Concord
NORTH CAROLINA
High Performance Living

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City of Concord

Water System Master Plan

Final Report

February 2021

Project 32180-012

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Hazen

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Executive Summary

The City of Concord authorized Hazen to develop a master plan for the water distribution system. The master plan identified capital improvements that eliminate low pressures, remedy deficient fire flows and supply future water demands. This report summarizes the project.

This project built on previous studies, including the 2017 Water System Master Plan and several 2019 and 2020 On-Call Hydraulic Modeling projects.

The master plan included updating the model with current GIS and customer billing records. The plan also included calibrating the model using field test data and SCADA records. Future demand conditions were developed and used in the model to simulate future conditions. The furthest planning period is the year 2050 with a projected maximum day demand of 41.3 mgd and average day demand of 28.5 mgd.

From the 2017 master plan, zone boundaries changes were still recommended to improve pressures. An intermediate zone shift was recommended to limit the flow out of Hillgrove WTP until raw water lines could be reinforced.

Future conditions were evaluated with the model to check for deficiencies in supply, storage, pumping and distribution capacity.

Major transmission mains are needed to carry the water into the 841 Pressure Zone to meet the Midland growth. A new 1 MG storage tank is also required to meet equalizing and fire storage requirements.

New control valves on Hwy 601 will supply water from the 890 Pressure Zone to the 841 Pressure Zone to help meet demand and supply the Pressure Zone 841 tanks. We recommend a valve that opens and closes to control water levels in the Midland tanks.

The new pump station in the existing Rockhill Booster Pump station (BPS) building will require new suction piping but will be able to use existing water mains for water conveyance from the 850 Pressure Zone into the proposed 890 Pressure Zone. The site is already owned by the city, allowing quicker construction. We recommend a 5 mgd pump station at 65 ft TDH to meet 2025 demands and beyond. The driver for this pump station is the projected demand for the Grounds at Concord. We recommend two duty pumps on VFDs and one standby pump.

The new pump station on Hwy 49 will serve two purposes. For normal operation, it will feed the new 16-inch line supplying Rocky River BPS and the existing pressure reducing valves (PRVs) that send water to the 841 Pressure Zone. We also recommend configuring the pump station so that it can pump from the 890 Pressure Zone to 850 Pressure Zone as an emergency feed if Coddle Creek WTP is out of service. We recommend a 4 mgd pump station at 44 ft TDH to meet demands by the year 2035. This pump station relies on the construction of the 24-inch along the George Liles Parkway extension from Roberta Road to Highway 49.

Other significant improvements are recommended to ensure the required distribution system capacity is met and to resolve low pressure problems. These improvements total 47.3 miles of pipe with an estimated cost of $95.7 million. There are 56 additional smaller projects to improve fire flows at an estimated total cost of $6.8 million.

Our opinion of probable construction costs for all recommended improvements is $88.9 million, divided into 5-year planning increments as follows:

- Year 2025 or MDD of 26.6 mgd: $24,500,000
- Year 2030 or MDD of 31.0 mgd: $11,440,000
- Year 2035 or MDD of 33.7 mgd: $24,970,000
- Year 2040 or MDD of 36.3 mgd: $23,894,000
- Year 2045 or MDD of 38.9 mgd: $4,104,000

Section 1 Scope of Work

The City of Concord authorized Hazen to develop a master plan for the water distribution system. The master plan identified capital improvements that eliminate low pressures, remedy deficient fire flows and supply future water demands. This report summarizes the project.

This project built on previous studies, including the 2017 Water System Master Plan and several 2019 and 2020 On-Call Hydraulic Modeling projects.

The scope of work included in a proposal dated December 17, 2019 is shown below.

A. Task 1 – Project Administration

This task includes project administration such as developing the scope and fee, preparing monthly progress reports, invoicing, and providing quality assurance by senior level staff throughout the duration of the project.

B. Task 2 – Update Existing Hydraulic Model

The Engineer will update the pipe network by importing water system data in the most recent version of the City's Geographic Information System (GIS) into the modeling software. The Engineer will compare the current GIS pipe network to the network in the existing model and add any missing pipes, simplifying where needed to improve software performance. C-factors will be assigned to reflect typical values for new pipes. New nodes will be defined at new hydrants and pipe intersections, and elevations will be assigned using digital topographic data.

The modeling software will update the demand distribution using billing data provided by the Owner. The billing data will include 12 recent months of metered use, preferably the 2019 calendar year, with meter identifiers that link to locations in GIS. The modeling software will assign each customer's water usage to the nearest model node. Flows from auto-flushers will be assigned to model nodes using information provided by the Owner. Adjustments for non-revenue water (NRW) will ensure the total demand in the model agrees with the total supplied to the system in the same 12-month period. The fifteen largest customers and auto-flushers will be excluded from NRW adjustments.

C. Task 3 - Conduct Field Tests

This task will develop a field test plan based on study of GIS data and discussions with city staff and fire chief. Hazen will deliver the plan at least 30 days before testing begins.

The Engineer will conduct one hydraulic grade line (HGL) test that measures flows and pressures along the transmission mains connecting Hillgrove WTP, the Downtown tank, Corbin Pump Station, Rocky River Pump Station, the Harrisburg control valve and the Midland Tank. Hazen will provide all test equipment. The Owner will make accessible existing taps or air valves for flow measurements using pitot tubes, or install new taps, if needed. Measured pressures will be added to gauge elevations in order to plot

HGLs against distance. This test will study the performance of improvements that have been installed since the last master plan.

Hazen will conduct 12 fire flow tests with assistance from the Owner. The tests will consist of flow and pressure measurements that assess the strength or weakness of the system in specific areas. Tests will be located for geographic coverage and in problem areas identified by city staff and or the fire department. Hazen will provide test equipment and the Owner will provide transportation and staff to operate hydrants and control traffic.

The Engineer will calculate demand patterns for 4 pressure zones using SCADA records provided by the Owner showing hourly flows from water plants and pump stations, as well as hourly changes in recorded tank levels. The owner shall provide this data in CSV or Excel format. These diurnal patterns will be used as input data for extended period simulation (EPS) modeling and for determining minimum night rates that indicate leakage potential. In addition to night rates, we will calculate non-revenue water in each zone using the SCADA data and billing records provided by the owner.

D. Task 4 - Calibrate the Model

The Engineer will first calibrate the model using the HGL test by plotting and comparing measured and predicted HGLs. These plots will show where the model needs adjustments, or locations where unusual conditions are suspected, such as closed valves. Major discrepancies that cannot be resolved with reasonable model adjustments will be reviewed with Owner to develop a plan for further investigations. This task will check macro calibration for the most important parts of the distribution system.

Next the Engineer will calibrate the model using fire flow tests. This task will check micro calibration of the model in areas where fire flow tests were performed. After checking predicted static pressures, the model will simulate the measured flow from each test. Predicted residual pressures will be compared to the measured residual pressures and reasonable model adjustments will be applied to eliminate discrepancies. Major discrepancies that cannot be resolved with reasonable model adjustments will be reviewed with the Owner to develop a plan for further investigations.

The Engineer will then calibrate the EPS model using SCADA records by comparing recorded tank water levels to predicted tank water levels. This task will ensure that the model accurately simulates tank performance, which has a significant impact on water age calculations.

Hazen will conduct an on-line webcast meeting will be held to review the calibration results.

E. Task 5 – Identify Existing Deficiencies

The model will map fire flows by calculating at each hydrant connection the flow available at a 20 psi residual pressure. Calculated flows will be plotted on a distribution system map using the color-coding format specified by the Insurance Services Office (ISO). The Engineer will identify any general areas with low fire flows, and these areas will be taken into consideration when subsequently testing improvements.

The Engineer will use the model to map water age for existing operation of the system. The model will predict water age based on a 35-day simulation of existing average daily demand using current pump controls and operating procedures. The map will highlight areas where water age is excessive.

The Engineer will use the model to evaluate pressure zone boundaries by identifying areas where pressures are outside design criteria agreed upon with city staff. These areas will be taken into consideration when subsequently testing improvements.

Hazen will conduct an on-line webcast Workshop with city staff to review the existing system analysis.

F. Task 6 – Forecast Future Water Demand

The Engineer will review population projections for Traffic Analysis Zones (TAZs) within the future water system service area agreed upon with the Owner. Hazen will conduct a workshop in person with city staff to review the TAZ projections as well as other planning information made available by the Owner. The TAZ projections will be adjusted if needed based on input from city staff.

The engineer will forecast water demands to 2050 in five-year increments based on the population projections for the service area. This task will include an evaluation of peaking factors and diurnal patterns in order to estimate future average day, maximum day and peak hour demands in each pressure zone. Demand forecasts will consider:

- historical water production records
- the Owner's billing records and meter installation trends
- information provided by city staff about new industrial use and its most likely location
- wholesale supply to other systems considering contractual agreements and input from city staff
- per capita usage trends identified from billing records
- projected population growth

The Engineer will summarize demand projections and sources in a technical memo. The demand projections will be agreed upon with the Owner before modeling any future demand scenarios. The tech memo will show how projected demands are supplied from the water treatment plants and purchased sources, including Albemarle, Kannapolis and specific Charlotte Water meters.

Based on the demand projections, the Engineer will evaluate pump and storage capacity by pressure zone. This task will include evaluating storage requirements for equalizing diurnal demand, sustaining fire flows and meeting state regulations for emergencies. The Engineer will compare needed storage in each pressure zone with existing tank capacities and recommend new tanks in zones with inadequate storage. The Engineer will evaluate pump capacity by comparing maximum day demands with firm pumping capacity. New pumps will be proposed as needed.

The Engineer will distribute new demands to model nodes based on TAZ projections and discussions with the Owner. The model's total demand for future scenarios will agree with the demand projections.

G. Task 7 – Simulate Future Demand Conditions

The Engineer will identify future deficiencies by simulating future supply and demand conditions and comparing predicted performance with design criteria agreed upon with the Owner.

Modeling will focus special attention on purchased supplies from Charlotte, including total purchase amount, as well as meter locations and flow rates.

Hazen modelers will test improvements that eliminate deficiencies. Viable alternatives will be compared to identify cost-effective methods of supplying future demands while meeting hydraulic design criteria and maintaining water quality. Improvements will take full advantage of the existing distribution system. Improvements will include adjustments to pressure zone boundaries, if needed. Hazen will conduct an in-person workshop to review the improvements, and final recommendations will be based on review comments from city staff.

H. Task 8 – Update the CIP

Hazen will update the capital improvement plan (CIP) from the previous master plan by mapping, prioritizing, and tabulating recommended pipes, tanks, and pump stations. The CIP will include planning level cost estimates and project sheets for each major improvement in initial phases. Cost estimates will include construction, land acquisition, contingencies, engineering, legal and administrative costs. The CIP will be divided into 5-year planning periods corresponding to a color-coded map. Project sheets will identify any demand benchmarks that trigger the need for proposed improvements.

The Engineer will review the new CIP with city staff through an on-line webcast meeting and make changes based on review comments.

I. Task 9 - Prepare Master Plan Report

The Engineer will prepare a draft report that:

- documents the process of updating the hydraulic model
- summarizes the field tests and model calibration
- identifies existing deficiencies
- maps water age and summarizes recommendations for improving operations and water quality
- summarizes the population and demand projections
- explains model results for future demand conditions
- tabulates proposed improvements, with planning level cost estimates

Hazen will respond to review comments and deliver a final report that is signed and sealed by a Professional Engineer. Deliverables will include five paper copies of the final report with flashdrives that contain electronic versions of the report, maps and model (EPAnet).

J. General Information

Concord's distribution system provides drinking water to retail customers in the City of Concord, the Town of Midland and unincorporated areas of Cabarrus County. Concord sells water to the City of Kannapolis and the towns of Harrisburg and Mount Pleasant. The average daily demand including these wholesale customers is approximately 13.4 mgd and maximum day demand was 17.0 mgd in 2019.

Two water treatment plants (WTPs) supply the distribution system from the Rocky River sub-basin of the Pee Dee River Basin. Coddle Creek WTP is supplied from Lake Don T. Howell and Hillgrove WTP is supplied from Lake Fisher on Cold Water Creek. Concord purchases approximately 3 mgd from the City of Albemarle and smaller amounts from Charlotte Water and the City of Kannapolis, currently. The city is working toward getting 4 mgd from Albemarle.

The distribution system includes four pressure zones that supply about 92,240 people through more than 45,482 meters. The pipe network includes over 753 miles of water mains ranging from 1-inch to 48-inch in size. Some of the water mains are unlined cast iron pipes that have been in service more than 80 years.

Figure 1-1 is a map of the existing distribution system showing trunk mains and pressure zones based on the information collected from the city. Table 1-1 and Table 1-2 provide general information about the storage tanks and pumps.

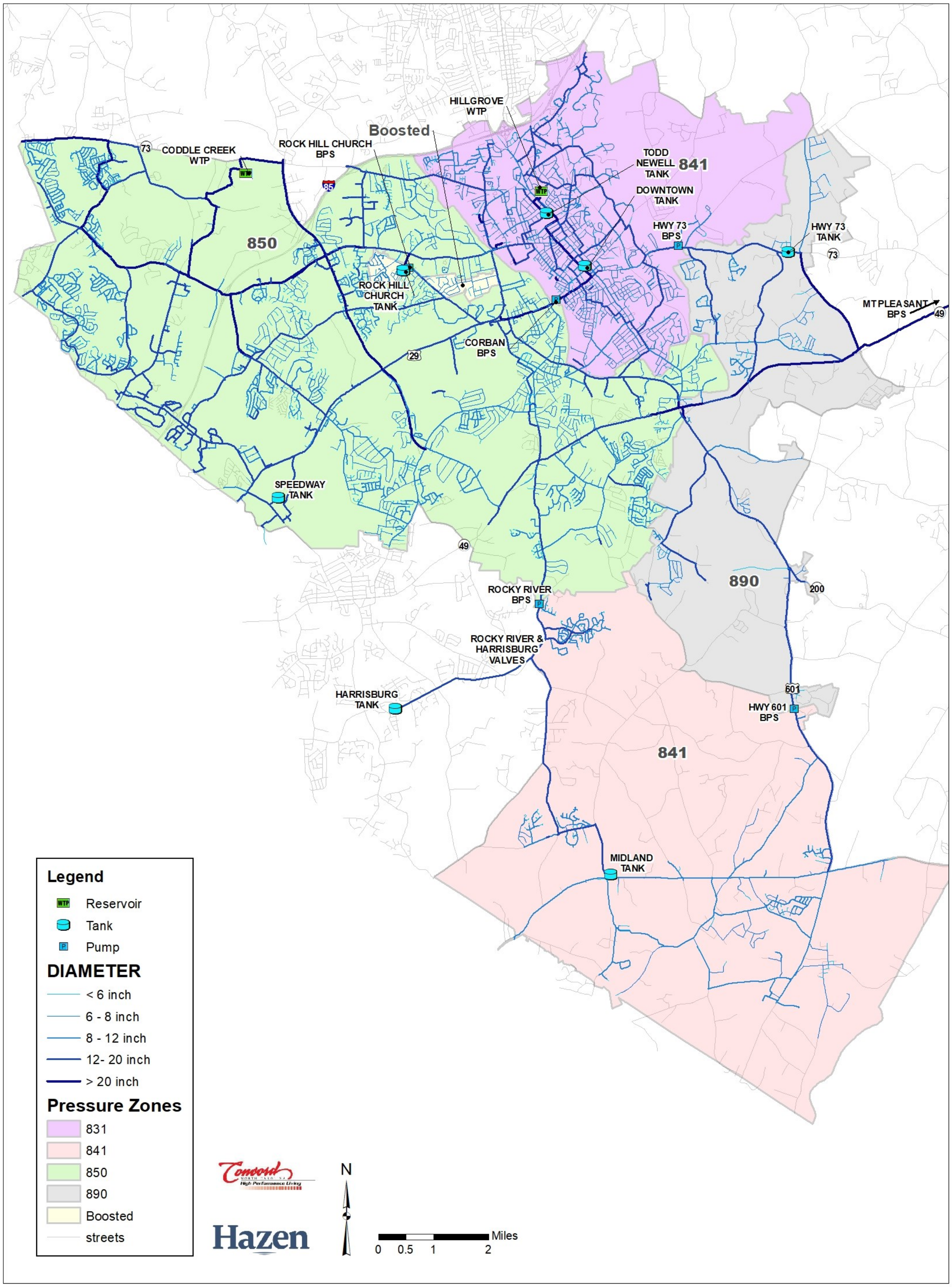


Figure 1-1: Existing Water Distribution System

Table 1-1: Storage Tank Information

Tank	Pressure Zone	Overflow Elevation (ft.)	Bowl Elevation (ft.)	Range (ft.)	Normal Capacity (MG)
Downtown	831	831	791	40	0.5
HWY 73	890	890	850	40	0.75
Midland	841	841	801	40	0.5
Rock Hill Church	850	850	810	40	2.0
Speedway	850	850	810	40	1.0
HWY 29	890	890	850	40	2.0
Todd Newell	831	831	801	30	1.0
Coddle Creek Clear Well 1	WTP	631	619	12	2.0
Coddle Creek Clear Well 2	WTP	631	619	12	2.0
Hill Grove Clear Well 1	WTP	725	710	15	2.0
Hill Grove Clear Well 2	WTP	725	710	15	2.0
Mt Pleasant Ground Storage	860	799.5	759.5	40	1.0

There are five existing pressure zones: 831, 850, 890, 841, and a boosted zone, as depicted in Figure 1-2. The 831 Pressure Zone includes the Downtown and Todd Newell tanks and is supplied by Hillgrove Water Treatment Plant (WTP). The 890 Pressure Zone consists of the Hwy 73 tank and is supplied by the Mt Pleasant Pump Station and HWY 73 PS. The 850 Pressure Zone is controlled by the Rock Hill Church and Speedway Tanks supplied by the Coddle Creek WTP and by the Corban Pump Station. The Rock Hill Church PS pumps water from 850 Pressure Zone to Boosted Pressure Zone. The 841 Pressure Zone is named for the overflow elevation of Midland Tank and receives water from 850 Pressure Zone via Rocky River PS and from 890 Pressure Zone via HWY 601 PS.

