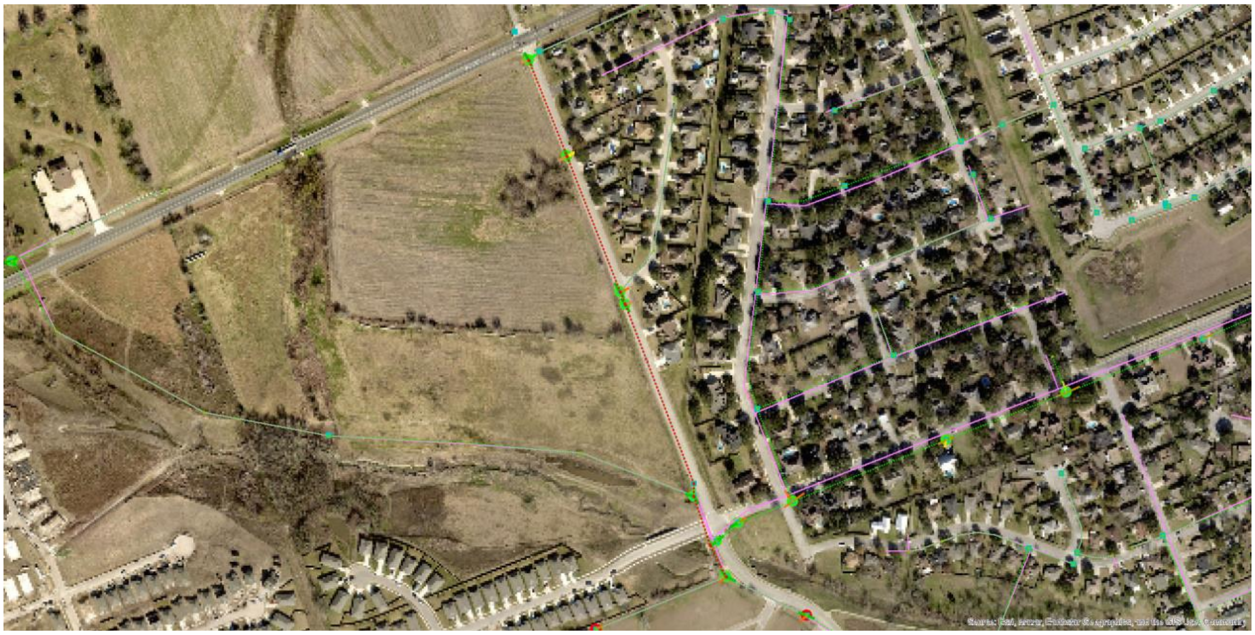
The estimated cost to design and construct this project is approximately $2,300,000. However, $1,200,000 of that project cost is to oversize and extend the project limits for future growth and is impact fee eligible, with the remaining $1,100,000 representing what is required to address existing system flows.

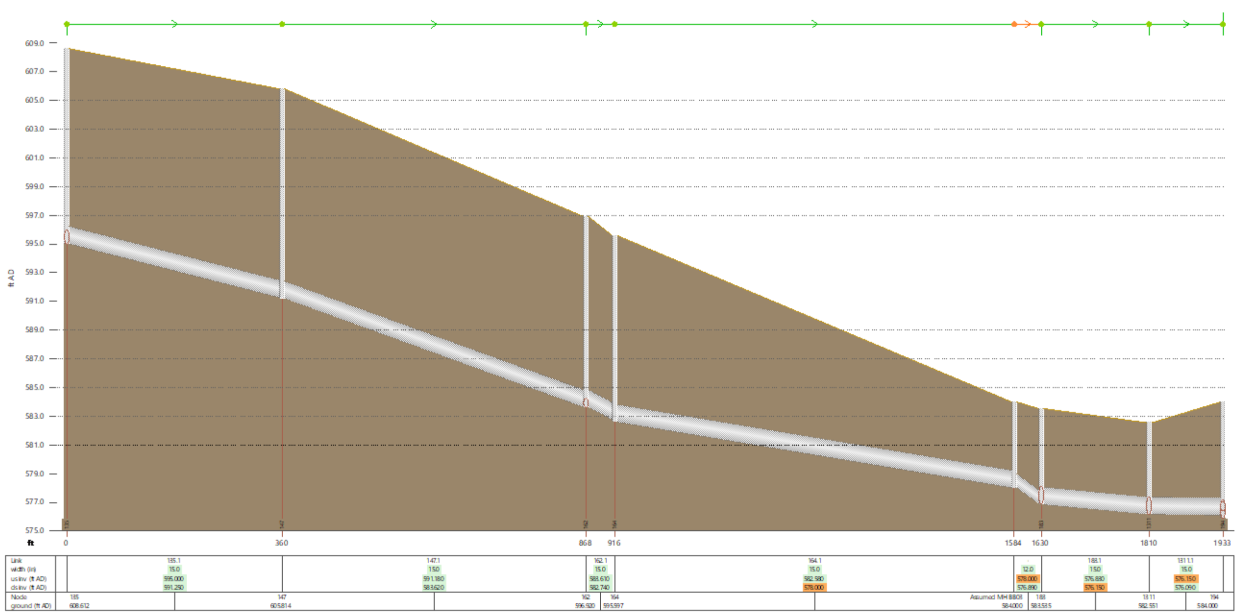
The City has budgeted approximately $3,300,000 to improve the Bull Branch interceptor. It is recommended that these funds be used to investigate and reduce inflow, with the remaining funds used to proceed with increasing the diameter of the downstream portion of the interceptor.

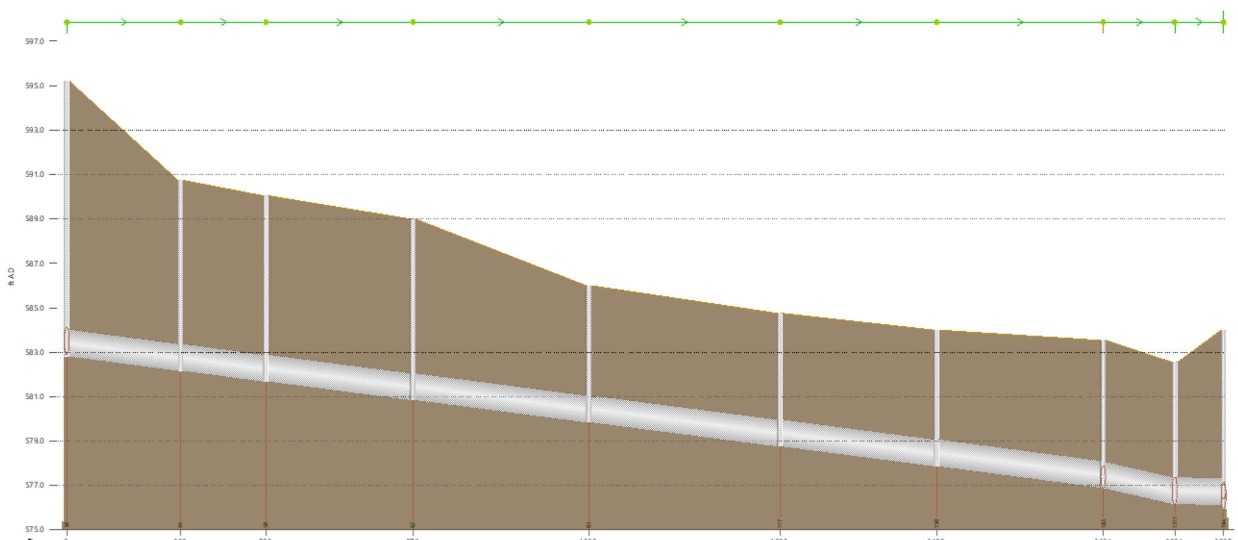
# Appendix A.  Network Connectivity Assumptions

The following is a comparison of the GIS, Survey and final Model alignment and profiles of the Bull Branch sanitary sewer. The final alignment chosen for the model is based upon the GIS, Survey, Flow Metering, Aerial and Street Views.

- GIS provided the initial alignment.
- The manhole survey modified this data based on surveyed size, inverts and azimuth of pipe.
- Flow metering provided model predicted flows and depths verse observed metered data.
- Aerial and street views used to find locations of non-surveyed manholes.