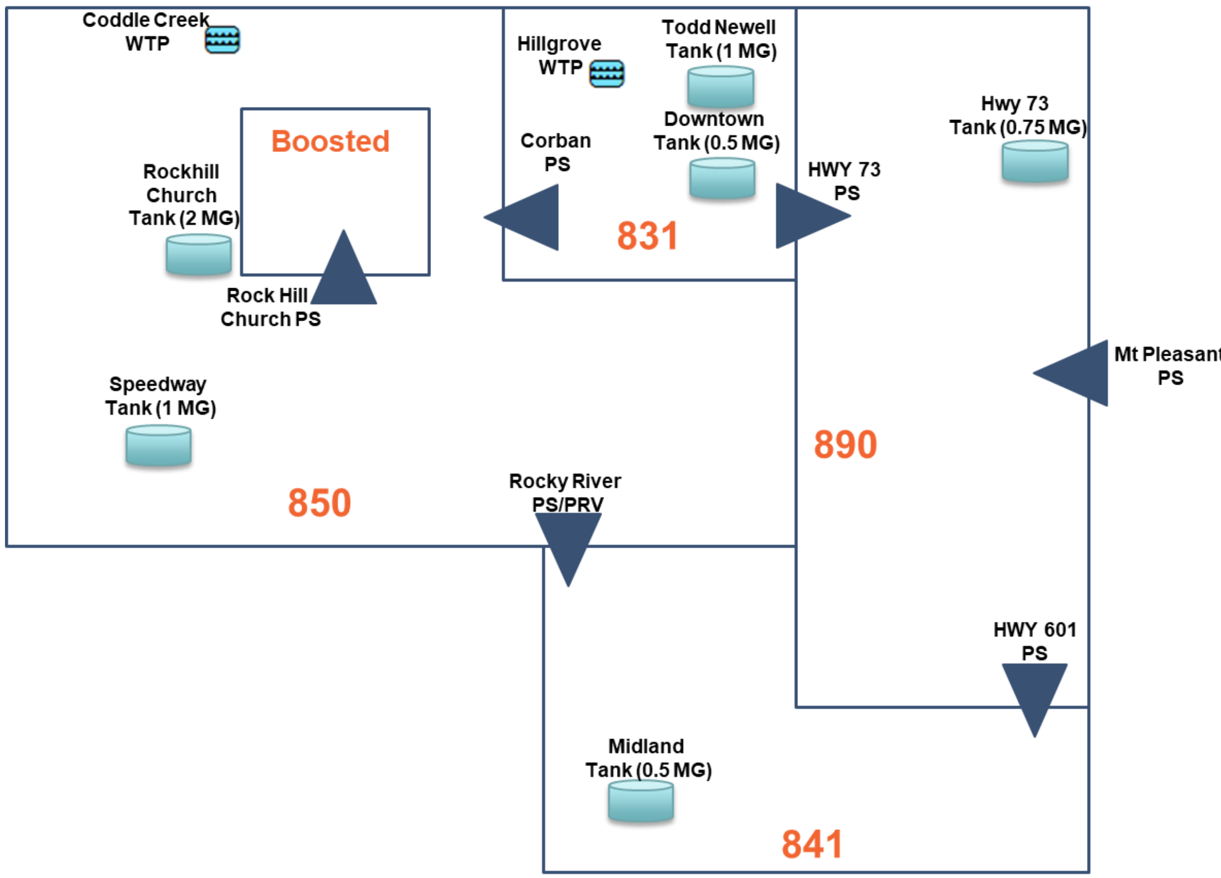


Figure 1-2: Existing Pressure Zones

Table 1-2: Pump Information

Location	Inlet Pressure Zone	Outlet Pressure Zone	Pump Number – Meter Type	Rated Capacity		Total Dynamic Head (ft.)	Horsepower
				mgd	gpm		
Hillgrove WTP High Service PS	(WTP)	831	1 - Electric	2.52	1750	140	75
			2 - Electric	3.96	2750	150	125
			3 - Electric	6.00	4200	150	250
			4 - VFD	9.00	6250	156	300
Coddle Creek High Service PS	(WTP)	850	1 - Electric	2.60	1800	260	250
			2 - Electric	2.60	1800	260	250
			3 - Electric	4.00	2800	305	400
			4 - Electric	4.00	2800	305	400
			5 - Electric	4.00	2800	305	400
Highway 601	890	841	1 - Electric	0.70	500	150	40
			2 - Electric	0.70	500	150	40
Rocky River PS	850	841	1 - VFD	2.80	1930	200	150
			2 - VFD	2.80	1930	200	150
Highway 73	831	890	1 - VFD	1.25	870	150	50
			2 - VFD	1.25	870	150	50
Rock Hill Church Road PS	850	Boosted	1 - Electric	1.30	900	243	65
			2 - Electric	0.50	350	80	14
			3 - Electric	0.13	89	125	10
Mt Pleasant PS	Mt Pleasant	890	1 - Electric	2.00	1390	164	72
			2 - Electric	5.00	3472	201	211
			3 - Electric (inactive)	5.00	3472	201	211
Corban PS	831	850	1 - VFD	2.30	1600	65	34
			2 - VFD	2.30	1600	65	34
			3 - VFD	2.30	1600	65	34

Section 2 Hydraulic Model Update

A. Update Pipe Network, Pump Stations (PS), and Tanks

Pipes that were in the 2019 GIS, but not in the model, were added to the model, as well as pipe diameter and material. Different C factors were assigned to pipes based on their materials as below,

- C factor of 140 to polyvinyl chloride (PVC) pipes
- C factor of 120 to Ductile Iron (DI) pipes

Existing pipes were compared with 2019 GIS and several problem areas were found and checked by City staff. Accurate pipe information was added to the model in accordance with staff input.

The Corban PS was updated from record drawings and pumps were updated with manufacture's pump curves (Peerless Pump – 8AE12).

Highway (HWY) 29 elevated water storage tank was added to the model to reflect record drawings, July 20, 2018.

B. Demand Distribution Update

Hazen received billing data for the 2019 calendar year from the City including 12 months of metered use with meter identifiers that link to locations in GIS. Each customer's water usage was assigned to the nearest model node.

The 2019 annual average water demand was 13.4 mgd, including water pumped from the water treatment plants and water purchased from other systems. Figure 2-1 summarized the average water demand. Fifteen large customers averaging approximately 1.19 mgd accounted for 9 percent of the total demand. Approximately 22 percent of the total was attributable to wholesale customers, including Charlotte Water, Harrisburg, and Kannapolis. Seventeen auto-flushers were assigned to model nodes with a total flow of 0.38 mgd, accounting for 3 percent of the total demand. The remaining small commercial and residential users accounted for 60 percent of the total demand.

The total water sold to all 45,482 customers was 12.46 mgd, leaving 0.92 mgd as non-revenue water, defined as the difference between total water sold and the total supplied to the system. The model demand was adjusted at each node with small commercial and residential use.

Historical peaking factors were used to estimate future maximum day and peak hour demands from average day projections. Production records show in Figure 2-2 that the maximum day peaking factor has exceeded 130% frequently but exceeded 140% only once in the last 12 years. Future demand projections in this report assumed a maximum day peaking factor of 140% to be conservative for available capacity in the future.

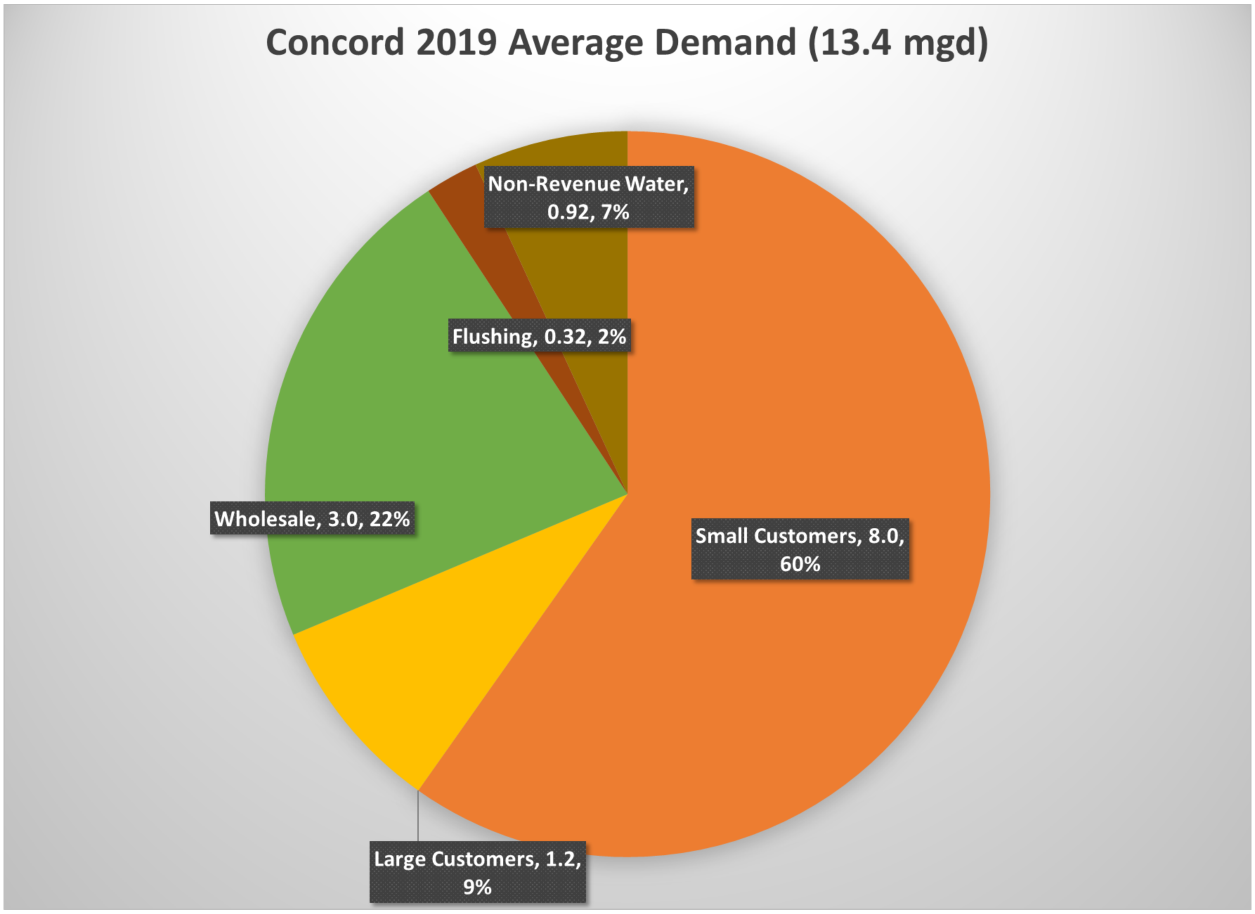


Figure 2-1: Summary of 2019 Average Water Demand

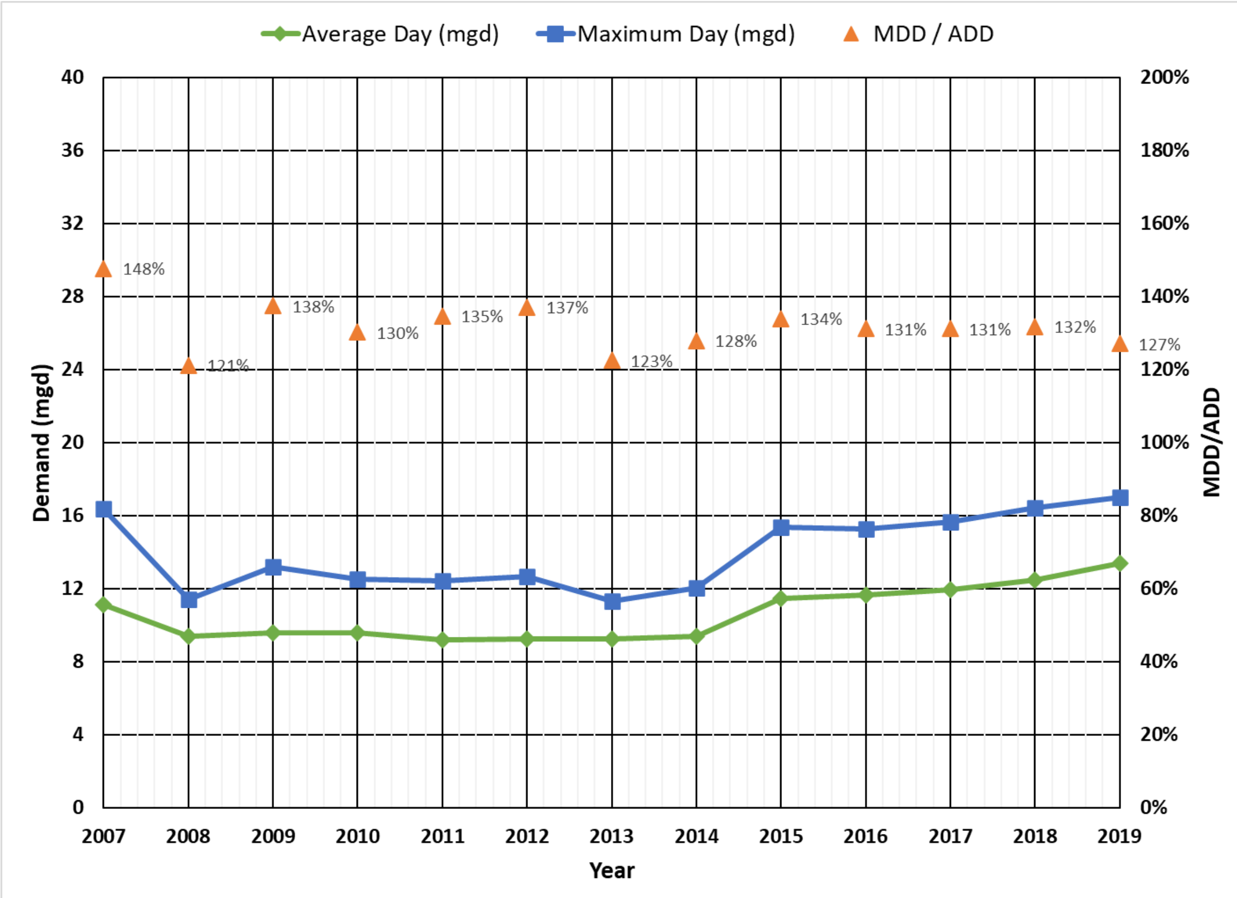


Figure 2-2: Historical Production Records and Peaking Factors

Section 3 Field Tests

A. Hydraulic Grade Line Test (HGL)

Hydraulic grade line tests show how head loss accumulates with distance from supply sources such as treatment plants and pump stations. HGL is elevation plus pressure expressed in feet of water. Hydraulic grade line tests consist of simultaneous flow and pressure measurements along trunk mains between water supply sources and tanks or flowing hydrants. In general, steep slopes between measurement points indicate overloaded pipes. We obtained flows from pitot tube measurements or the city's SCADA system. We measured pressures with calibrated pressure gauges. A map of the HGL path is shown in Figure 3-1. The following tables show the results of the HGL test. Measured HGLs are plotted against distance in a subsequent section covering model calibration.

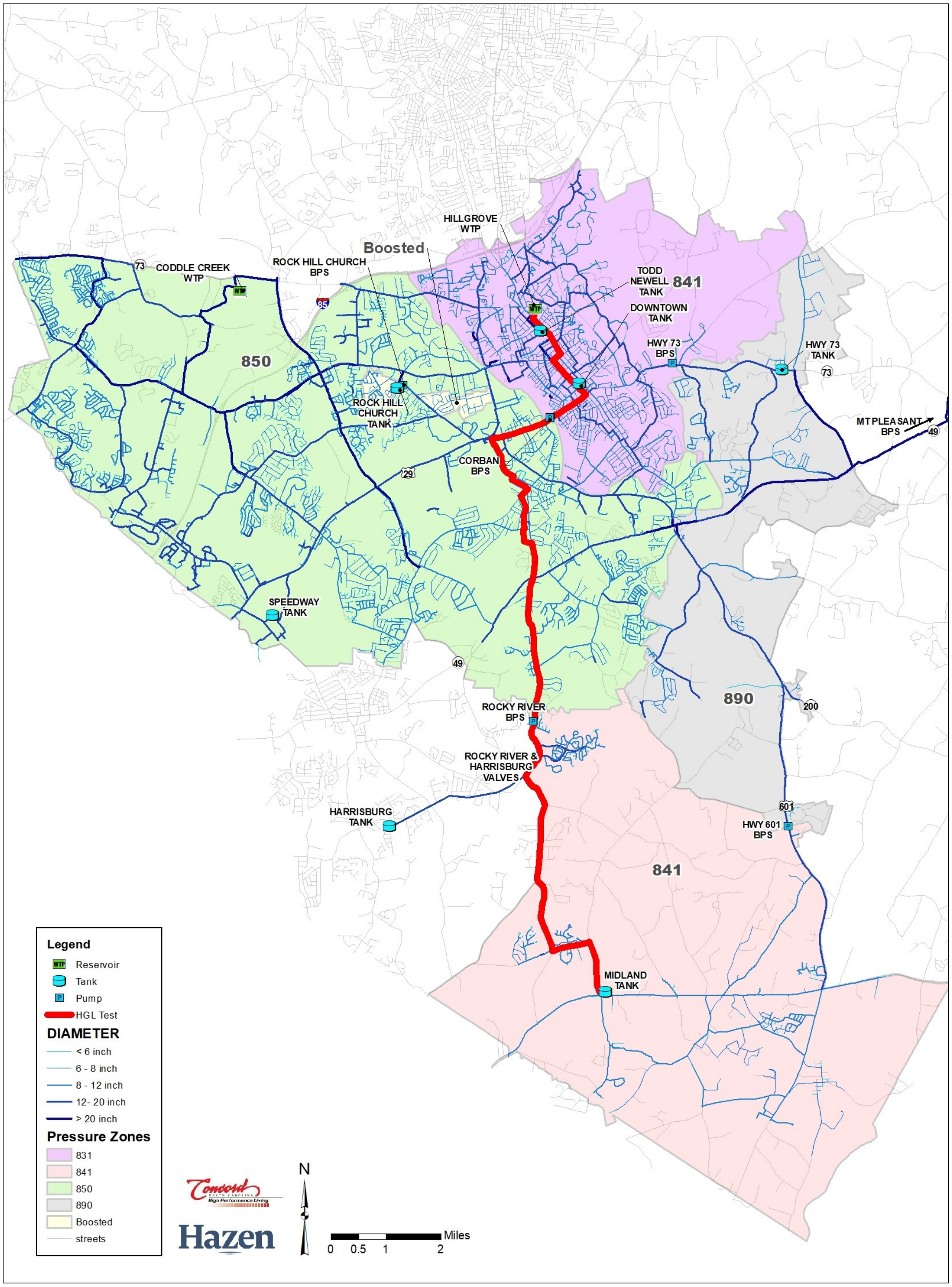


Figure 3-1: Hydraulic Grade Line Test Path

The HGL Test was conducted on June 23, 2020 starting from Hillgrove WTP, through Midland PRV, and ending at Midland Tank. Results are shown in Table 3-1 and in Figure 3-2.

Table 3-1: HGL Test

Location	Distance (ft.)	Elevation (ft.)	Measured Pressure Head (ft.)	Measured Hydraulic Grade (ft.)	Measured Flow MGD
Hillgrove WTP - Arbor St & Palaside Dr	0	730	81	811	3.10
Newell Tank - Church St & Todd Dr	3,036	740	71	811	
Downtown Tank - Spring St & Cabarrus Ave	11,102	700	111	811	
Corban PS - Suction	16,540	598	213	811	
Corban PS - Discharge	16,540	598	238	836	1.10
Cabarrus Ave -E- US 601	21,344	750	81	831	
210 Highland Ave	27,756	739	86	825	
Old Charlotte Rd -S- NC 49	38,379	705	106	811	
Rocky River PS - Suction	52,224	551	228	779	
Rocky River PS - Discharge	52,224	551	302	853	1.89
Rocky River PS - Discharge Hydrant	52,424	551	301	852	
Harrisburg PRV - Upstream	58,088	607	239	846	
Harrisburg PRV - Downstream	58,088	604	234	838	1.06
Morrison Rd - E Cedarvale Farm Pkwy	77,110	622	208	830	
Midland Tank	85,440	801	27	828	

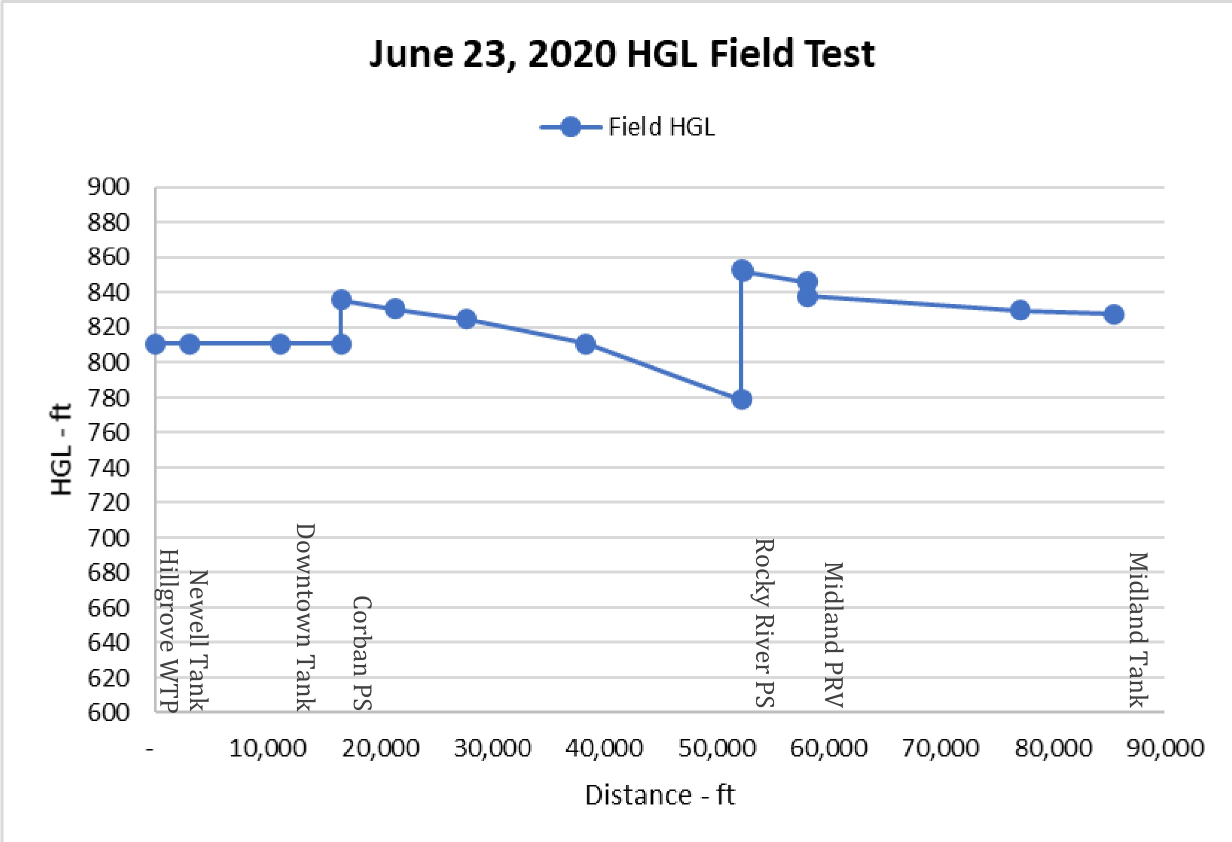


Figure 3-2: HGL Test

A. Fire Flow Tests

Eleven fire flow tests were conducted on April 2, 2020, focusing on areas of concern identified by the city. Figure 3-3 shows the location of the tests.

Test results are summarized in Table 3-2. Static and residual pressures were measured using calibrated test equipment. From the test data, the available flow at 20 psi was calculated for each test location. The needed fire flow came from the 2013 study and was estimated based on building size, type of development and spacing between buildings.

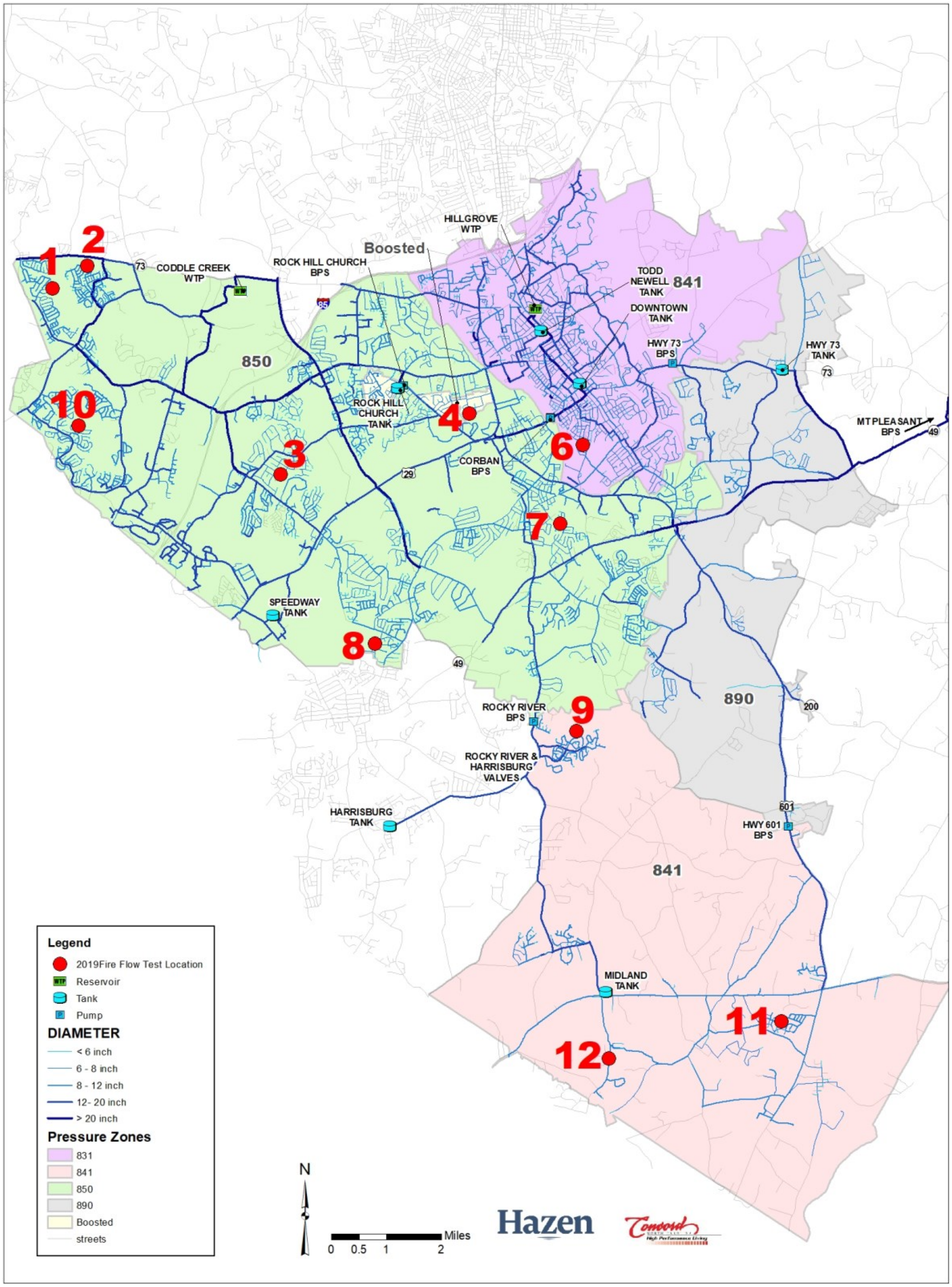


Figure 3-3: Fire Flow Test Locations

Table 3-2: Field Measurements and Calculated Available Fire Flow

Test No	Location	Pressure Zone	Pressure		Test Flow (gpm)	20-psi Flow (gpm)	Estimated Needed Flow (gpm)
			Static (psi)	Residual (psi)			
1	1317 Middlecrest Dr	850	75	52	1,810	2,900	1,500
2	Dartmoor Ave & Ravenscroft Ln	850	70	54	2,000	3,700	1,000
3	6135 Ferncliff Dr	850	75	62	1,620	3,500	1,000
4	Groff St & Montford Ave	890	94	45	1,000	1,200	1,000
5*	911 Davidson Dr	-	-	-	-	-	-
6	Lincoln St & H Goodman Cir	831	90	74	2,700	6,000	1,000
7	Amhurst St & Rockingham Ln	890	74	29	1,300	1,400	1,000
8	Juanita Dr & Kim St	850	105	68	1,020	1,600	1,000
9	Millstone Cir & Millstone Cir	890	111	66	1,670	2,400	1,000
10	Aragorn Ln & Anduin Falls Dr	850	74	52	1,060	1,700	1,500
11	Brickwood Cir & Hill Pine Rd	841	120	91	1,040	2,000	1,000
12	Old Camden Rd & Community Dr	841	81	62	1,110	2,100	1,000

*Did not run Fire Flow Test 5, due to Covid-19 and proximity to the hospital.

B. Diurnal Demand Patterns

The previous master plan developed specific diurnal patterns for Town of Harrisburg, City of Kannapolis and Perdue Farms. These specific patterns were left as previously determined. However, the general pressure zone diurnal demand patterns were reevaluated to determine if they needed to be updated from the existing patterns currently in the model. Diurnal demand patterns were developed for each pressure zones using SCADA records provided by the City.

Two sets of SCADA data were received,

- Winter data: from January 31, 2020 to March 11, 2020
  - flow rate data was missing for Coddle Creek WTP, Rocky River PS, and Hwy 601 PS
- Summer data: from June 1, 2020 to July 11, 2020
  - Flow rate for Coddle Creek WTP was only provided periodically, not hourly

The diurnal patterns calculated from the two sets of SCADA for the 831 Pressure Zone were compared with the existing model diurnal pattern. The calculated winter weekday, winter weekend and summer data patterns were found to be similar to the pattern already in the model, shown in Figure 3-4. Therefore, we left the existing pattern in the model for this zone.

Due to data gaps, determining the patterns for the other pressure zones during winter was not possible. The summer diurnal patterns calculated were compared to the existing pattern as shown in in Figure 3-5, Figure 3-6,and Figure 3-7. The summer data patterns were atypical and appeared to be impacted by variations in water usage due to COVID-19. The current patterns were more in line with expected patterns; therefore, the existing model diurnal patterns were used for all modeling simulations.

The 841 Pressure Zone, existing diurnal pattern appeared to account for the Corning Industry water usage. There was an early morning peak at 3pm in this diurnal pattern, which is not typical for residential, commercial, or institutional usage. The 841 Pressure Zone does not have a lot of industry besides Corning Industry; therefore, the existing diurnal pattern wasn't representing the zone demands. We opted to use the 850-diurnal pattern for the 841 Pressure Zone, as the 850 Pressure Zone includes a good combination of usage types. The existing 841 Pressure Zone pattern was applied to the Corning Industry demand.

Figure 3-8 summarizes all the diurnal patterns used for the various pressure zones.