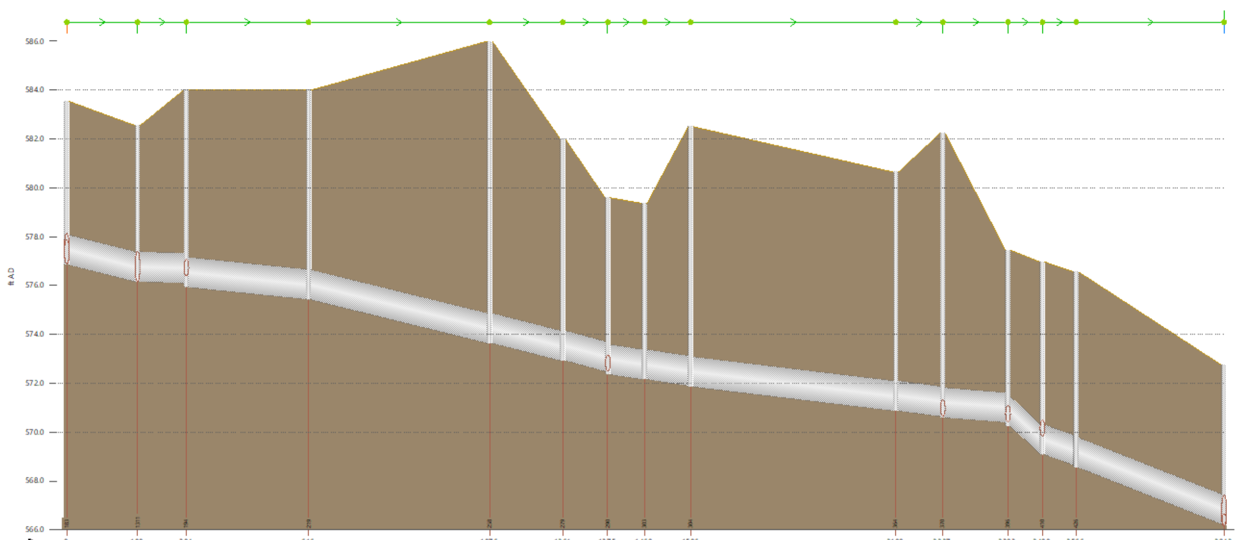






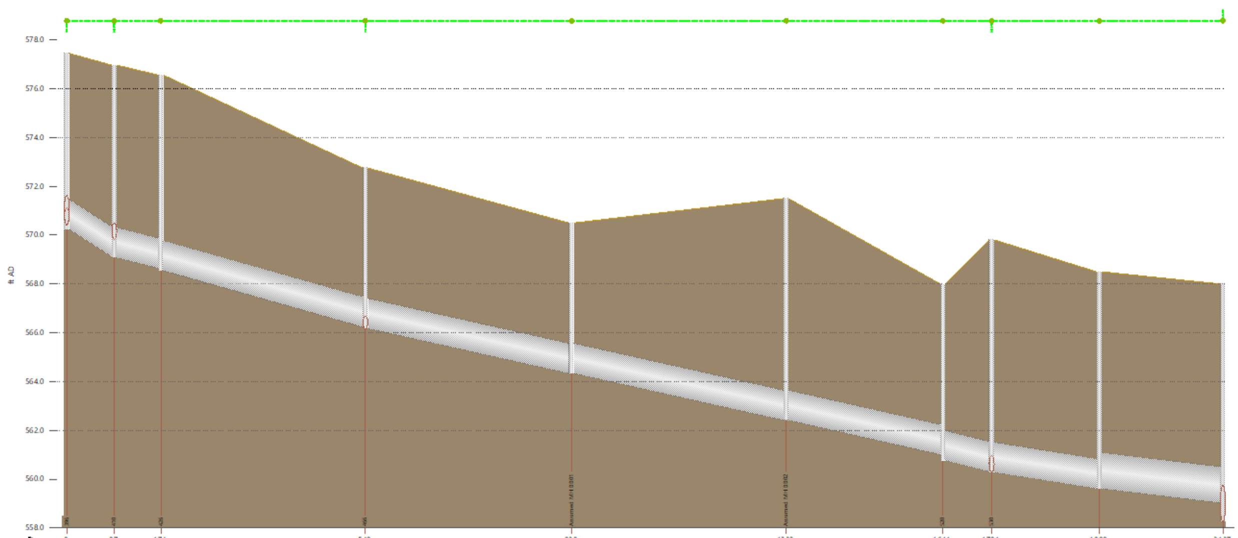

| Link | 38.1 | 44.1 | 50.1 | 62.1 | 88.1 | 117.1 | 150.1 | 183.1 | 1311.1 |
|---|---|---|---|---|---|---|---|---|---|
| width (in) | 15.0 | 15.0 | 15.0 | 15.0 | 15.0 | 15.0 | 15.0 | 15.0 | 15.0 |
| us inv (ft AD) | 582.790 | 582.142 | 581.657 | 580.820 | 579.820 | 578.734 | 577.842 | 576.880 | 576.150 |
| ds inv (ft AD) | 582.142 | 581.657 | 580.820 | 579.820 | 578.734 | 577.842 | 576.880 | 576.150 | 576.090 |

| Node | 38 | 44 | 50 | 62 | 88 | 117 | 150 | 183 | 1311 | 194 |
|---|---|---|---|---|---|---|---|---|---|---|
| ground (ft AD) | 595.219 | 590.740 | 590.046 | 588.999 | 586.000 | 584.753 | 584.000 | 583.535 | 582.551 | |



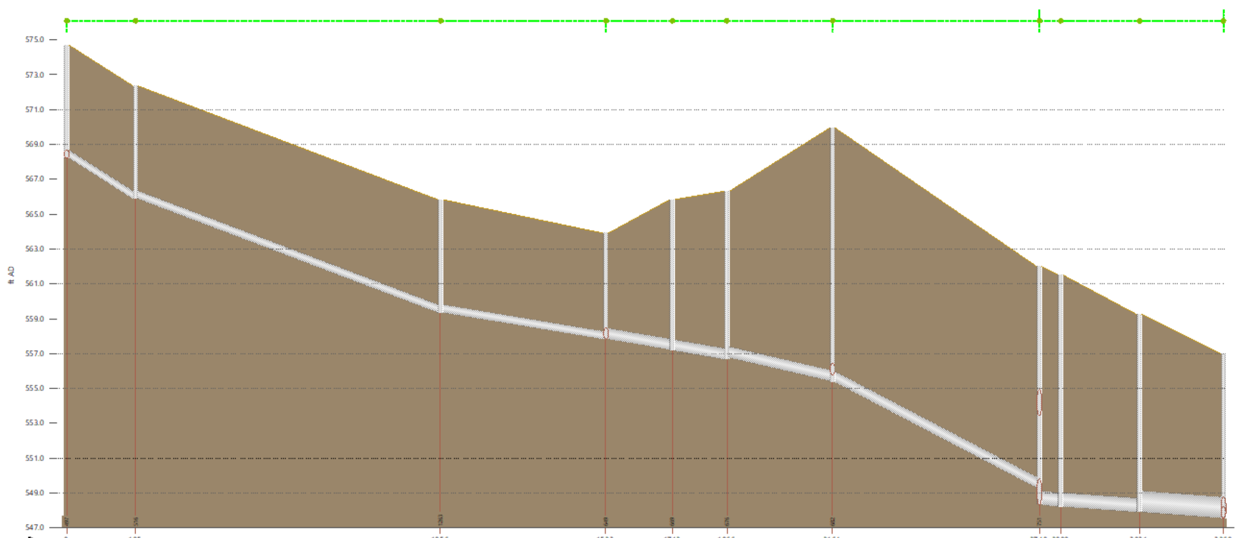

| Link | 188.1 | 1311.1 | 194.1 | 219.2 | 258.1 | 279.1 | 290.1 | 303.1 | 304.1 | 364.1 | 370.1 | 396.1 | 410.1 | 426.1 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| width (in) | 15.0 | 15.0 | 15.0 | 15.0 | 15.0 | 15.0 | 15.0 | 15.0 | 15.0 | 15.0 | 15.0 | 15.0 | 15.0 | 15.0 |
| us inv (ft AD) | 576.880 | 576.150 | 575.920 | 575.410 | 573.686 | 572.920 | 572.350 | 572.150 | 571.850 | 570.865 | 570.580 | 570.230 | 569.070 | 568.540 |
| ds inv (ft AD) | 576.150 | 576.090 | 575.480 | 573.686 | 572.920 | 572.480 | 572.150 | 571.890 | 570.865 | 570.640 | 570.390 | 569.110 | 568.640 | 566.230 |

| Node | 188 | 1311 | 194 | 219 | 258 | 279 | 290 | 303 | 304 | 364 | 370 | 396 | 410 | 426 | 466 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ground (ft AD) | 588.585 | 582.551 | 584.000 | 584.000 | 586.000 | 581.998 | 579.590 | 579.367 | 582.501 | 580.644 | 582.233 | 577.449 | 576.968 | 576.559 | 572.751 |



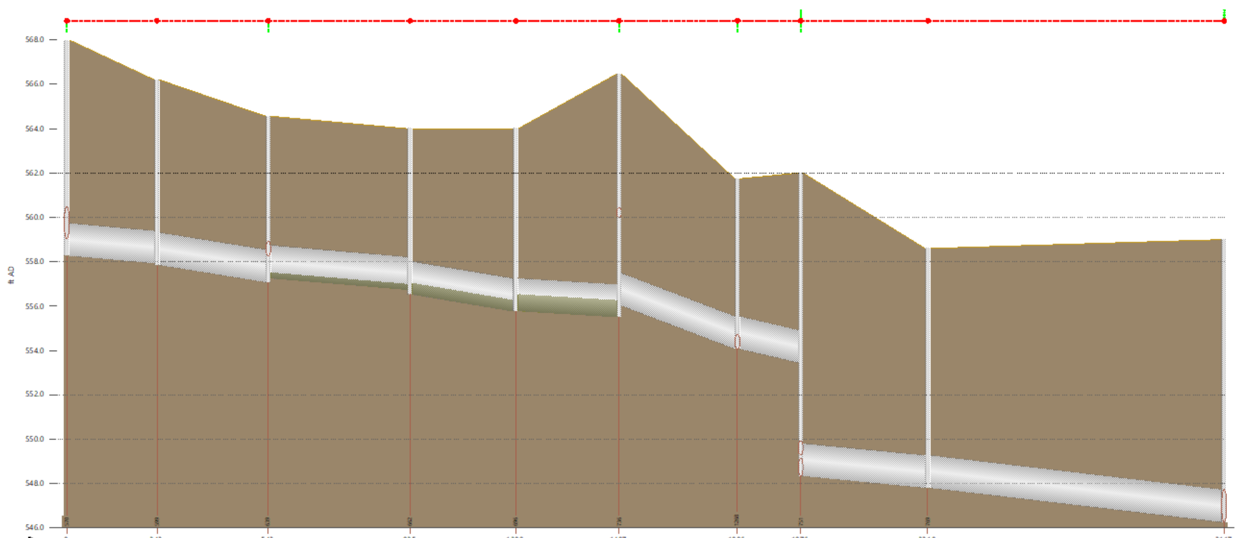

| Link | 396.1 | 410.1 | -426.1 | 466.1 | Assumed MH BB01.1 | Assumed MH BB02.1 | 520.1 | 530.1 | MH Junction 15 18.1 |
|---|---|---|---|---|---|---|---|---|---|
| width (in) | 15.0 | 15.0 | 15.0 | 15.0 | 15.0 | 15.0 | 15.0 | 15.0 | 18.0 |
| us inv (ft AD) | 570.230 | 569.070 | 568.540 | 566.170 | 564.326 | 562.408 | 560.750 | 560.270 | 559.600 |
| ds inv (ft AD) | 569.110 | 568.640 | 566.230 | 564.326 | 562.408 | 561.000 | 560.300 | 559.600 | 559.020 |

| Node | 396 | 410 | -426 | 466 | Assumed MH BB01 | Assumed MH BB02 | 520 | 530 | MH Junction 15 18 | 570 |
|---|---|---|---|---|---|---|---|---|---|---|
| ground (ft AD) | - | 576.968 | 576.559 | 572.751 | 570.500 | 571.500 | 568.000 | 569.806 | 568.500 | 568.000 |