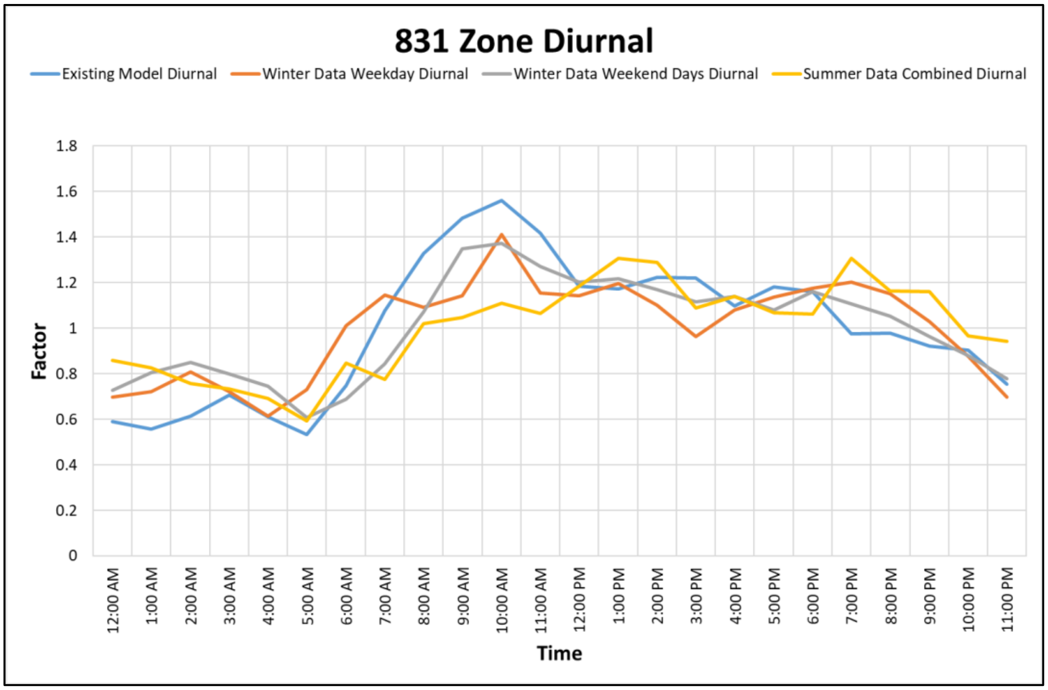


Figure 3-4: Pressure Zone 831 Diurnal Patterns

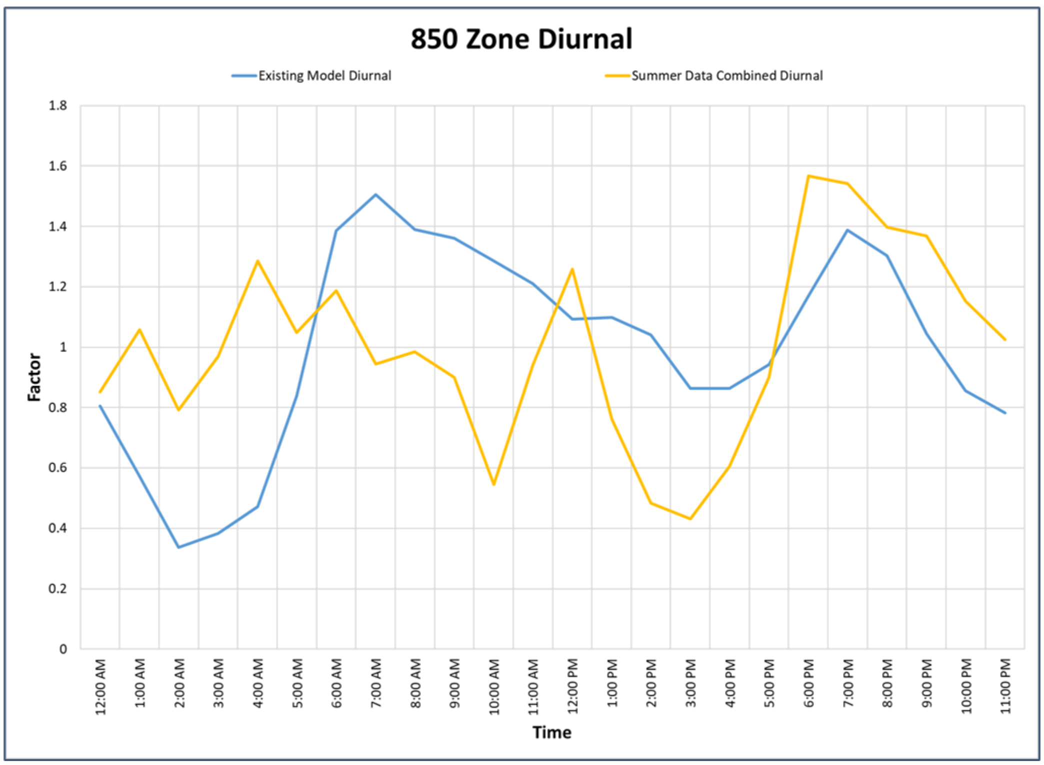


Figure 3-5: Pressure Zone 850 Diurnal Patterns

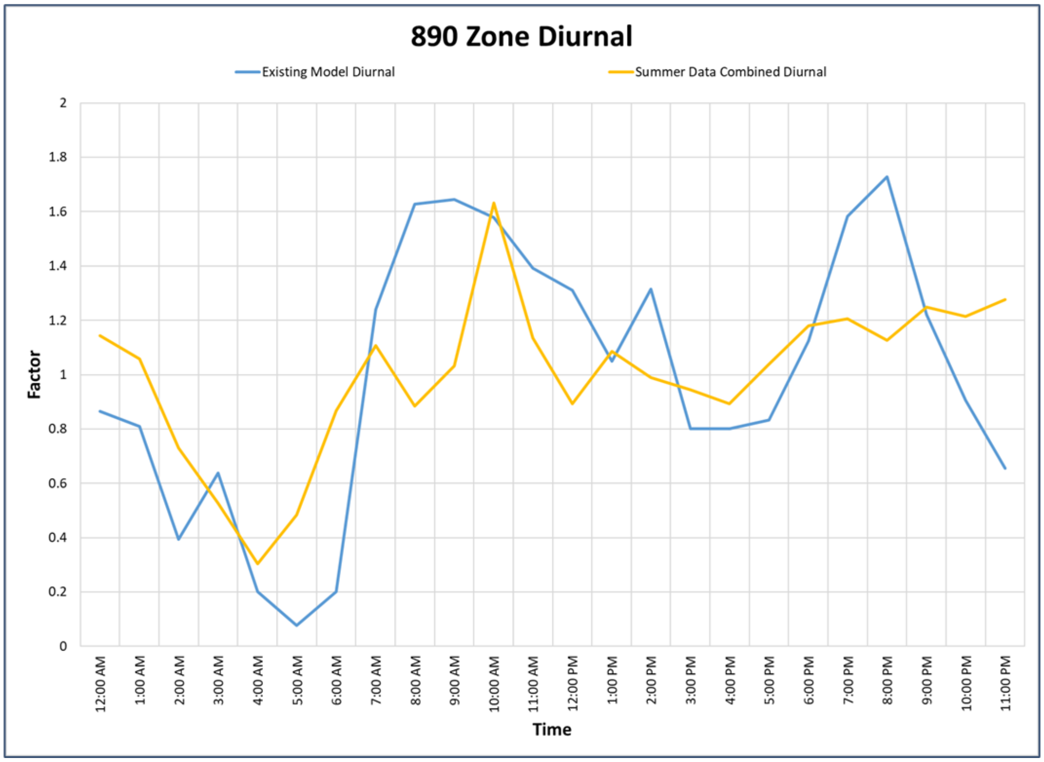


Figure 3-6: Pressure Zone 890 Diurnal Patterns

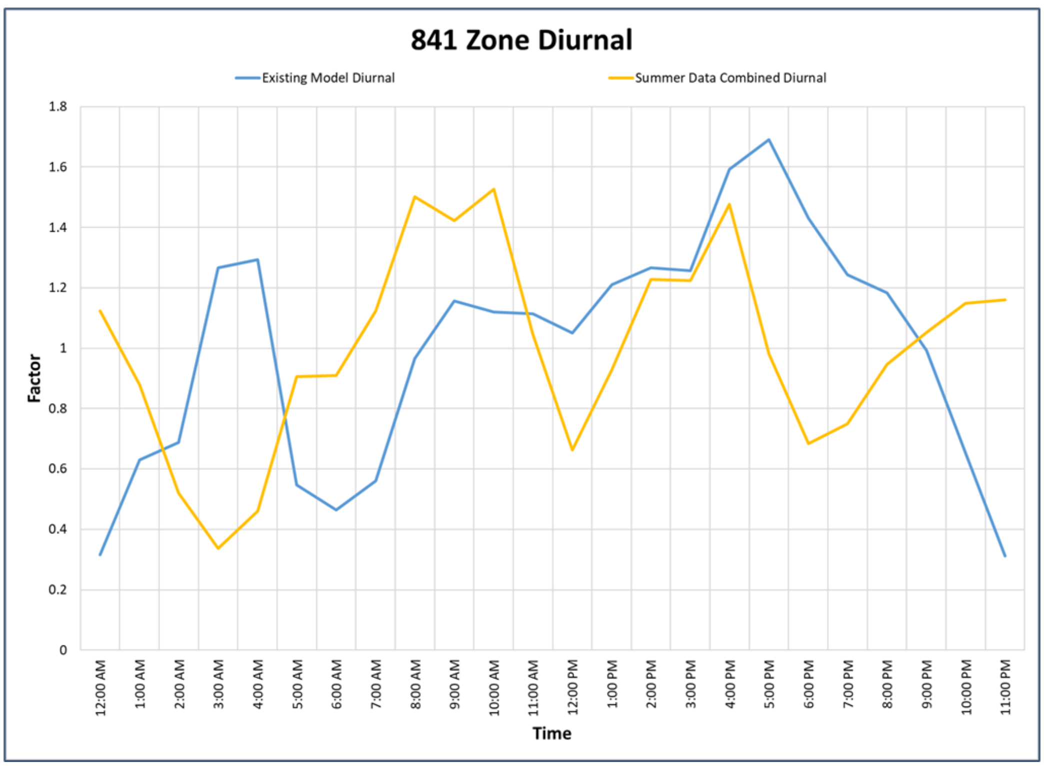


Figure 3-7: Pressure Zone 841 Diurnal Patterns

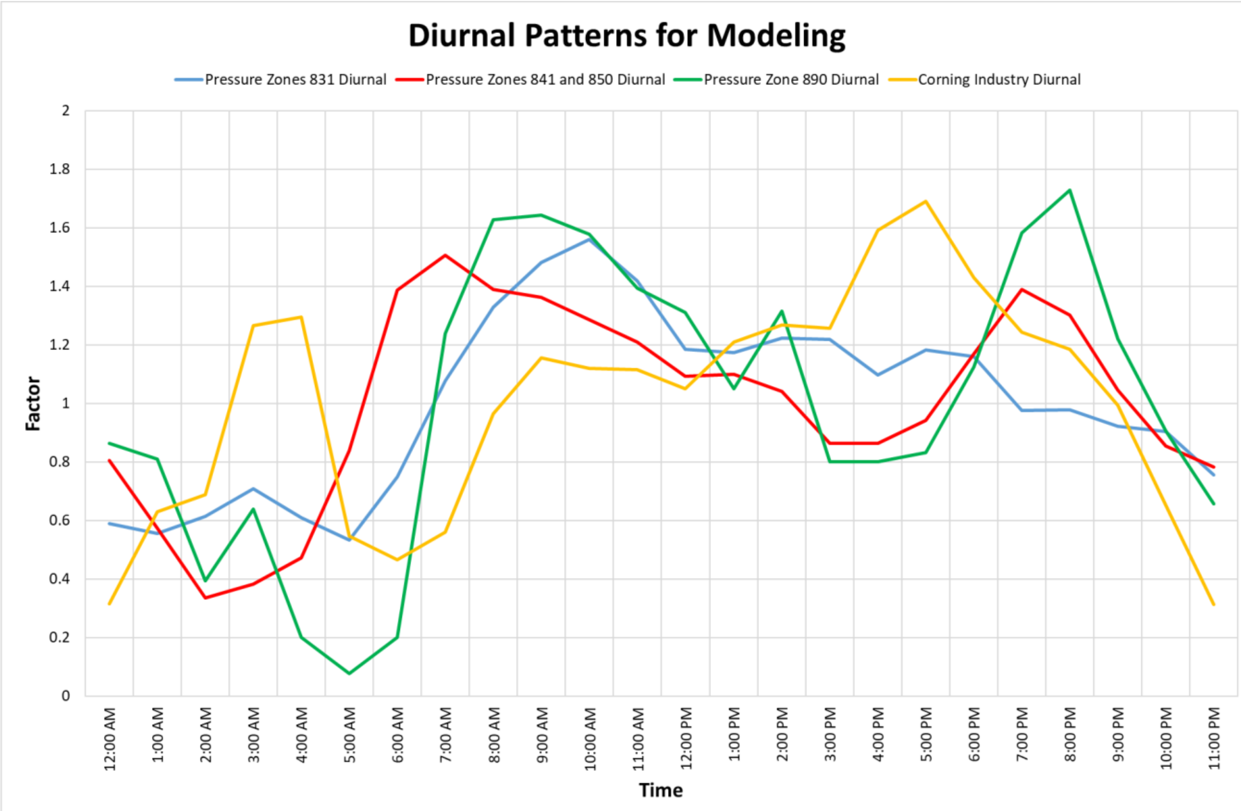


Figure 3-8: Summary of Diurnal Patterns Used for Modeling Future Scenarios

Section 4 Model Calibration

Calibration is comparing the model's predictions with test data and adjusting the model to obtain agreement within reasonable tolerances. Calibration often uncovers unusual conditions in the field such as closed valves, anomalies in input data, or SCADA inaccuracies. We calculated the model using a three-step process:

1. Macro calibration using hydraulic grade line tests
2. Micro calibration using fire flow tests
3. Extended Period Simulation (EPS) calibration using SCADA recorded tank levels and pump flow rates.

Macro-calibration using hydraulic grade line tests refers to calibrating key components of the model such as tanks, pumps, and transmission mains. This process helps identify gross model errors such as erroneous connectivity or inaccurate operational settings. This type of calibration can also detect unusual conditions in the distribution system such as closed valves. Errors in the model along transmission mains and at pump stations will propagate to the rest of the distribution system, so macro-calibration is very important.

A. Hydraulic Grade Line Test Calibration

The pressures from Corban PS discharge, through Rocky River PS suction and discharge, to Harrisburg PRV could not be matched initially, as well as the flow rate at Harrisburg PRV. Several different adjustments were made in the model, such as

- Hillgrove WTP variable frequency drive (VFD) pumps adjusted to match SCADA flow.
- Speed of Corban PS pumps adjusted to match SCADA.
- Adjusted estimate of bleeder flow between Zones 890 and 850.
- Adjusted speed of Rocky River PS to match SCADA flow.
- Adjusted Midland PRV to have 5 psi drop.
- Turned US 601 PS off.

After these adjustments, model predicted HGLs matched field measurements within 4 feet and flow was predicted by the model within 10%. Table 4-1 and Figure 4-1 compare the adjusted model predictions to the measured results.

Table 4-1: HGL Test Comparison of Predicted and Measured Results after Model Adjustments

Location	Distance (ft.)	Measured HGL (ft.)	Predicted HGL (ft.)
Hillgrove WTP - Arbor St & Palaside Dr	0	811	812
Newell Tank - Church St & Todd Dr	3,036	811	812
Downtown Tank - Spring St & Cabarrus Ave	11,102	811	812
Corban PS - Suction	16,540	811	811
Corban PS - Discharge	16,540	836	835
Cabarrus Ave -E- US 601	21,344	831	835
210 Highland Ave	27,756	825	829
Old Charlotte Rd -S- NC 49	38,379	811	812
Rocky River PS - Suction	52,224	779	778
Rocky River PS - Discharge	52,224	853	855
Rocky River PS - Discharge Hydrant	52,424	852	855
Harrisburg PRV - Upstream	58,088	846	849
Harrisburg PRV - Downstream	58,088	838	837
Morrison Rd - E Cedarvale Farm Pkwy	77,110	830	832
Midland Tank	85,440	826	826

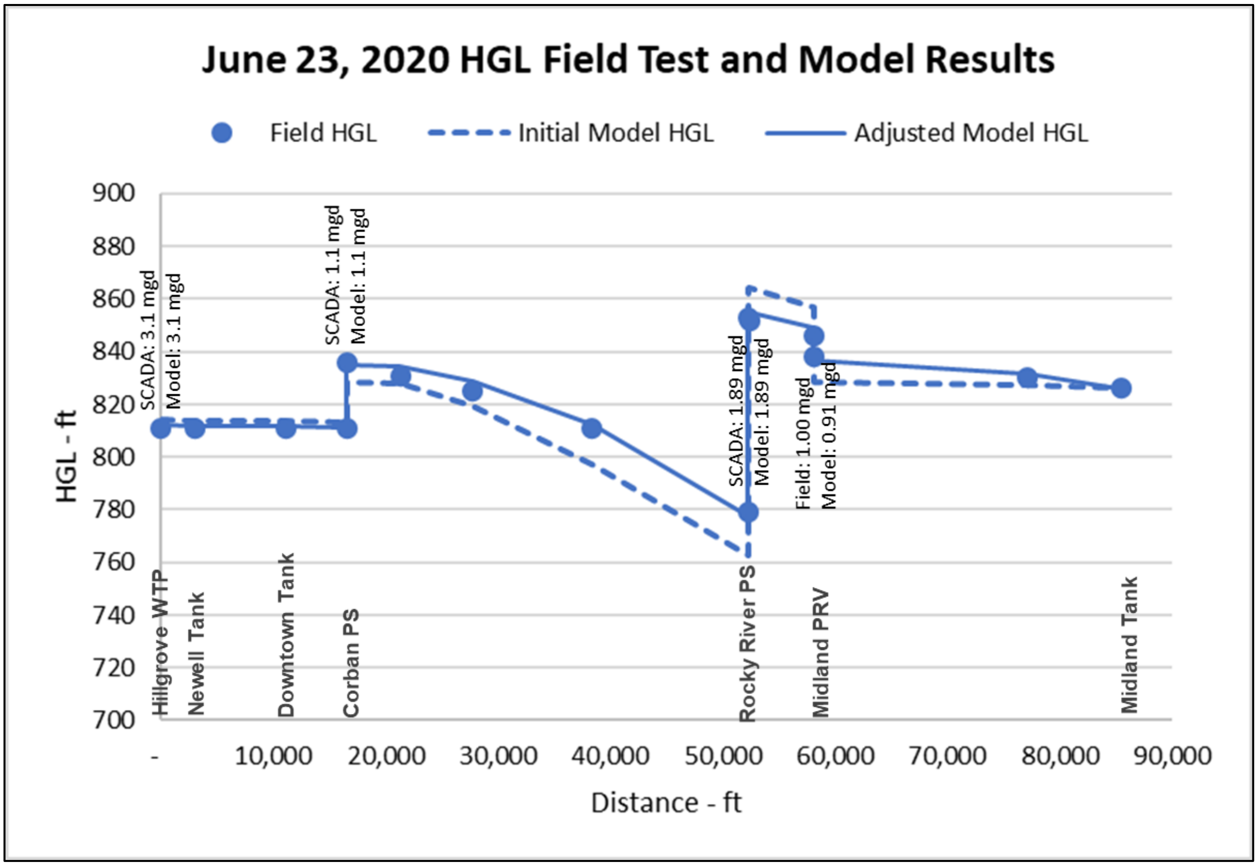


Figure 4-1: HGL Model Predicting and Measured Results Comparison

B. Calibrate Using Fire Flow Tests

Micro-calibration, such as fire flow tests, verifies that localized conditions in a specific area are being accurately simulated in the model. Table 4-2 compares model pressures to field measurements from the fire flow tests before and after calibration adjustments. The model simulated the measured flow for each test and adjustments were made to match both static and residual pressures within 4 psi.

For Fire Flow Test 2, the model predicted a residual pressure 4 psi higher than measured. The discrepancy was resolved by identifying a potential closed valve in the area, shown in Figure 4-1. The City checked and found the valve in the green circle was closed.

For Fire Flow Test 4, the static pressure discrepancy was more than 6 psi. The discrepancy was resolved by correcting the Boosted Pressure Zone boundary.

For Fire Flow Test 8, the model predicted residual pressure was high by 18 psi. A closed valve was suspected as shown in Figure 4-2. However, the city checked and found no closed valves. The City then did three additional fire flow tests around this area, shown in Figure 4-4, and one closed valve at Cochran Rd (green circle) was found. We recommend further investigations in this area.

For Fire Flow Test 9, multiple adjustments to model inputs were made, but residual pressures predicted by the model were still too high. Suspected closed valves, as shown in Figure 4-5, were shown to city staff. The City checked and found the valves in the green circles were closed.

Table 4-2: Fire Flow Test Results after Adjustment

Test No	Location	Test Flow GPM	Measured Pressure		Model Pressure- before Adjustments		Model Pressure – after Adjustments	
			Static (psi)	Residual (psi)	Static (psi)	Residual (psi)	Static (psi)	Residual (psi)
1	1317 Middlecrest Dr	1,810	75	52	72	52	74	55
2	Dartmoor Ave & Ravenscroft Ln	2,000	70	54	71	58	70	54
3	6135 Ferncliff Dr	1,620	75	62	74	59	75	60
4	Groff St & Montford Ave	1,000	94	45	58	43	95	44
5	911 Davidson Dr	-	-	-	-	-	-	-
6	Lincoln St & H Goodman Cir	2,700	90	74	92	83	91	72
7	Amhurst St & Rockingham Ln	1,300	74	29	72	17	73	29
8	Juanita Dr & Kim St	1,020	105	68	109	86	107	68
9	Millstone Cir & Millstone Cir	1,670	111	66	105	71	111	68
10	Aragorn Ln & Anduin Falls Dr	1,060	74	52	75	44	73	53
11	Brickwood Cir & Hill Pine Rd	1,040	118	91	132	109	120	92
12	Old Camden Rd & Community Dr	1,110	81	62	84	67	81	61

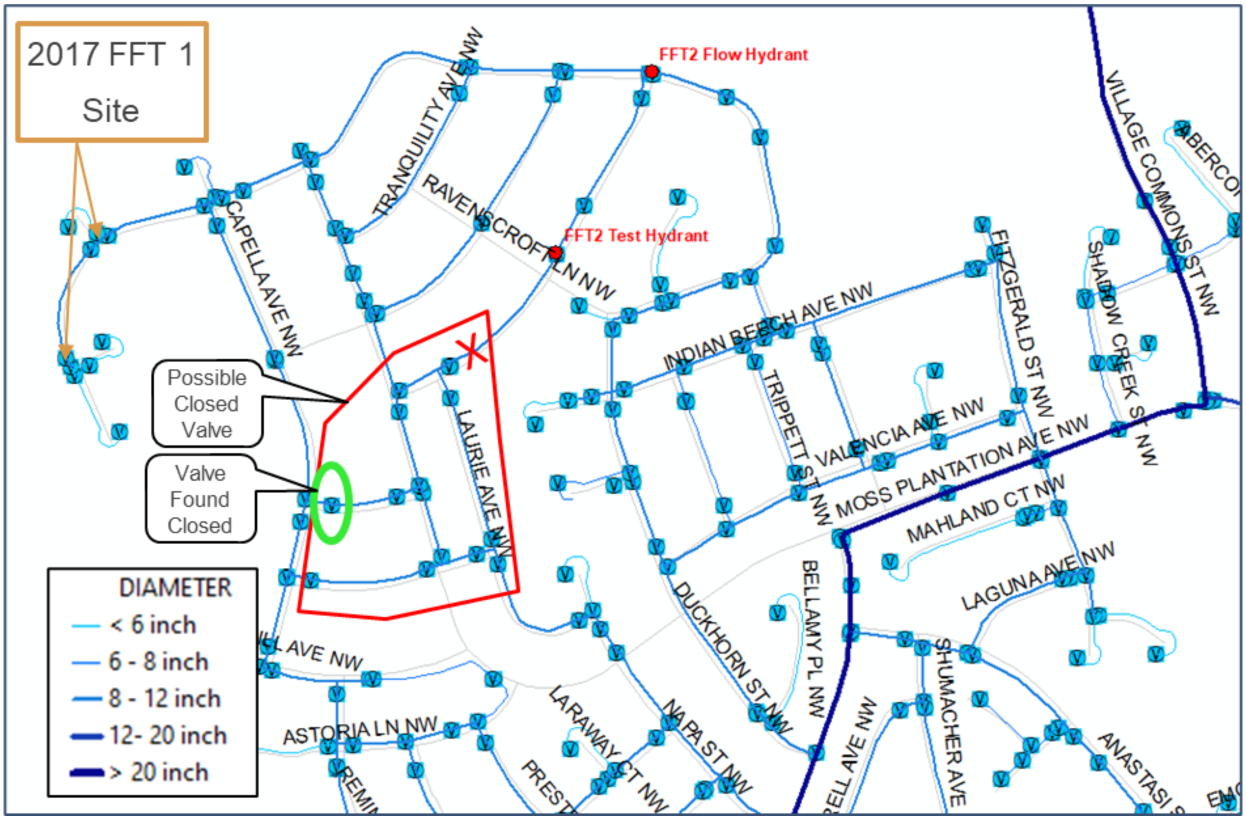


Figure 4-2: Suspected Closed Valve near Fire Flow Test 2

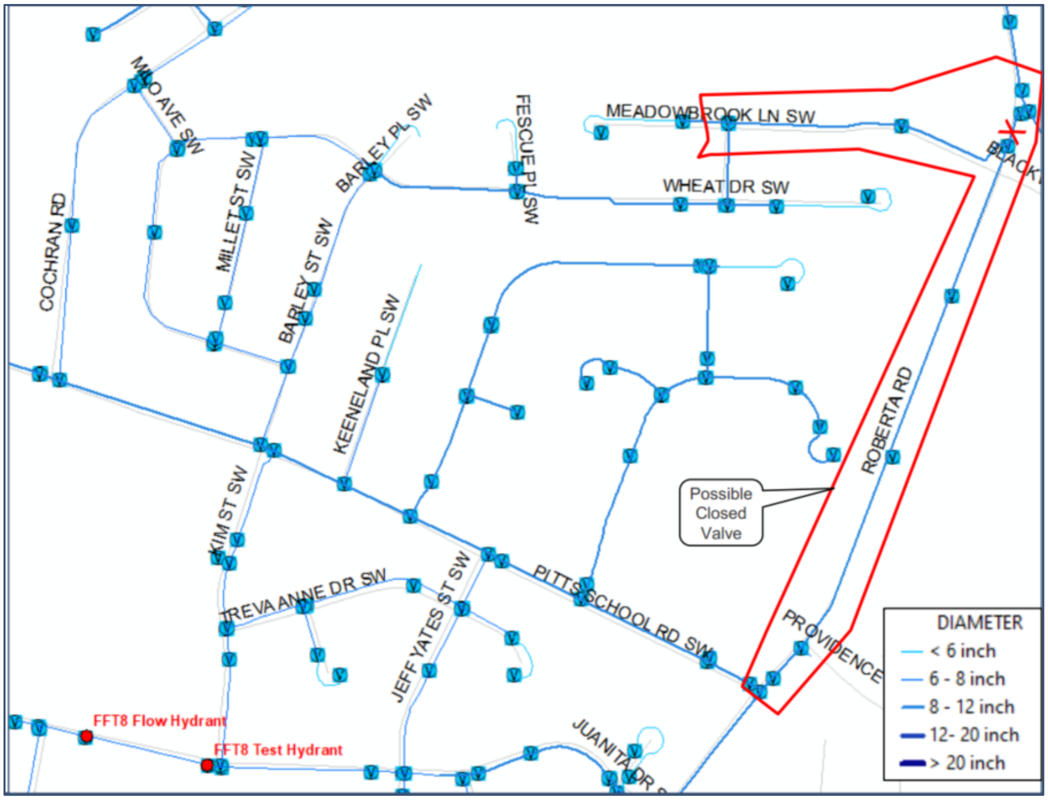


Figure 4-3: Suspected Closed Valve Area near Fire Flow Test 8

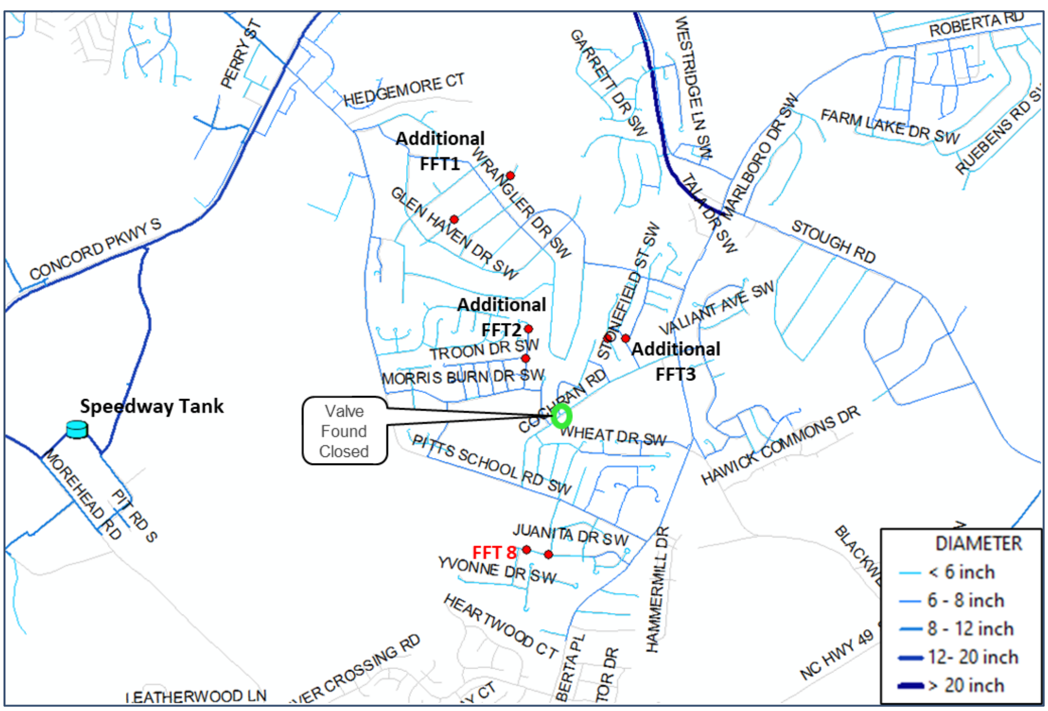


Figure 4-4: Three Additional Fire Flow Tests and Valve Found Closed for Fire Flow Test 8

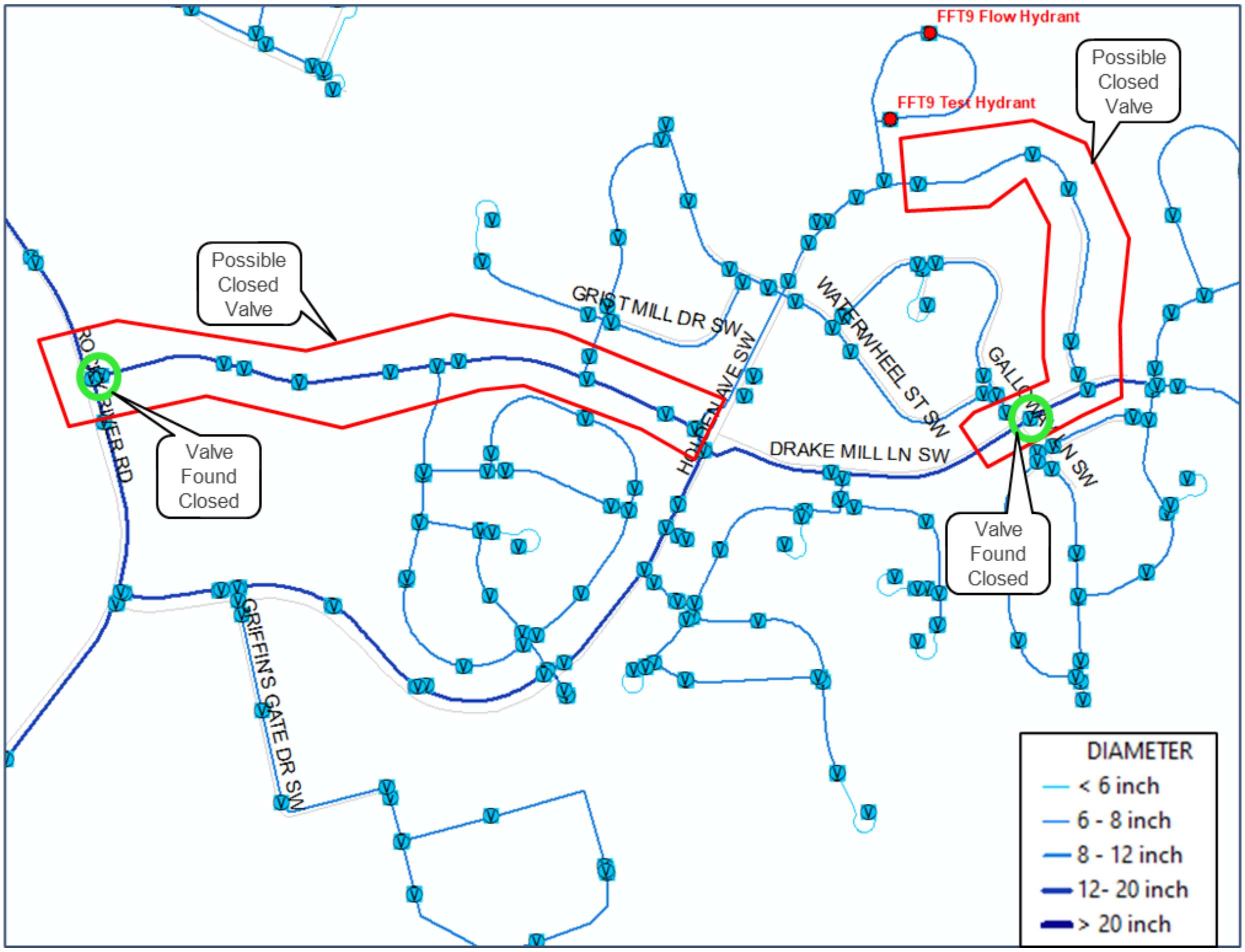


Figure 4-5: Suspected Closed Valve Areas and Valves Found Closed near Fire Flow Test 9

C. Calibration Using SCADA Records

The final step in calibrating the model was running a 24-hour extended-period simulation (EPS) and comparing predicted tank levels to SCADA. This is a critical step for modeling distribution system operations water quality because the operation of pumps and tanks significantly impacts water age.

Predicted tank levels were compared to February 12, 2020 SCADA records.