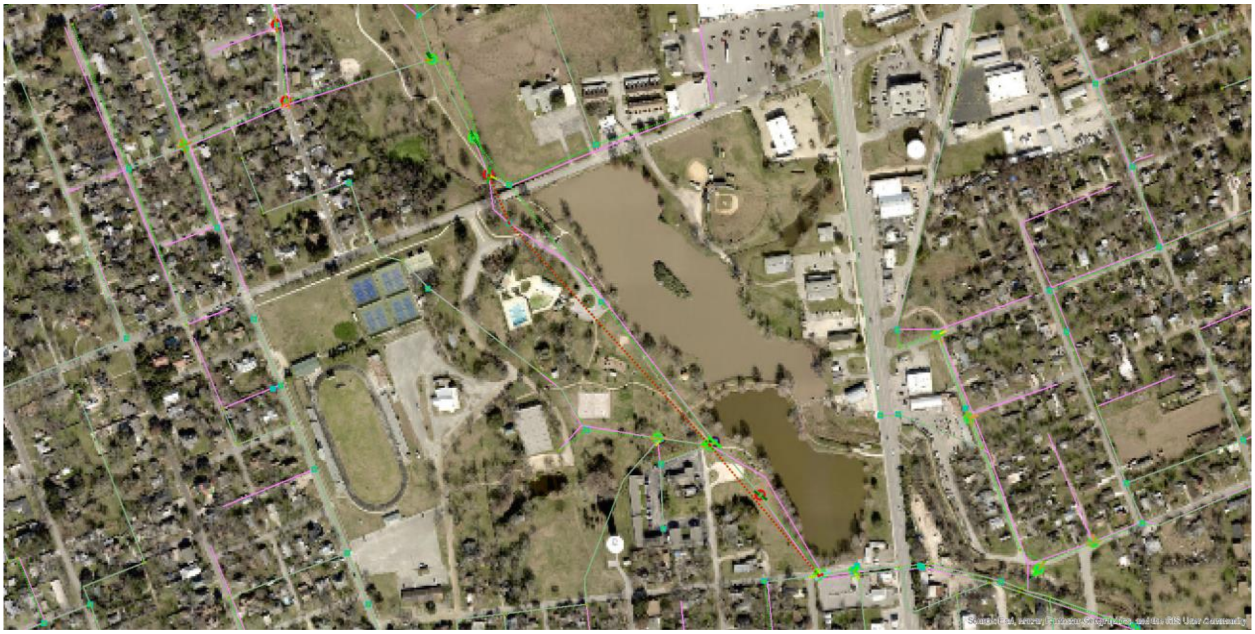

| Link | 497.2 | 516.2 | 1263.1 | 649.1 | 669.1 | 676.1 | 682.1 | 751.2 | MH Question.1 | MH Q2.1 |
|---|---|---|---|---|---|---|---|---|---|---|
| width (in) | 6.0 | 6.0 | 6.0 | 8.0 | 8.0 | 8.0 | 8.0 | 10.0 | 10.0 | 15.0 |
| us inv (ft AD) | 568.194 | 565.857 | 559.333 | 557.810 | 557.150 | 556.750 | 555.340 | [illegible] | 548.200 | 547.900 |
| ds inv (ft AD) | 565.857 | 559.333 | 557.810 | 557.210 | 556.660 | 555.420 | 549.280 | [illegible] | 547.900 | 547.550 |

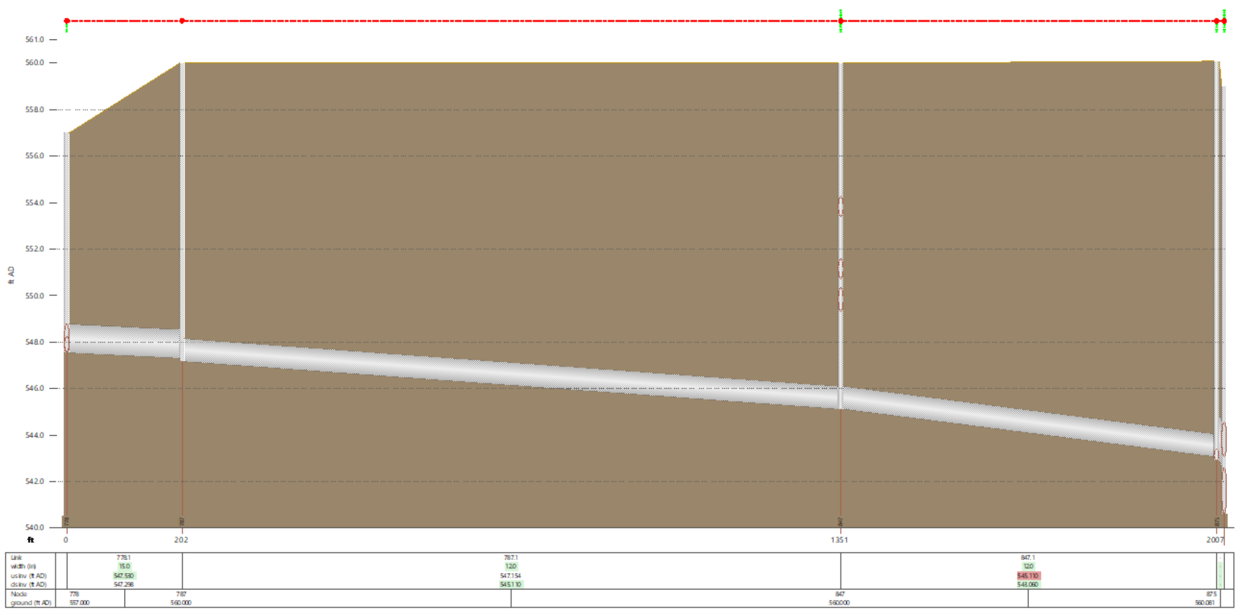
| Node | 497 | 516 | 1263 | 649 | 669 | 676 | 682 | 751 | MH Question | MH Q2 | 778 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ground (ft AD) | 574.694 | 572.357 | 565.833 | 563.910 | 565.842 | 566.322 | 569.961 | 562.000 | 561.500 | 559.250 | 557.000 |



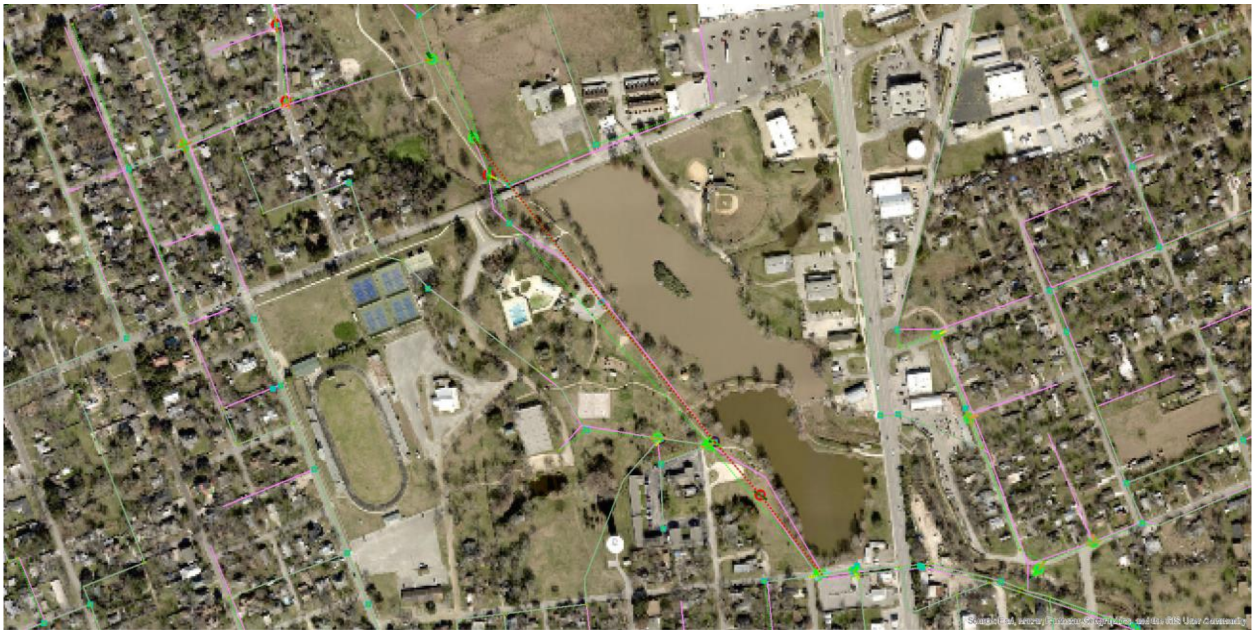

| Link | 570.1 | 599.1 | 639.1 | 662.1 | 696.1 | 736.2 | 1268.1 | 751.1 | 769.1 |
|---|---|---|---|---|---|---|---|---|---|
| width (in) | 18.0 | 18.0 | 18.0 | 18.0 | 18.0 | 18.0 | 18.0 | 18.0 | 18.0 |
| us inv (ft AD) | 558.250 | 557.830 | 557.250 | 556.520 | 555.770 | 556.000 | 554.054 | 548.320 | 547.790 |
| ds inv (ft AD) | 557.910 | 557.060 | 556.730 | 555.760 | 555.500 | 554.054 | 553.440 | 547.790 | 546.250 |

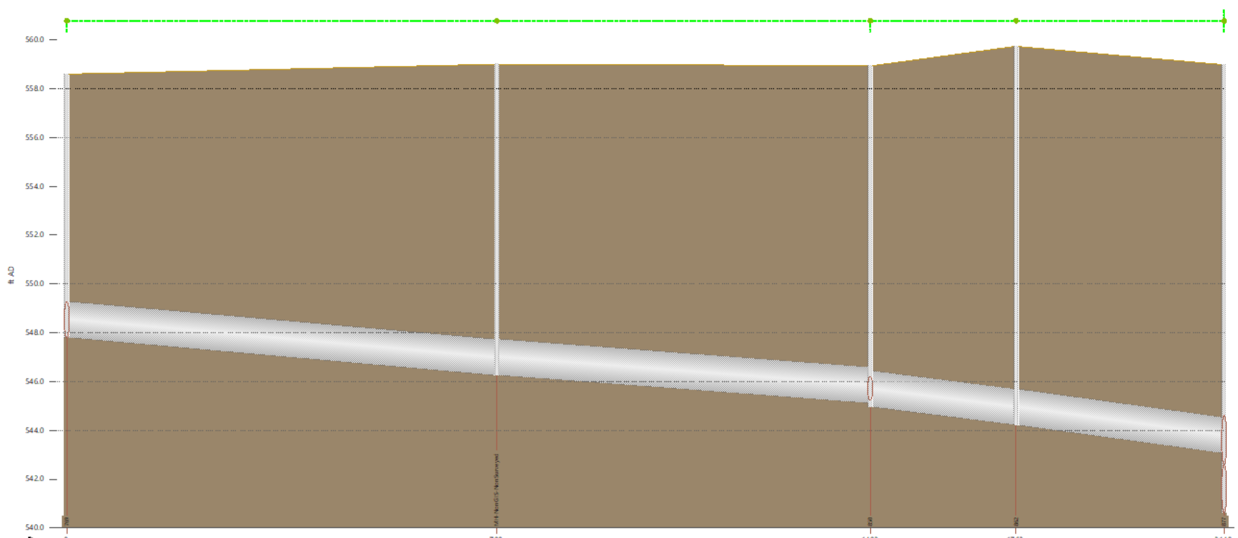
| Node | 570 | 599 | 639 | 662 | 696 | 736 | 1268 | 751 | 769 | MH NonGIS NonSurveyed |
|---|---|---|---|---|---|---|---|---|---|---|
| ground (ft AD) | 568.000 | 566.200 | 564.556 | 564.000 | 564.000 | 566.464 | 561.742 | 562.000 | 558.999 | 559.000 |





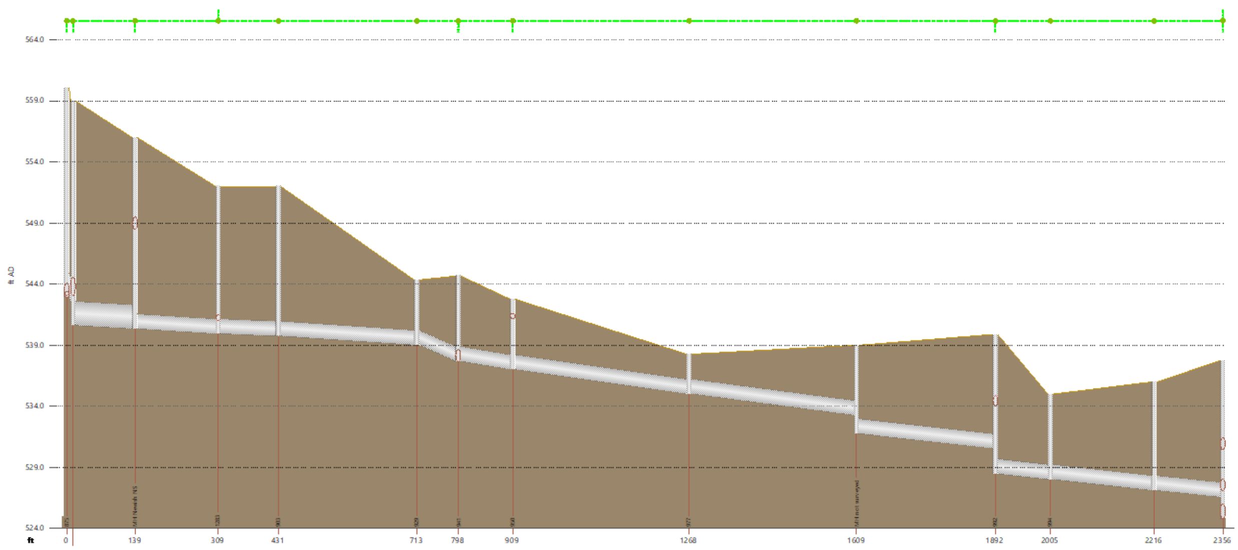
561.0
560.0
558.0
556.0
554.0
552.0
550.0
548.0
546.0
544.0
542.0
540.0
ft AD
ft
0
202
1351
2007
Link
width (in)
usinv (ft AD)
dsinv (ft AD)
778.1
15.0
547.530
547.298
787.1
12.0
547.154
545.110
847.1
12.0
545.110
543.060
Node
ground (ft AD)
778
557.000
787
560.000
847
560.000
875
560.081





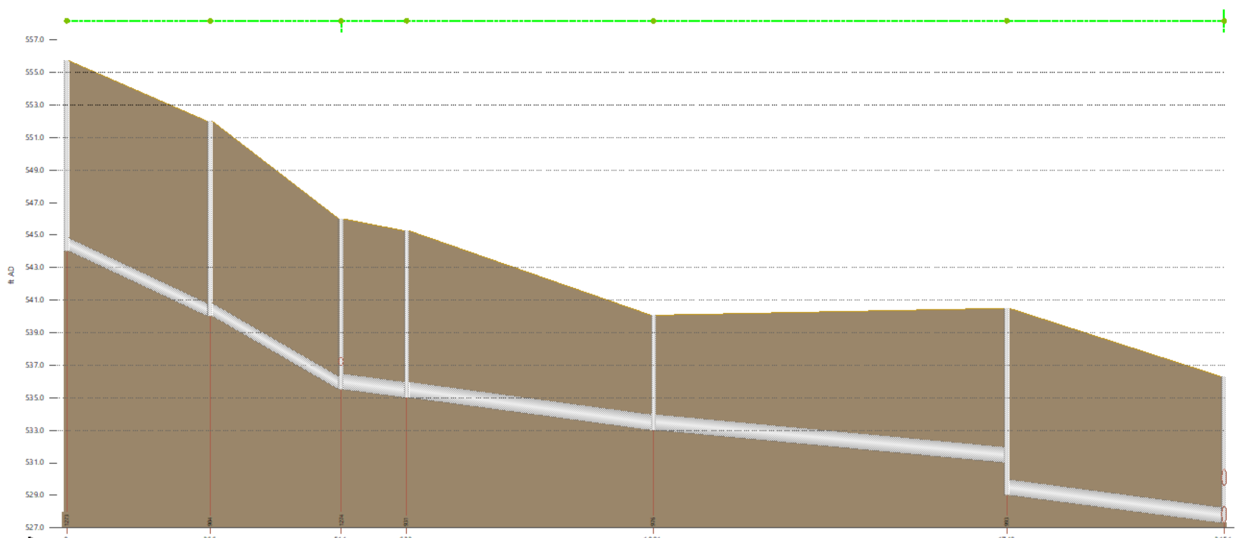

| | | | | |
|---|---|---|---|---|
| Link | 769.1 | MH-NonGIS-NonSurveyed.1 | 850.1 | 862.1 |
| width (in) | 18.0 | 18.0 | 18.0 | 18.0 |
| us inv (ft AD) | 547.790 | 546.250 | 544.950 | 544.200 |
| ds inv (ft AD) | 546.250 | 545.110 | 544.220 | 543.060 |

| | | | | | |
|---|---|---|---|---|---|
| Node | 769 | MH-NonGIS-NonSurveyed | 850 | 862 | 877 |
| ground (ft AD) | 558.599 | 559.000 | 558.944 | 559.725 | 558.987 |



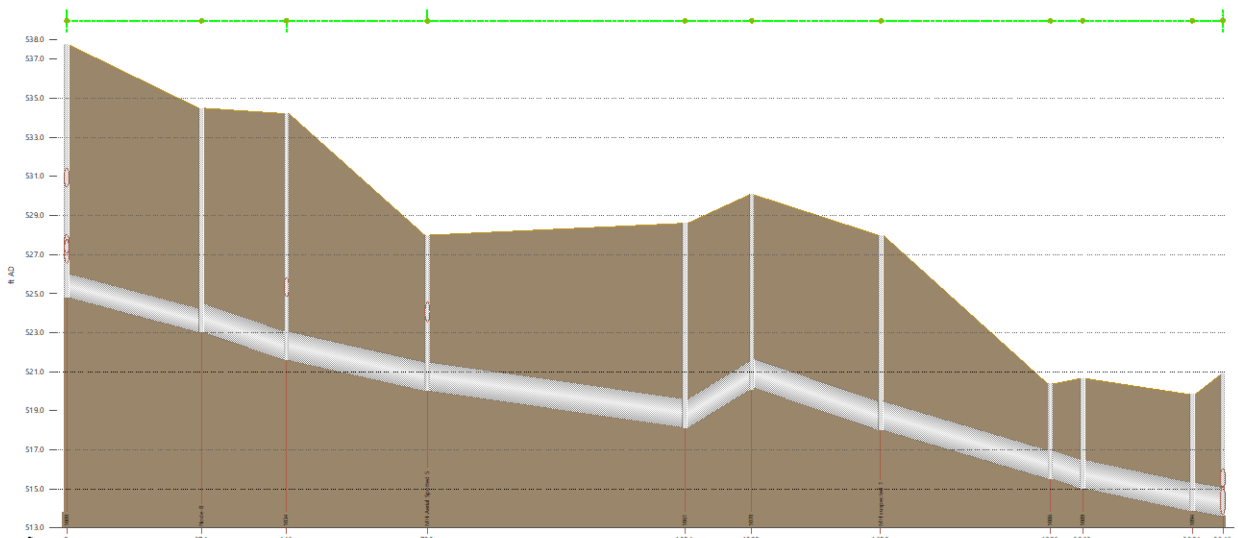

| Link | 877.1 | MH Nowish NS.1 | 1288.2 | 908.1 | 929.1 | 941.1 | 950.1 | 977.2 | MH not surveyed.2 | 992.1 | 994.1 | MH Maybe here.1 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| width (in) | 24.0 | 15.0 | 15.0 | 15.0 | 15.0 | 15.0 | 15.0 | 15.0 | 15.0 | 15.0 | 15.0 | 15.0 |
| us inv (ft AD) | 540.610 | 540.331 | 539.928 | 539.742 | 539.000 | 537.673 | 537.000 | 535.000 | 531.750 | 528.470 | 528.000 | 527.100 |
| ds inv (ft AD) | 540.331 | 539.928 | 539.742 | 539.000 | 537.673 | 537.000 | 535.000 | 533.250 | 530.530 | 528.000 | 527.100 | 526.550 |