I. 831 Pressure Zone

The 831 Pressure Zone includes Downtown Tank and Todd Newell Tank. Figure 4-6 compares the model results to SCADA for these tanks. The water level in Downtown Tank fluctuated between 26 feet and 32 feet during the calibration period (hydraulic grade between 817 feet and 823 feet). The water level in Todd Newell Tank fluctuated between 21 feet and 28 feet during the calibration period (hydraulic grade between 822 feet and 829 feet). The tank levels drop due to diurnal demand patterns and operation of the Corban Pump Station. The tank is filled by the pumps at the Hillgrove WTP. The model closely matched the recorded water levels in the tank.

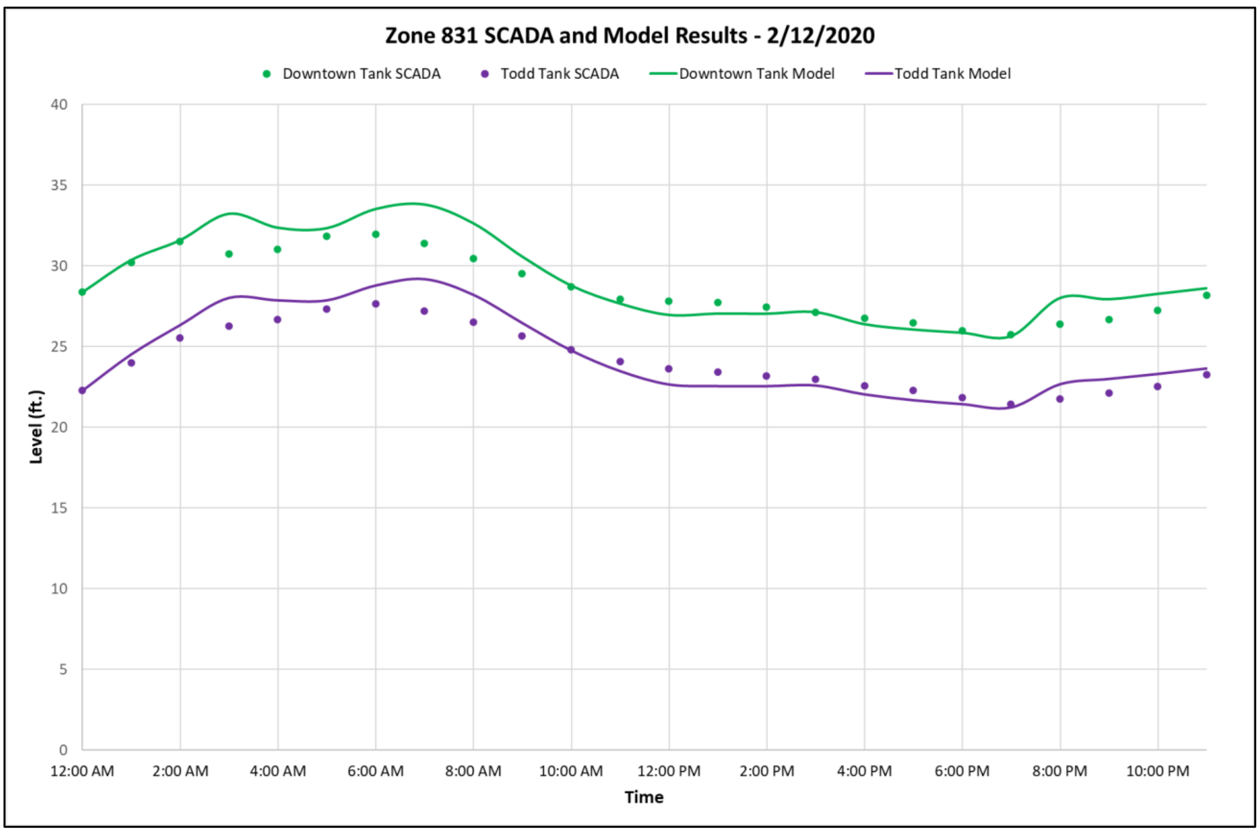


Figure 4-6: Downtown Tank and Todd Newell Tank Levels Comparison

II. 841 Pressure Zone

SCADA records for the water level in the Midland Tank fluctuated between 22 feet and 32 feet (hydraulic grade between 803 feet and 833 feet). This tank level is influenced by demand patterns in the 841 Pressure Zone and operation of Hwy 601 and Rocky River pump stations. The pump stations alternate to fill the tank when levels drop. The model closely matched recorded tank levels as shown in Figure 4-7.

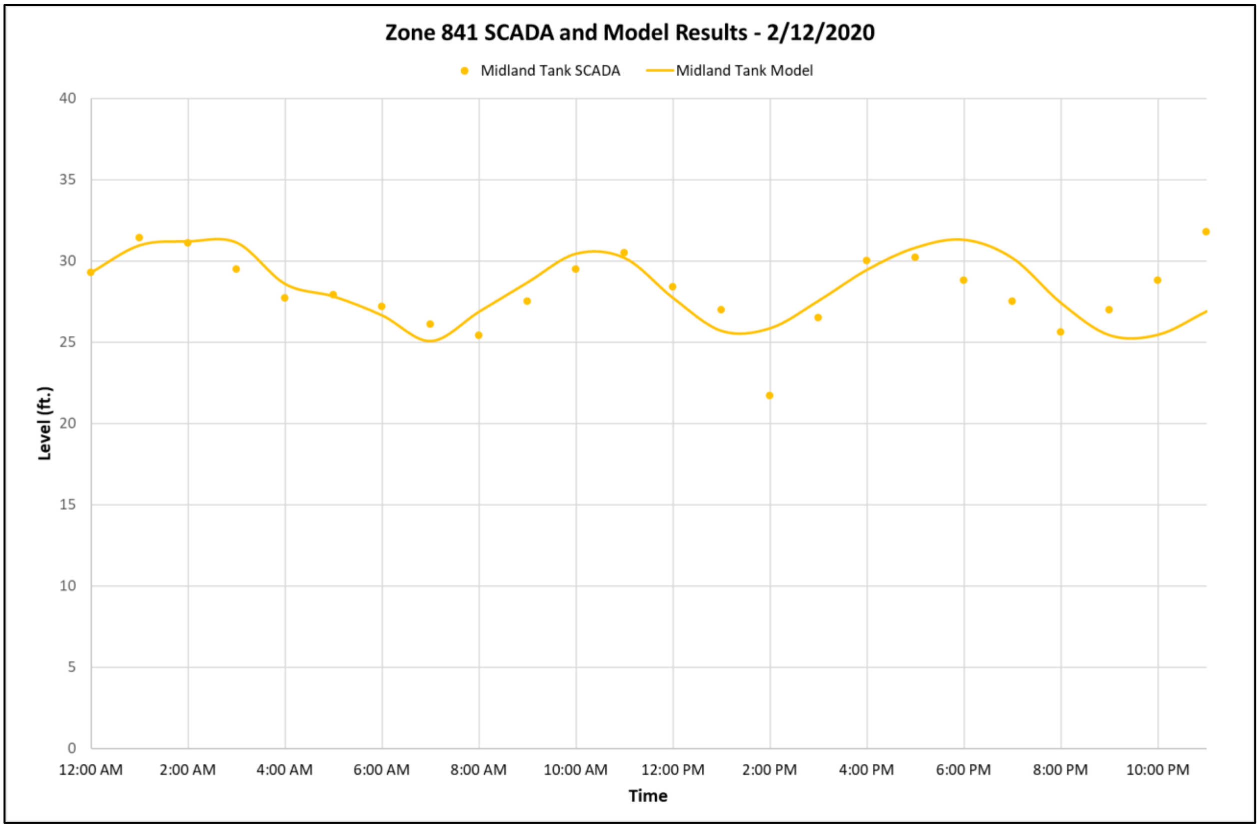


Figure 4-7: Midland Tank Level Comparison

III. 850 Pressure Zone

The 850 Pressure Zone includes Speedway Tank and Rock Hill Church Tank. Tank levels in the 850 Pressure Zone are impacted by demand, Rocky River Pump Station, Corban Pump Station and Coddle Creek WTP.

SCADA records from Speedway Tank showed that levels fluctuated between 19 feet and 34 feet during the calibration period, as shown in Figure 4-8. The hydraulic grade of the Speedway Tank varied from 829 feet to 844 feet. The model matched the general pattern recorded by SCADA for both tanks.

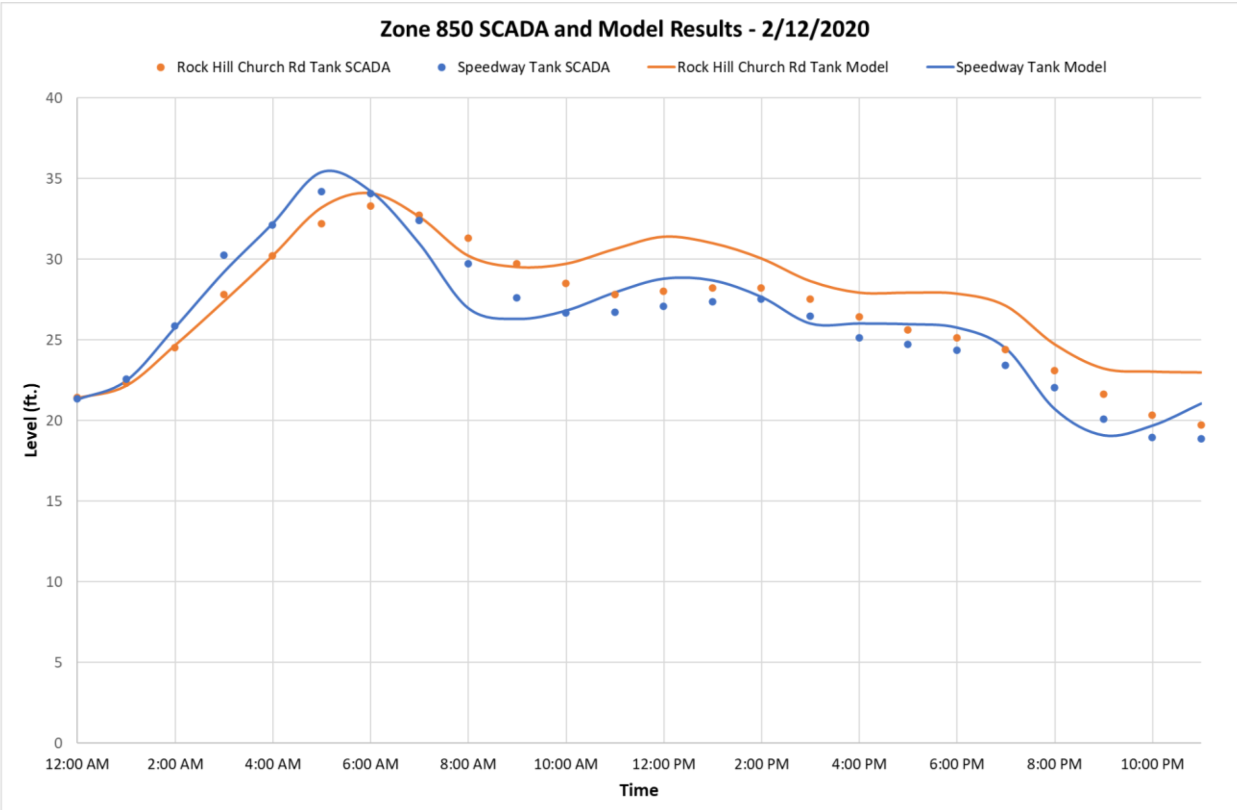


Figure 4-8: Speedway Tank and Rock Hill Church Tank Level Comparison

IV. 890 Pressure Zone

SCADA records showed the Hwy 73 Tank in the 890 Pressure Zone fluctuated between 17 feet and 28 feet during the calibration period (hydraulic grade between 867 feet and 878 feet). This tank is influenced by demand patterns in the 890 Pressure Zone and operation of Mt Pleasant and Hwy 601 pump stations. The model matched the general pattern recorded by SCADA, as shown in Figure 4-9.

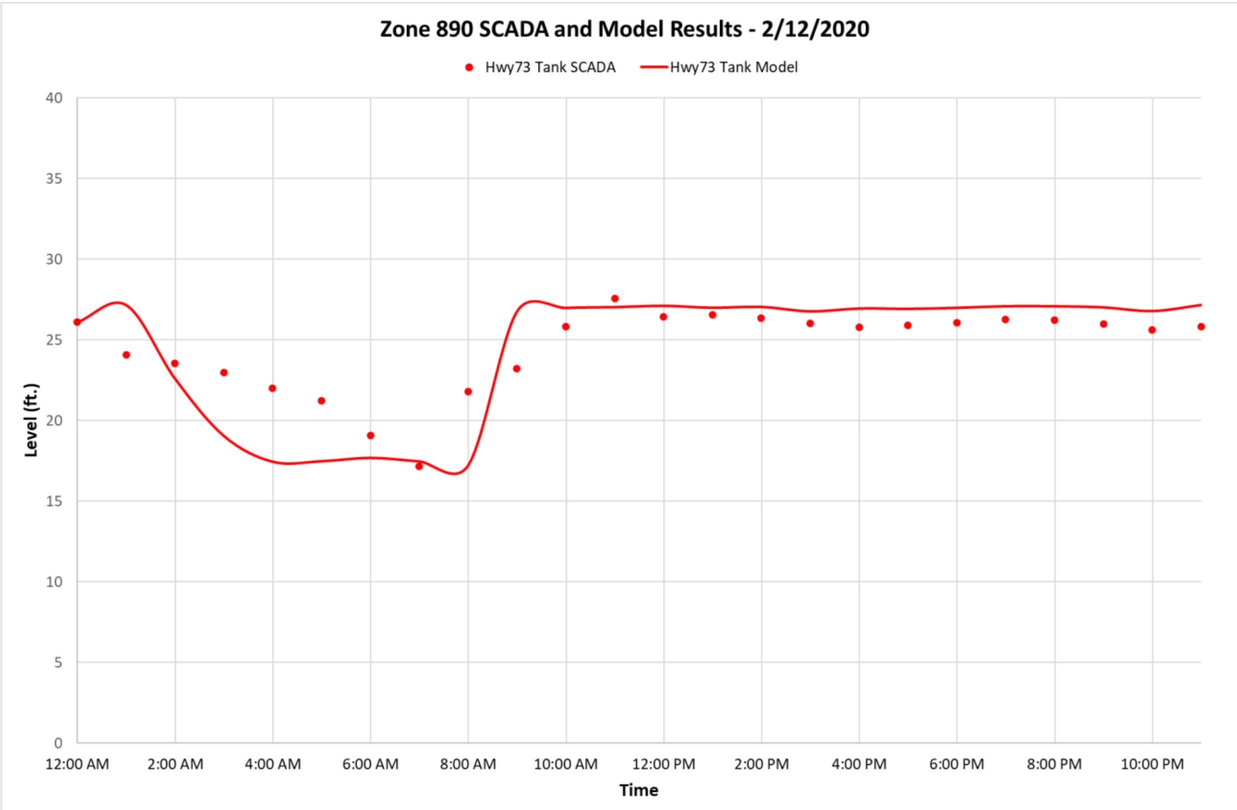


Figure 4-9: Hwy 73 Tank Level Comparison

Section 5 Identify Existing Deficiencies

Existing deficiencies were identified by simulating 2019 conditions and comparing them to design criteria, the design goals included:

- Maintain pressures above 40 psi under peak hour conditions. North Carolina Department of Environmental Quality Public Water Supply (PWS) code allows for 30 psi during peak hour conditions (**15A NCAC 18C .0405 B**).
- Maintain pressures above 20 psi during fire flows.
- Maintain water age less than 7 days.
- Provide adequate storage to meet existing system water demands.
- Provide adequate pumping for existing system water demands.

A. Available Fire Flow

In the 2012 City of Concord Hazard Mitigation Fire Flow Study, needed fire flows were determined throughout the city based on building size and material. At each hydrant location, the predicted available fire flow from the calibrated model was compared to the needed fire flow. The model predictions are representative of fire flows for the worst-case condition: maximum demand day with tanks at the lower limit of their operating ranges. Figure 5-1 shows fire flow deficiencies color coded to indicate the difference between needed flows and design fire flows at 20 psi. Table 5-1 summarizes how many hydrants were deficient by various amounts.

Fire flow problem areas were considered in the sizing and routing of proposed improvements, as described in a subsequent section of this report.

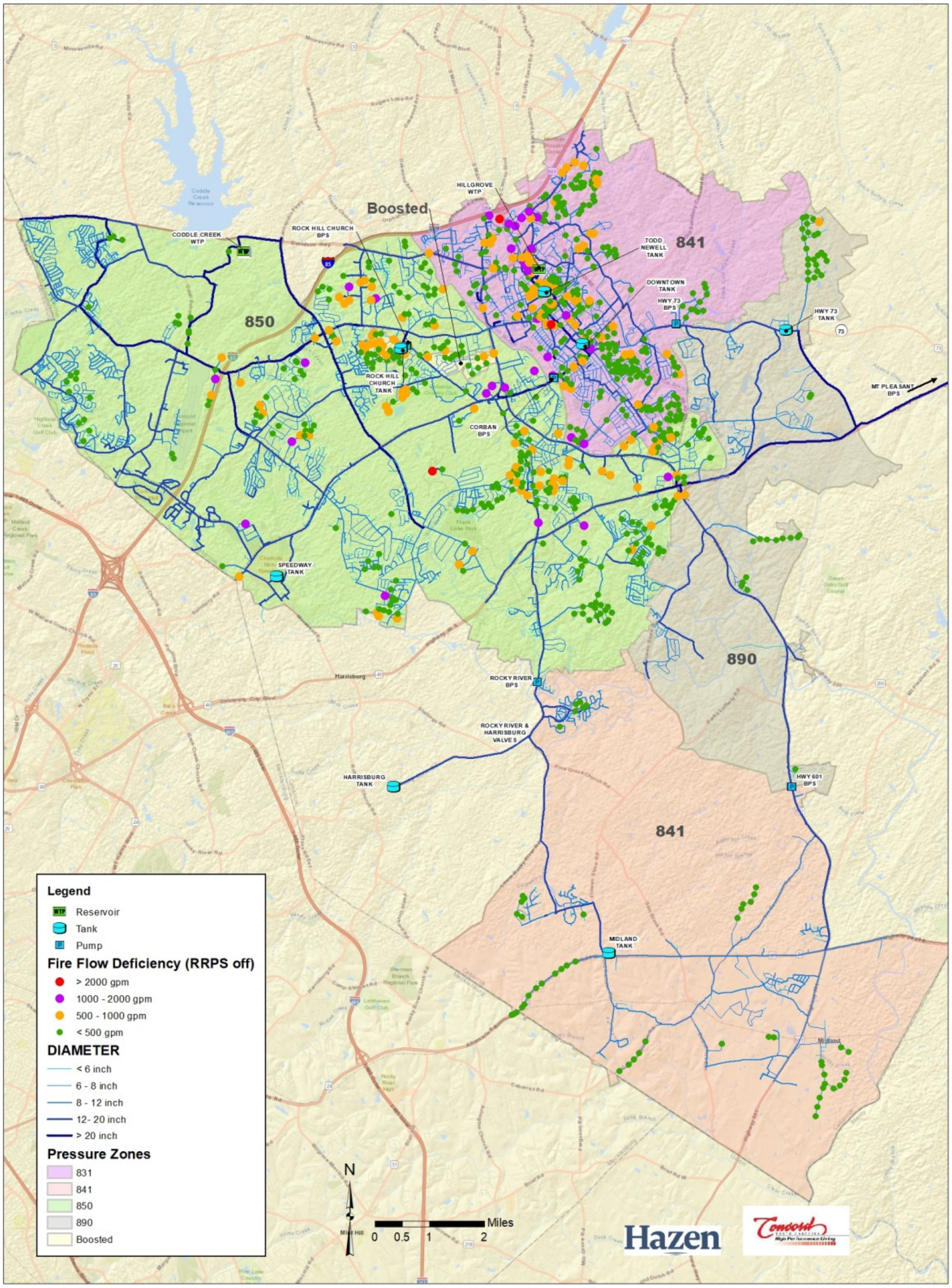


Figure 5-1: 2019 Fire Flow Deficiency

Table 5-1: 2019 Fire Flow Deficiency Summary

Fire Flow Deficiencies	# of Hydrants
< 500 gpm	618
500 – 1,000 gpm	150
1,000 – 2,000 gpm	32
> 2,000 gpm	3

B. Water Age

Water age was predicted based on a 35-day simulation of existing average daily demand using current pump controls and operating procedures. Figure 5-2 shows water age color coded to indicate water age problem areas. Problem areas include the Hwy 73 Tank area in Pressure Zone 890, Rock Hill Church Tank area in Boosted Pressure Zone, and Midland Tank area in Pressure Zone 841.

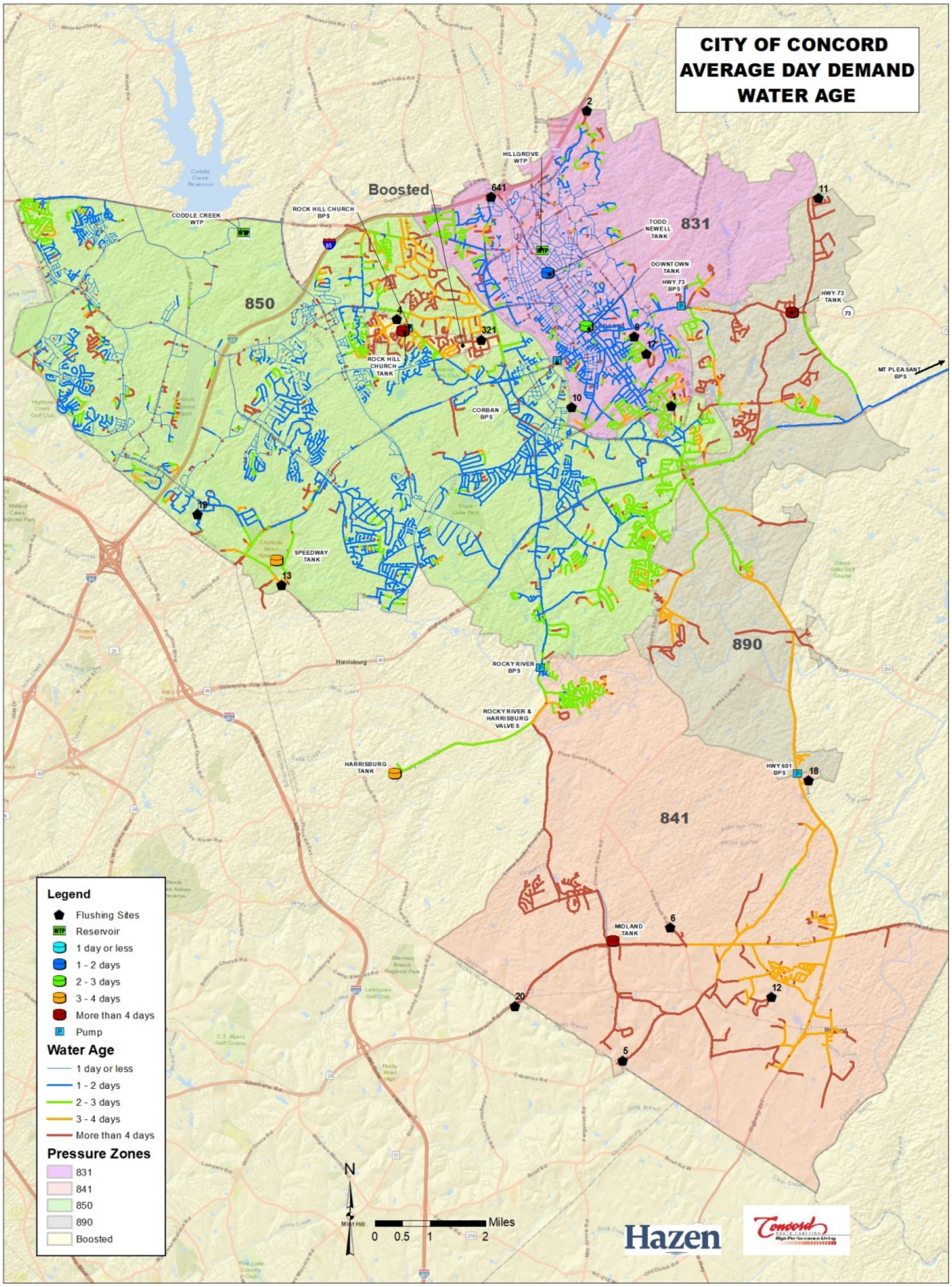


Figure 5-2: 2019 Average Day Demand Water Age

C. Low Pressure

A model simulation of 2017 Master Plan showed several areas where customers experience low pressures due to high ground elevations. The existing system with peak hour demand had five areas of low pressure as shown in detail in Figure 5-3. Low pressures in these areas are addressed by changing pressure zone boundaries as described in the Pressure Zone Evaluation.

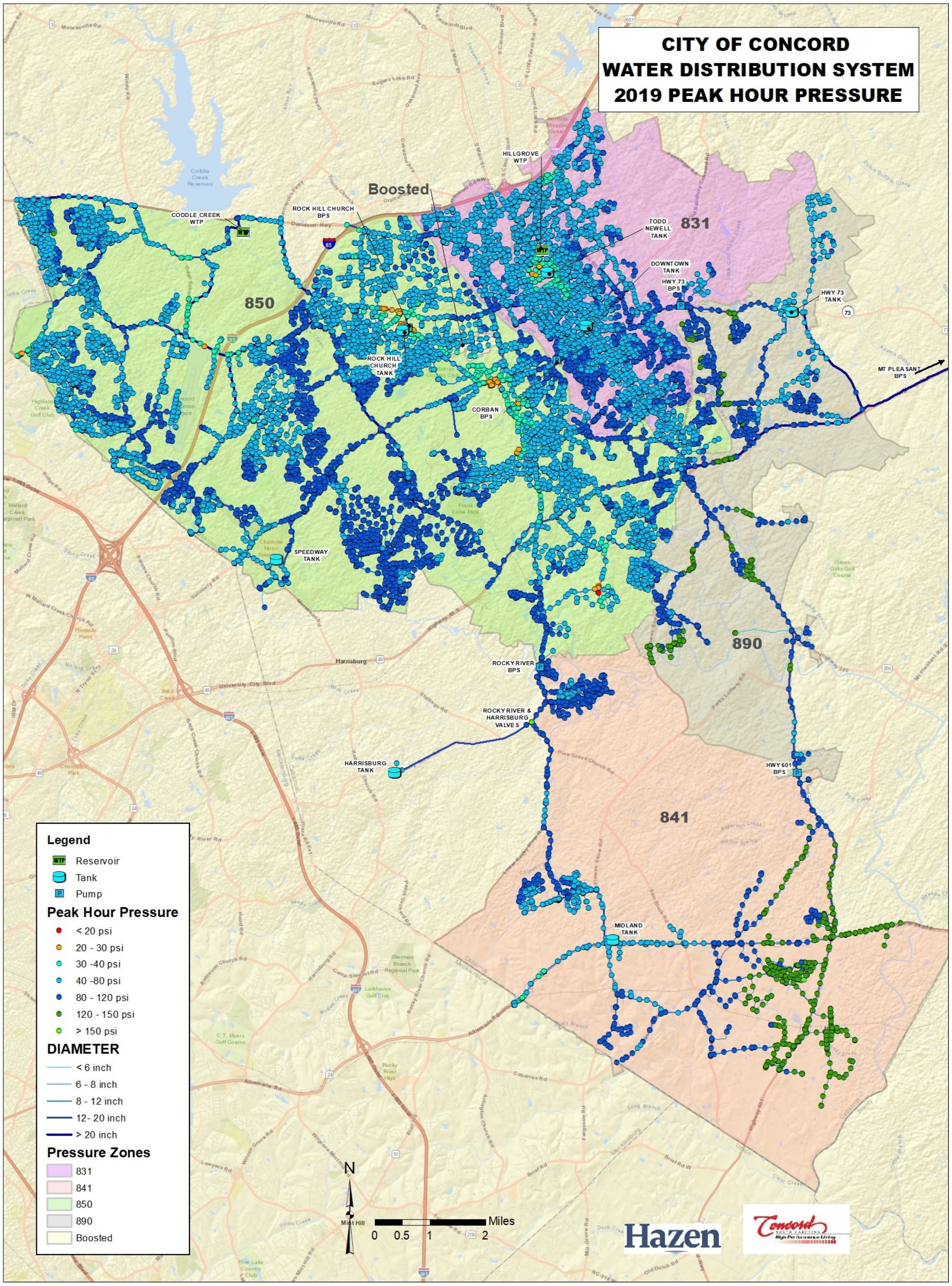


Figure 5-3: 2019 Peak Hour Pressure

D. Existing Pump and Storage Capacity

The existing system's pump and storage capacity are shown in the following tables. Table 5-2 compares pump station existing firm capacity. Including wholesale demand; 2. Firm BPS supply to other pressure zones

Table 5-3 compares existing storage capacity to requirements for equalizing and fire storage in each pressure zone. Equalization storage optimally allows water to be pumped into the system at a constant rate equal to the 24-hour average demand rate. This approach minimizes the required capacity of transmission mains and finished water pumps. The amount of storage required depends on the amount of variable in hourly demand. Diurnal demand patterns were used to calculate the amount of storage required to equalize diurnal demand. We subtracted equalization storage associated with wholesale customers, assuming wholesale customers provide their own equalizing storage.

The North Carolina Department of Environmental Quality Public Water System Department (PWS) requirement of total distribution system storage of half the average day demand, approximately 5.21 mgd, was also checked. Existing storage capacity including the WTP clear wells (4 MG each) and Mt Pleasant ground storage (1 MG). All pressure zones were not deficient. The total available storage in the water distribution system is 13.75 mgd (including clear wells and Mt. Pleasant ground storage). This meets the emergency storage requirement of 5.21 mgd.

Table 5-2: Existing Pump Capacity Evaluation

2019 System	831	841	850+Boosted	890
Pump Capacity				
Maximum Day – MGD[1]	3.66	2.26	11.88	0.56
Supply to Other Zones[2]	4.60	0.00	2.80	1.00
Total Pumping Requirement	8.26	2.26	14.68	1.56
Existing Firm Pump Capacity – MGD				
Hillgrove WTP	15.20			
Coddle Creek WTP			13.2	
Hwy 601 PS		0.70		
Rocky River PS		2.80		
Highway 73 PS				1.25
Rock Hill Church Road PS			0.63	
Mt Pleasant PS				7.00
Corban PS			4.60	
Total Pump Capacity – MGD	15.20	3.50	18.43	8.25
Pump Capacity Surplus/ Deficit	6.94	1.24	3.75	6.69

1. Including wholesale demand; 2. Firm BPS supply to other pressure zones

Table 5-3: Existing Storage Capacity Analysis

2019 Service Area	831	841	850+Boosted	890
Equalizing Storage				
Average Day Demand – MGD[1]	2.67	1.05	6.30	0.40
Maximum Day Demand – MGD[2]	3.68	1.42	8.81	0.55
Equalizing Percentage	13%	16%	14%	20%
Equalizing Volume – MG	0.48	0.23	1.23	0.11
Fire Storage				
Needed Fire Flow – GPM	3000	2500	3000	3000
Duration – Hours	3	2	3	3
Fire Storage Volume – MG	0.54	0.30	0.54	0.54
Equalizing + Fire Storage Volume – MG	1.02	0.53	1.77	0.65
EMERGENCY STORAGE (1/2 ADD)	1.34	0.53	3.15	0.20
Existing Storage - MG				
Clearwell - Hillgrove WTP	4.00			
Clearwell - Coddle Creek WTP			4.00	
Midland		0.50		
Speedway			1.00	
Rockhill Church Hill			2.00	
Downtown	0.50			
Todd Newell	1.00			
Highway 73				0.75
Hwy 29				
Total System Storage - MG	1.50	0.50	3.00	0.75
Equalizing & Fire Storage - MG				
Existing	1.50	0.50	3.00	0.75
Storage Capacity Surplus/Deficit	0.48	-0.03	1.23	0.10

1. Without wholesale demand
2. 1.4 factor applied except on flushers

Section 6 Forecast Future Water Demand

A. Population and Employment Growth

The City of Concord provides water to the majority of the City of Concord and the Town of Midland as well as much of the unincorporated area between the two municipalities. Figure 6-1 displays municipal boundaries and the current pressure zones.