| Node | 877 | MH Nowish NS | 1288 | 908 | 929 | 941 | 950 | 977 | MH not surveyed | 992 | 994 | MH Maybe here | 1009 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ground (ft AD) | 558.987 | 556.000 | 552.000 | 552.000 | 544.353 | 544.673 | 542.773 | 538.271 | 539.000 | 539.890 | 535.000 | 536.000 | 537.745 |



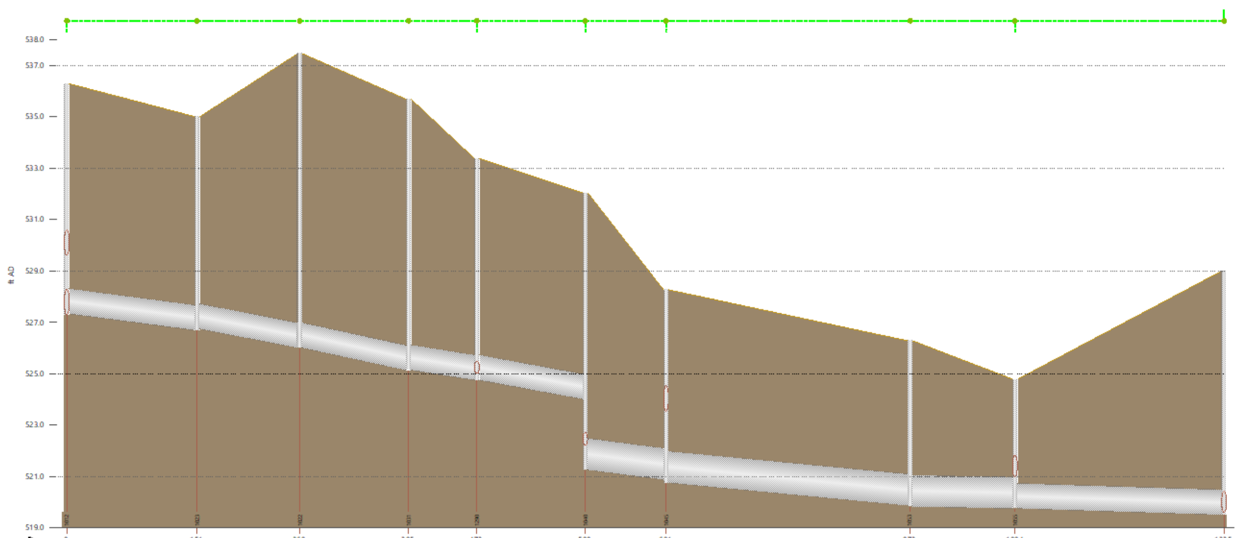

| Link | 1273.2 | 904.1 | 1274.1 | 931.1 | 976.1 | 998.1 |
|---|---|---|---|---|---|---|
| width (in) | 10.0 | 10.0 | 12.0 | 12.0 | 12.0 | 12.0 |
| us inv (ft AD) | 544.000 | 540.000 | 535.500 | 535.000 | 533.000 | 529.000 |
| ds inv (ft AD) | 540.000 | 535.500 | 535.000 | 533.000 | 531.000 | 527.290 |

| Node | 1273 | 904 | 1274 | 931 | 976 | 998 | 1012 |
|---|---|---|---|---|---|---|---|
| ground (ft AD) | 555.711 | 552.000 | 546.000 | 545.265 | 540.077 | 540.499 | 536.297 |


| Link | 1009.1 | Node 8.1 | 1034.1 | MH Aerial Spotted 5.1 | 1061.1 | 1070.1 | MH suspected 1.1 | 1086.1 | 1089.1 | 1094.1 |
|---|---|---|---|---|---|---|---|---|---|---|
| width (in) | 15.0 | 18.0 | 18.0 | 18.0 | 18.0 | 18.0 | 18.0 | 18.0 | 18.0 | 18.0 |
| us inv (ft AD) | 524.780 | 523.000 | 521.570 | 520.000 | 518.100 | 520.160 | 518.000 | 515.500 | 515.000 | 513.849 |
| ds inv (ft AD) | 523.000 | 521.990 | 520.000 | 518.140 | 520.040 | 518.000 | 515.500 | 515.000 | 513.849 | 513.600 |

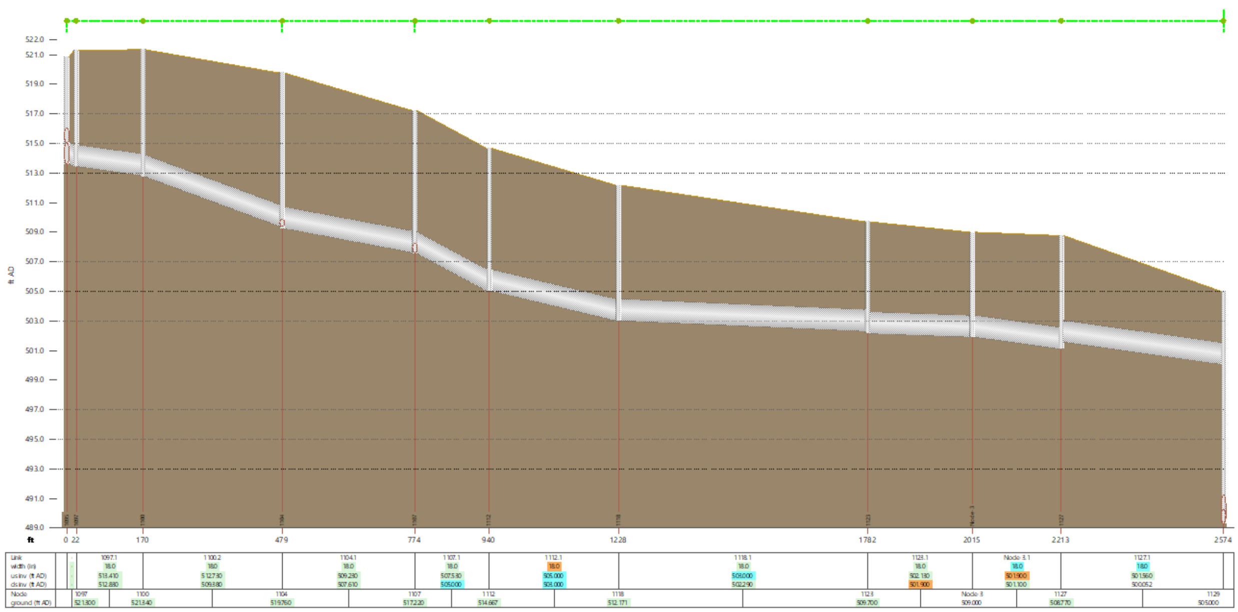
| Node | 1009 | Node 8 | 1034 | MH Aerial Spotted 5 | 1061 | 1070 | MH suspected 1 | 1086 | 1089 | 1094 |
|---|---|---|---|---|---|---|---|---|---|---|
| ground (ft AD) | 537.745 | 534.500 | 534.222 | 528.000 | 528.600 | 530.062 | 528.000 | 520.382 | 520.657 | 519.838 |

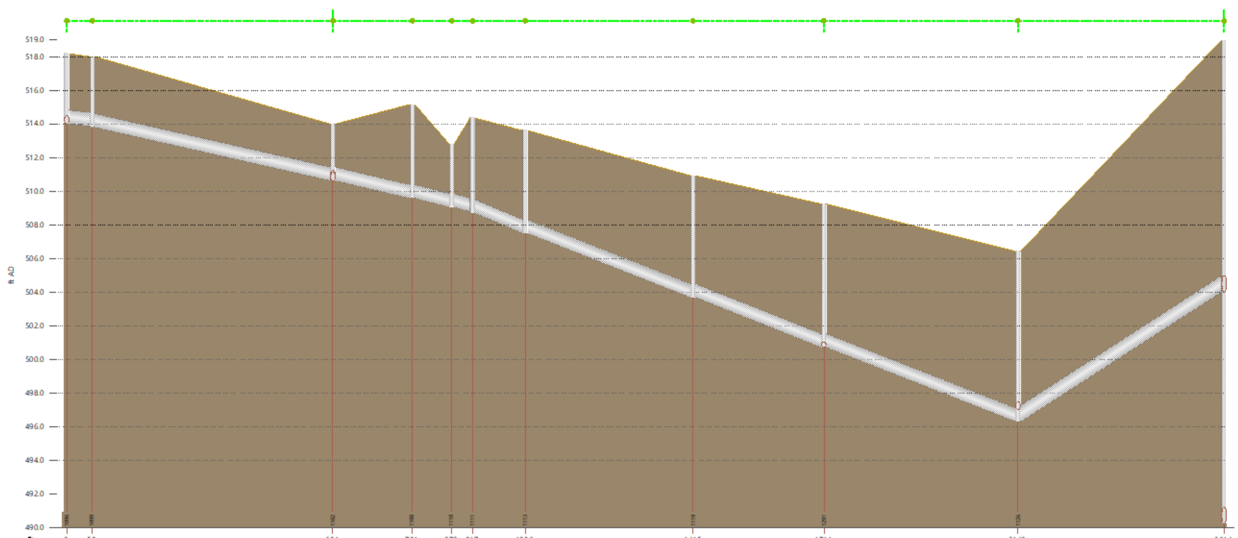

| Link | 1012.1 | 1023.1 | 1022.1 | 1031.1 | 1290.1 | 1040.1 | 1045.1 | 1053.1 | 1055.3 |
|---|---|---|---|---|---|---|---|---|---|
| width (in) | 12.0 | 12.0 | 12.0 | 12.0 | 12.0 | 15.0 | 15.0 | 15.0 | 12.0 |
| us inv (ft AD) | 527.330 | 526.740 | 526.000 | 525.140 | 524.750 | 521.290 | 520.750 | 519.820 | 519.750 |
| ds inv (ft AD) | 526.690 | 526.000 | 525.120 | 524.750 | 524.000 | 520.870 | 519.860 | 519.750 | 519.500 |

| Node | 1012 | 1023 | 1022 | 1031 | 1290 | 1040 | 1045 | 1053 | 1055 | MH AV1 |
|---|---|---|---|---|---|---|---|---|---|---|
| ground (ft AD) | 536.297 | 535.010 | 537.462 | 535.690 | 533.381 | 532.040 | 528.270 | 526.289 | 524.793 | 529.000 |

Source: Esri, Maxar, Earthstar Geographics, and the GIS User Community

ft AD
522.0
521.0
519.0
517.0
515.0
513.0
511.0
509.0
507.0
505.0
503.0
501.0
499.0
497.0
495.0
493.0
491.0
489.0
ft
0
22
170
479
774
940
1228
1782
2015
2213
2574
Link
width (in)
us inv (ft AD)
ds inv (ft AD)
1097.1
18.0
513.410
512.880
1100.2
18.0
512.730
509.380
1104.1
18.0
509.230
507.610
1107.1
18.0
507.530
505.000
1112.1
18.0
505.000
503.000
1118.1
18.0
503.000
502.290
1123.1
18.0
502.130
501.900
Node 3.1
18.0
501.900
501.100
1127.1
18.0
501.560
500.052
Node
ground (ft AD)
1097
521.300
1100
521.340
1104
519.760
1107
517.220
1112
514.667
1118
512.171
1123
509.700
Node 3
509.000
1127
508.770
1129
505.000

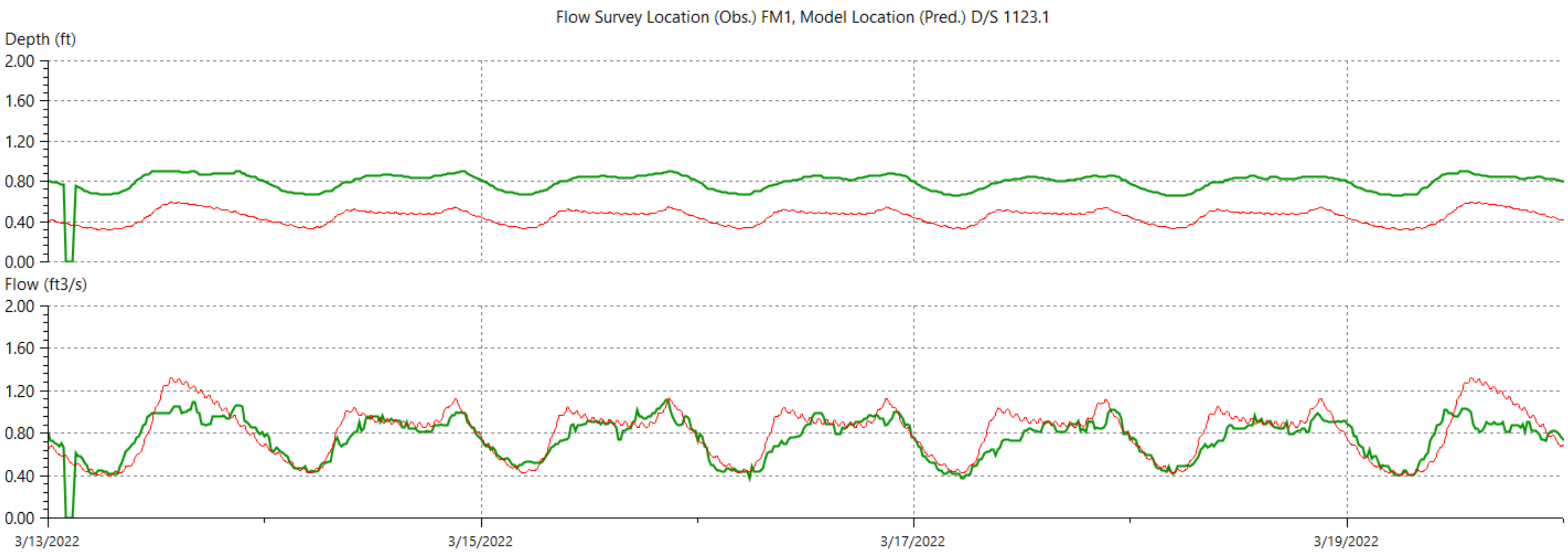




| Link | 1096.1 | 1099.1 | 1102.1 | 1108.1 | 1110.1 | 1111.1 | 1113.1 | 1119.1 | 1291.1 | 1126.2 |
|---|---|---|---|---|---|---|---|---|---|---|
| width (in) | 10.0 | 10.0 | 10.0 | 10.0 | 10.0 | 10.0 | 10.0 | 10.0 | 10.0 | 12.0 |
| us inv (ft AD) | 514.005 | 513.790 | 510.620 | 509.570 | | 508.690 | 507.492 | 503.685 | 500.714 | 496.300 |
| ds inv (ft AD) | 513.860 | 510.620 | 509.570 | 509.052 | | 507.492 | 503.685 | 500.714 | 496.311 | 504.000 |

| Node | 1099 | 1102 | 1108 | 1110 | 1111 | 1113 | 1119 | 1291 | 1126 | 1114 |
|---|---|---|---|---|---|---|---|---|---|---|
| ground (ft AD) | 518.000 | 514.000 | 515.165 | 512.817 | 514.352 | 513.612 | 510.981 | 509.247 | 506.437 | 518.980 |

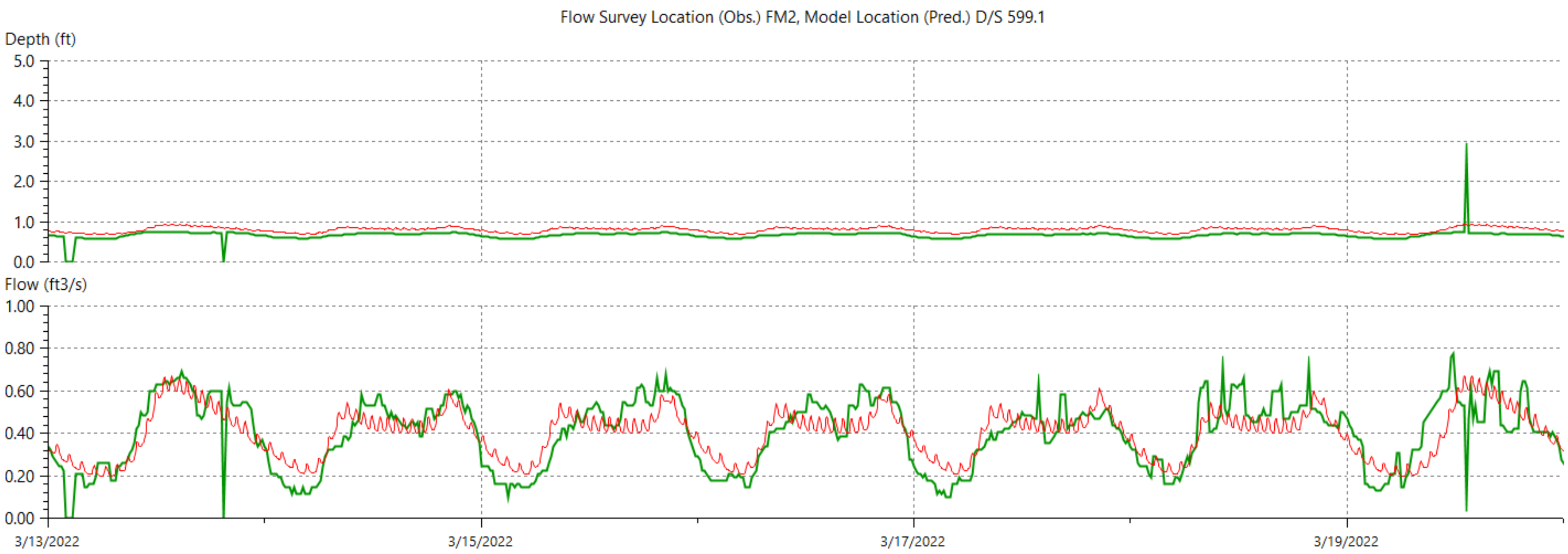
# Appendix B.  Dry Weather Calibration Plots

# Flow Meter 1 Dry Weather Calibration Plot



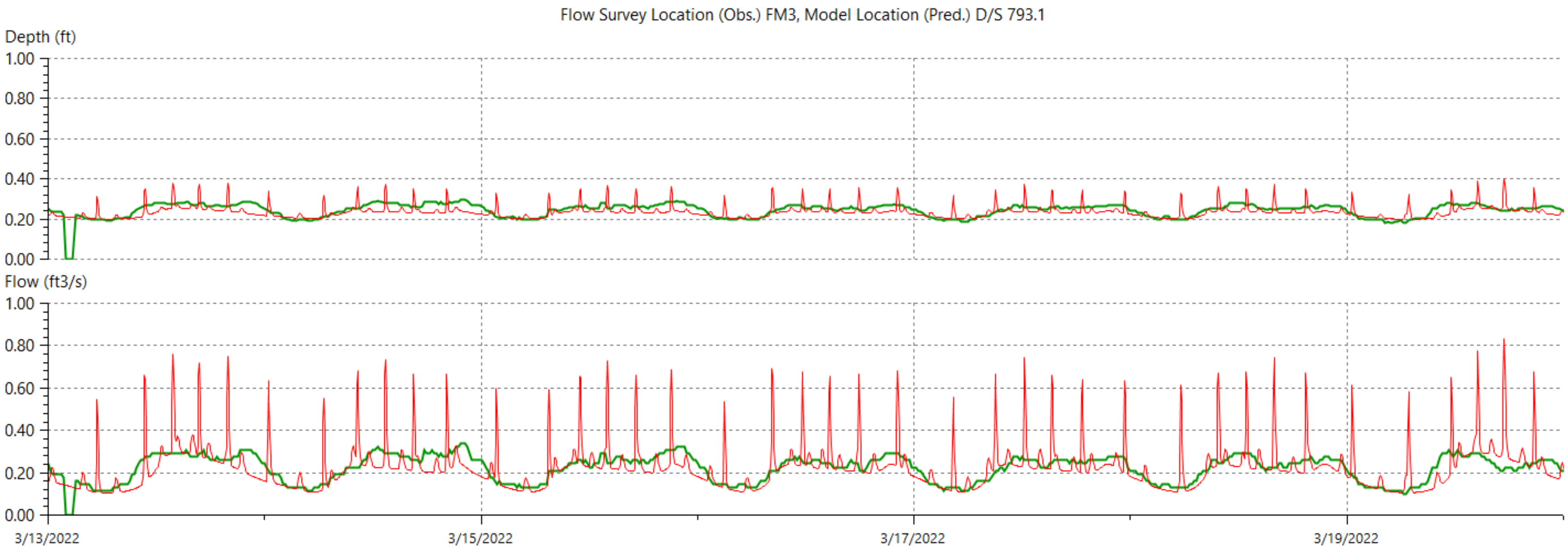

| | Depth | | Flow | | |
|---|---|---|---|---|---|
| | Min (ft) | Max (ft) | Min (MGD) | Max (MGD) | Volume (US Mgal) |
| Observed | 0.000 | 0.904 | 0.000 | 1.115 | 5.281 |
| ...l GMI (Guage only) Full>DWF | 0.320 | 0.595 | 0.399 | 1.325 | 5.651 |

# Flow Meter 2 Dry Weather Calibration Plot



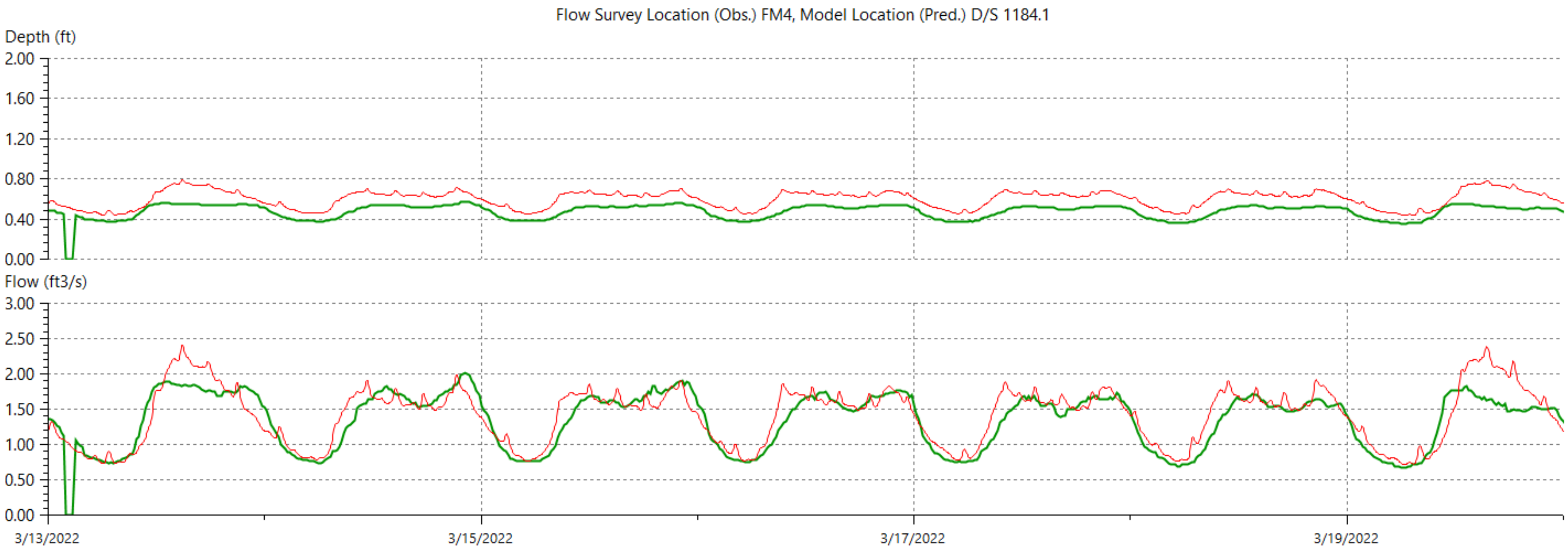

| | Depth | | Flow | | |
|---|---|---|---|---|---|
| | Min (ft) | Max (ft) | Min (MGD) | Max (MGD) | Volume (US Mgal) |
| Observed | 0.000 | 2.696 | 0.000 | 0.776 | 2.809 |
| ...l GMI (Guage only) Full>DWF | 0.682 | 0.938 | 0.197 | 0.672 | 2.806 |

# Flow Meter 3 Dry Weather Calibration Plot




| | Depth | | Flow | | |
|---|---|---|---|---|---|
| | Min (ft) | Max (ft) | Min (MGD) | Max (MGD) | Volume (US Mgal) |
| Observed | 0.000 | 0.296 | 0.000 | 0.339 | 1.544 |
| ...l GMI (Guage only) Full>DWF | 0.199 | 0.399 | 0.105 | 0.832 | 1.597 |

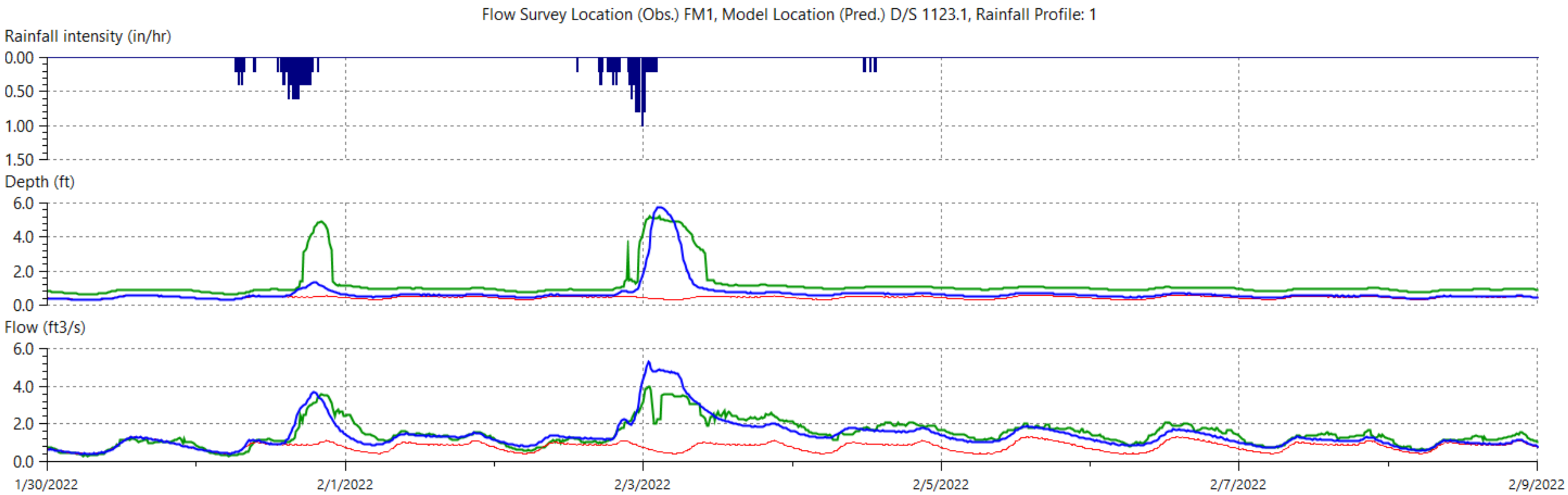
# Flow Meter 4 Dry Weather Calibration Plot




| | Depth | | Flow | | |
|---|---|---|---|---|---|
| | Min (ft) | Max (ft) | Min (MGD) | Max (MGD) | Volume (US Mgal) |
| Observed | 0.000 | 0.571 | 0.000 | 2.020 | 9.424 |
| ...l GMI (Guage only) Full>DWF | 0.439 | 0.786 | 0.720 | 2.412 | 9.932 |

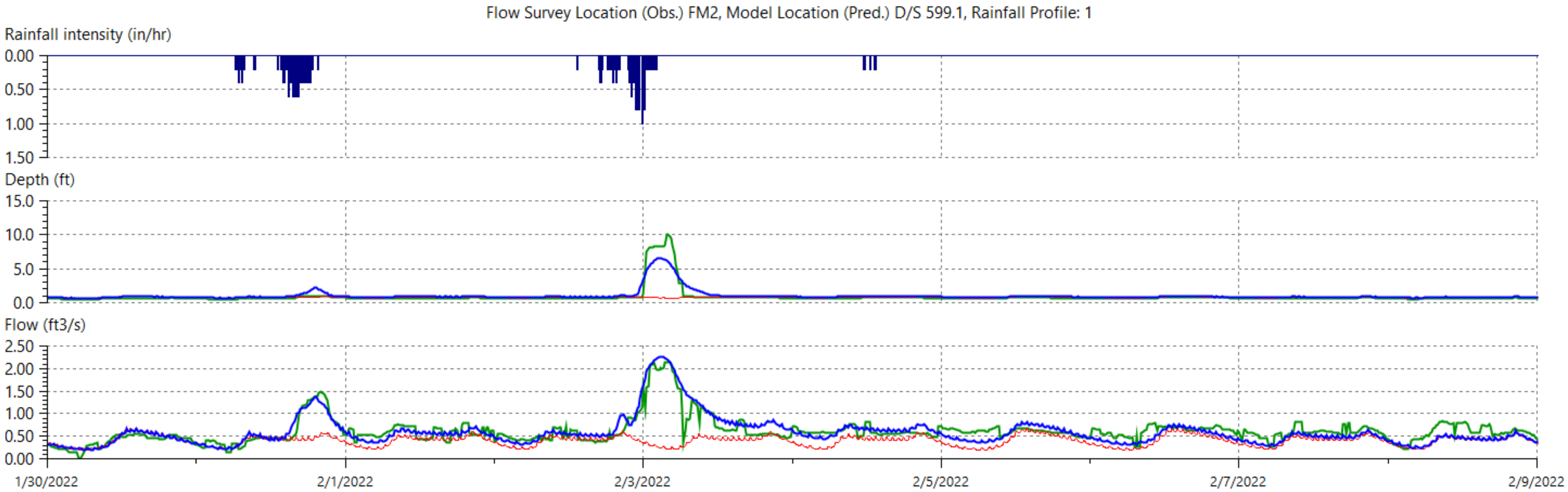
# Appendix C.  Wet Weather Calibration Plots

# Flow Meter 1 Wet Weather Calibration Plot



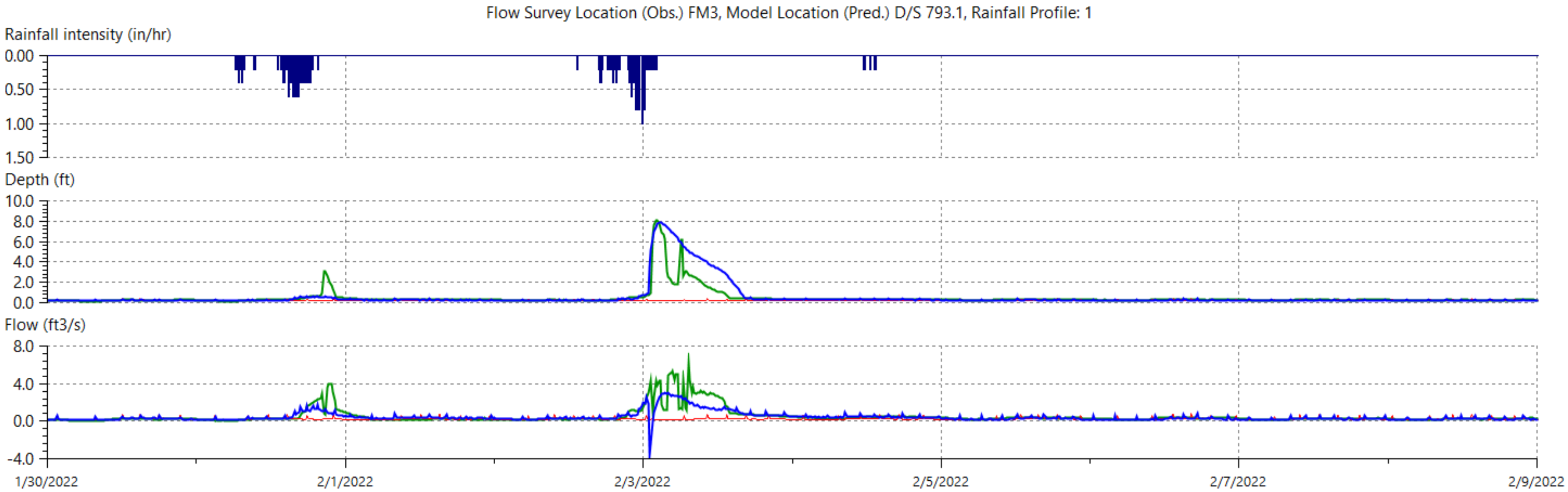

| | Rainfall | | | Depth | | Flow | | |
|---|---|---|---|---|---|---|---|---|
| | Depth (in) | Peak (in/hr) | Average (in/hr) | Min (ft) | Max (ft) | Min (MGD) | Max (MGD) | Volume (US Mgal) |
| Rain | 5.150 | 1.000 | 0.021 | | | | | |
| Observed | | | | 0.654 | 5.217 | 0.307 | 3.959 | 14.790 |
| ...age only) Full>DWF | | | | 0.320 | 0.595 | 0.398 | 1.326 | 8.077 |
| ...l Period Meter Data | | | | 0.320 | 5.769 | 0.399 | 5.299 | 14.034 |

# Flow Meter 2 Wet Weather Calibration Plot



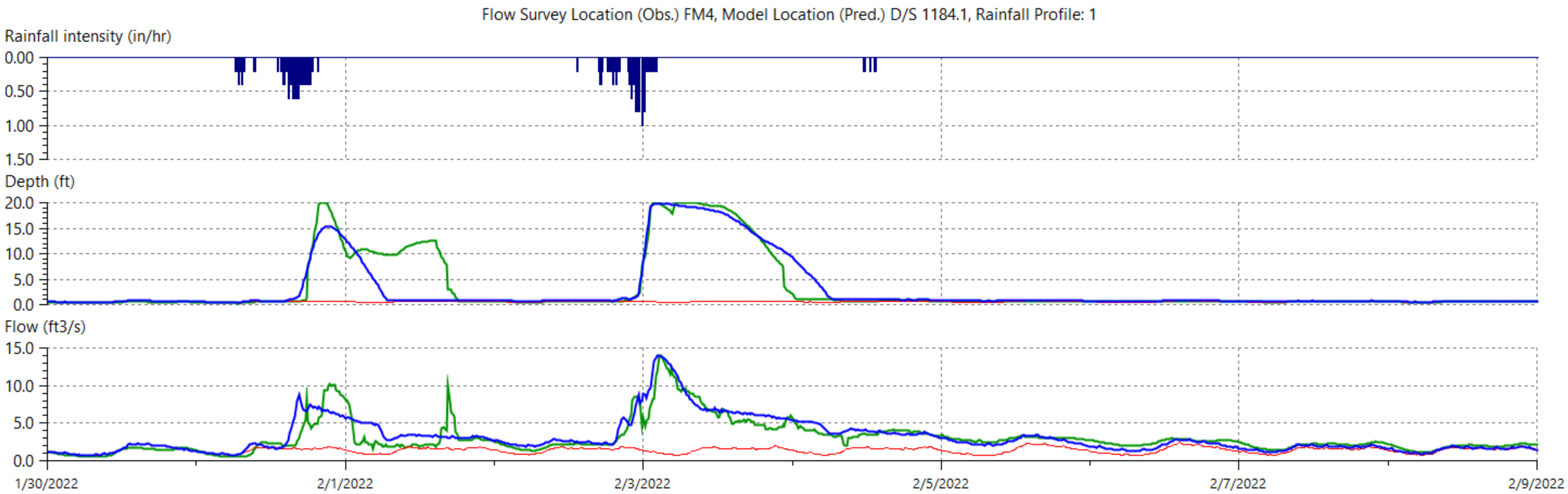

| | Rainfall | | | Depth | | Flow | | |
|---|---|---|---|---|---|---|---|---|
| | Depth (in) | Peak (in/hr) | Average (in/hr) | Min (ft) | Max (ft) | Min (MGD) | Max (MGD) | Volume (US Mgal) |
| Rain | 5.150 | 1.000 | 0.021 | | | | | |
| Observed | | | | 0.517 | 10.100 | 0.000 | 2.149 | 6.116 |
| ...age only) Full>DWF | | | | 0.683 | 0.940 | 0.197 | 0.674 | 4.012 |
| ...l Period Meter Data | | | | 0.683 | 6.531 | 0.198 | 2.254 | 5.896 |

# Flow Meter 3 Wet Weather Calibration Plot




| | Rainfall | | | Depth | | Flow | | |
|---|---|---|---|---|---|---|---|---|
| | Depth (in) | Peak (in/hr) | Average (in/hr) | Min (ft) | Max (ft) | Min (MGD) | Max (MGD) | Volume (US Mgal) |
| Rain | 5.150 | 1.000 | 0.021 | | | | | |
| Observed | | | | 0.133 | 8.121 | 0.048 | 6.108 | 4.965 |
| ...age only) Full>DWF | | | | 0.199 | 0.391 | 0.105 | 0.792 | 2.284 |
| ...l Period Meter Data | | | | 0.199 | 7.877 | -3.536 | 2.998 | 4.345 |

# Flow Meter 4 Wet Weather Calibration Plot




| | Rainfall | | | Depth | | Flow | | |
|---|---|---|---|---|---|---|---|---|
| | Depth (in) | Peak (in/hr) | Average (in/hr) | Min (ft) | Max (ft) | Min (MGD) | Max (MGD) | Volume (US Mgal) |
| Rain | 5.150 | 1.000 | 0.021 | | | | | |
| Observed | | | | 0.317 | 19.933 | 0.517 | 13.993 | 30.126 |
| ...age only) Full>DWF | | | | 0.438 | 0.790 | 0.716 | 2.428 | 14.189 |
| ...l Period Meter Data | | | | 0.438 | 19.814 | 0.717 | 14.005 | 31.363 |