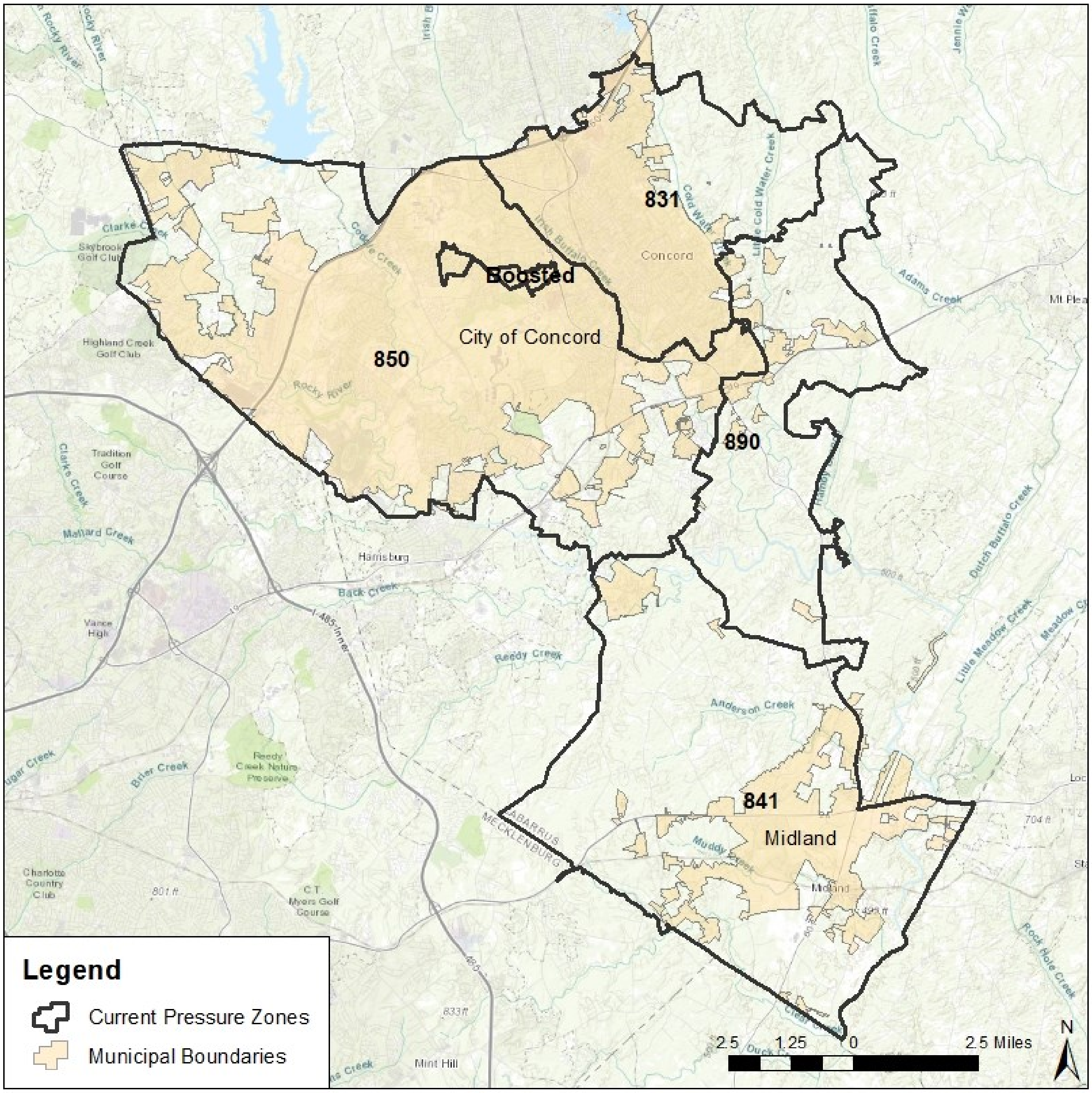


Figure 6-1: Service Area Map with Municipal and Pressure Zone Boundaries

Population is highly correlated with water use and was therefore used as a basis for estimating the future water demand requirements for the City's distribution system. Both projections for residential population as well as the number of employees within the service area were used. To determine the total residential population served by the City's distribution system, the number of meters was used in combination with the persons per household determined from data by the U.S. Census Bureau's American Community Survey (ACS) for the City of Concord. The calculation assumed that one residential meter represented one household in the service area.

Table 6-1 displays the number of residential meters, the persons per household calculated from ACS number of households and population data for the City of Concord, and the calculated service area population.

Table 6-1: Historical Meter-Based Population

Year	Residential Meter Count	Persons per Household (ACS)	Service Area Population
2011	33,635	2.48	83,575
2012	34,172	2.47	84,327
2013	34,830	2.64	91,903
2014	35,430	2.60	91,960
2015	36,133	2.57	92,794
2016	36,911	2.63	97,102
2017	37,777	2.65	99,939
2018	38,917	2.57	100,191

The NC Office of State Budget and Management (NCOSBM) provides county level population projections for the next twenty years linearly based off historical growth at the county level. The City of Concord also developed population projections in 2018 with a constant yearly growth rate of 1.33% based off the City's historical growth, similar to the NCOSBM projection method. Based on both the County's and City's projection methods, Midland's population was also projected based off historical growth. Similarly, the service area population was projected linearly based off the 2011 to 2018 average growth rate using the meter-based population estimates. An estimate of the unincorporated population was calculated based off the total service area population and the City of Concord and Midland populations. The historical growth-based population projections are displayed in Figure 6-2.

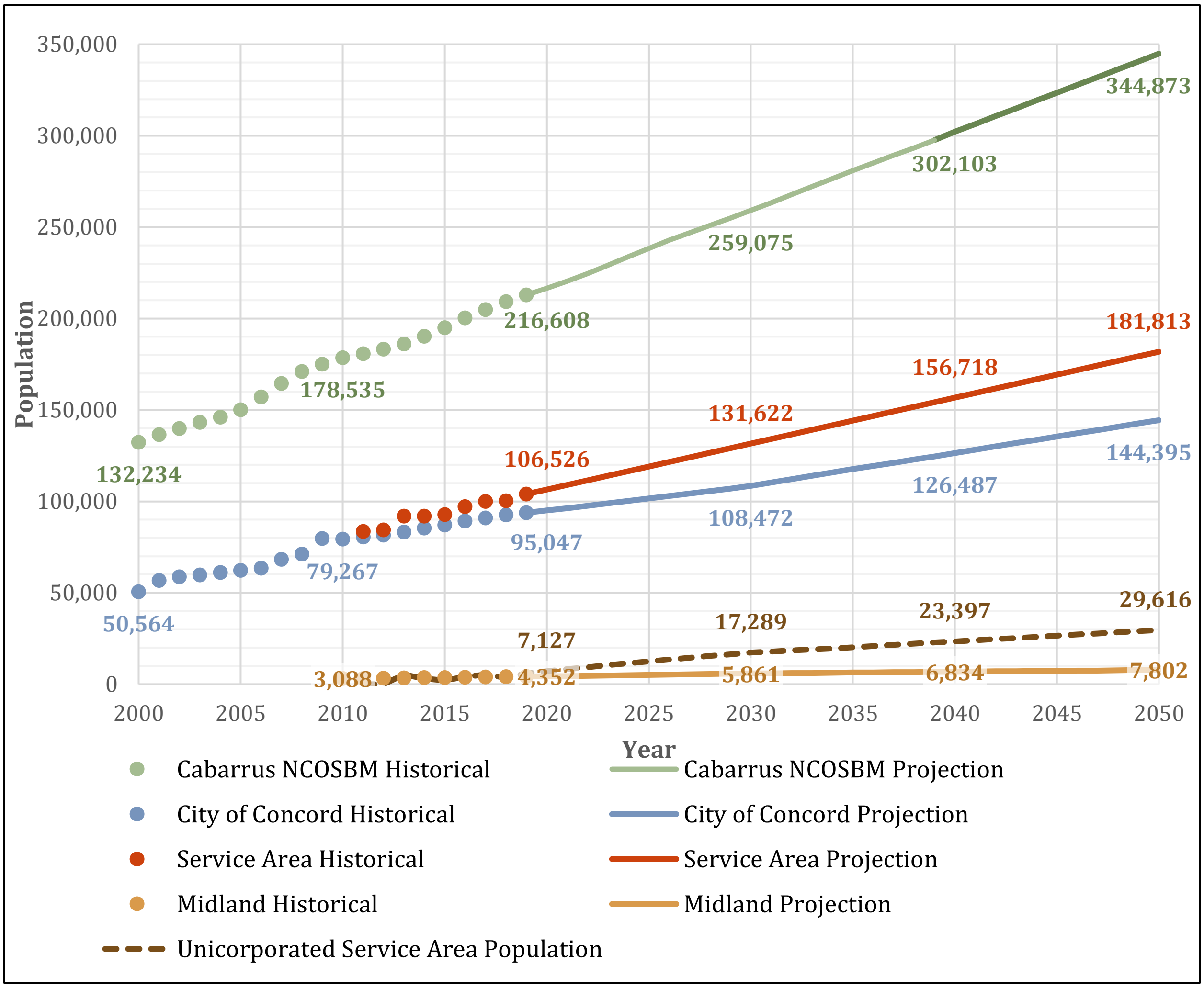


Figure 6-2: Historical Growth-Based Population Projections

Traffic Analysis Zone (TAZ) data was also acquired from the Cabarrus-Rowan Metropolitan Planning Organization (CRMPO) through the City including population and employment data from 2018 and projections for 2050. The TAZ spatial units included in this data set are significantly smaller than either municipality in the service area and therefore allow a higher degree of geospatial resolution in determining the distribution of population growth, which is relevant to better targeting distribution system improvements.

The growth projected by the TAZ data provided a different growth rate from the historical-growth based projections, however. As shown in Figure 6-3, the projected growth rate based off this data is higher than what the historical growth suggested at about 4.1% per year. The meter-based population data had an average yearly growth of 2.5%. It was determined it would be more conservative to use the TAZ based growth rate because it projected a faster increase in the next thirty years. Additionally, this data was selected because it had area-specific input by the CRMPO.

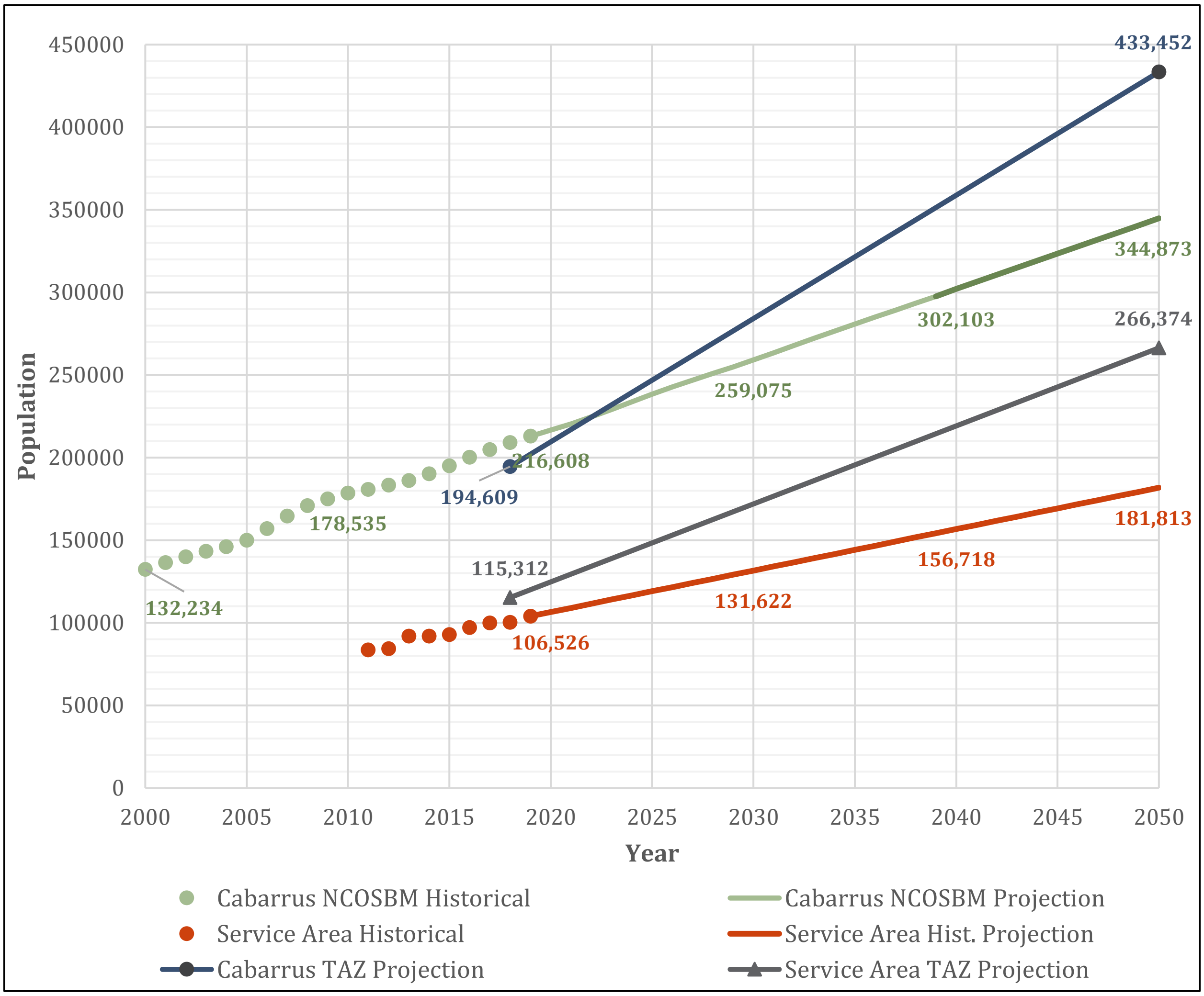


Figure 6-3: Comparison of TAZ-based and Historical Rate-based Population Growth

Using the TAZ data allowed for area specific demand forecast calculations to be determined. Figure 6-4 illustrates population density within the service area by TAZ zones and indicates the current pressure zone boundaries. Population density is greatest in the northern part of the system in and around the City of Concord with Midland and the unincorporated areas being more sparsely populated.

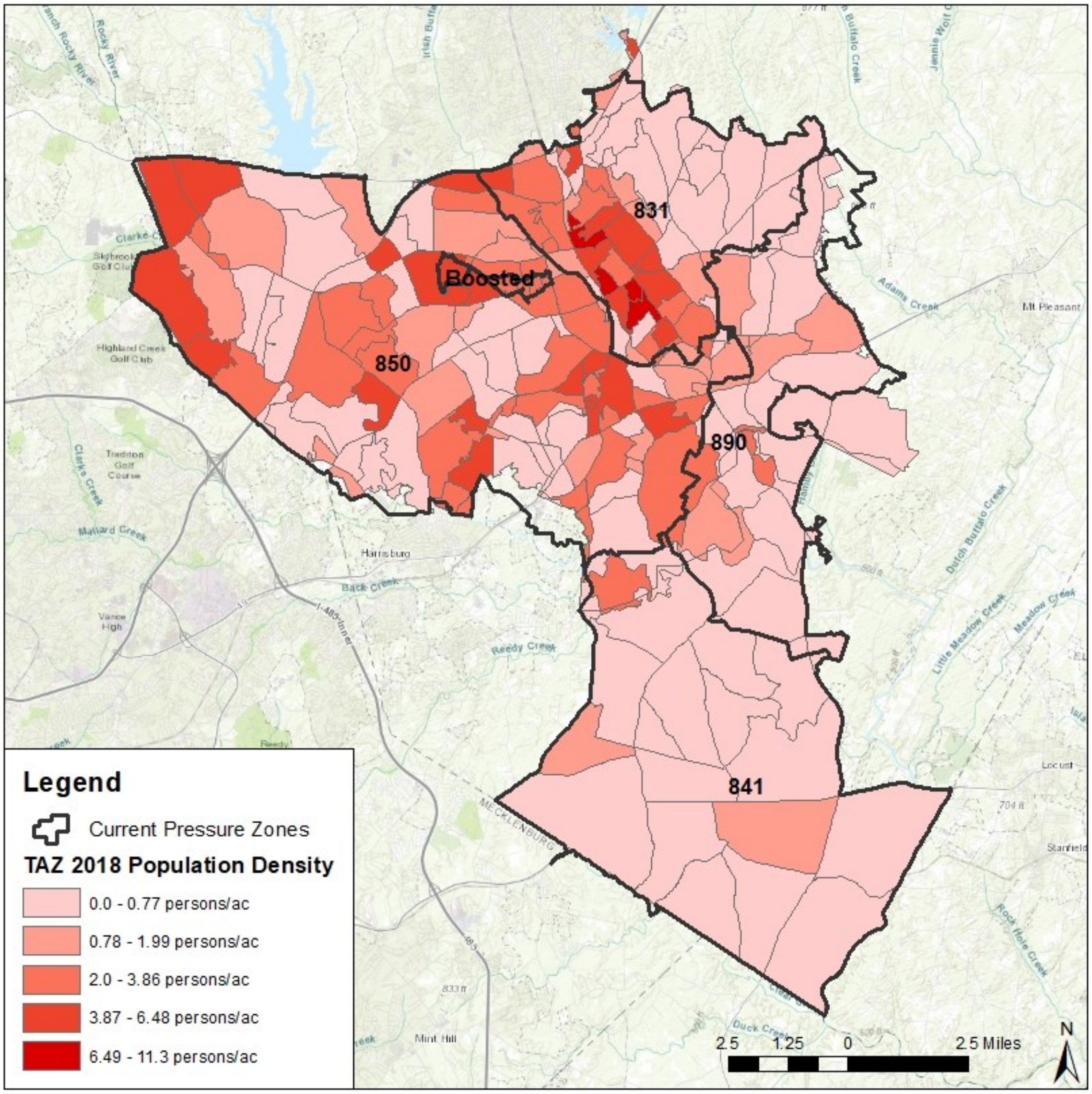


Figure 6-4: TAZ Population Density 2018

Figure 6-5 displays the anticipated population density for 2050 as well as the re-alignment of pressure zone boundaries. The TAZ data shows increased density in and around the City of Concord and a lesser increase in density in Midland with the unincorporated areas experiencing slower growth rates.

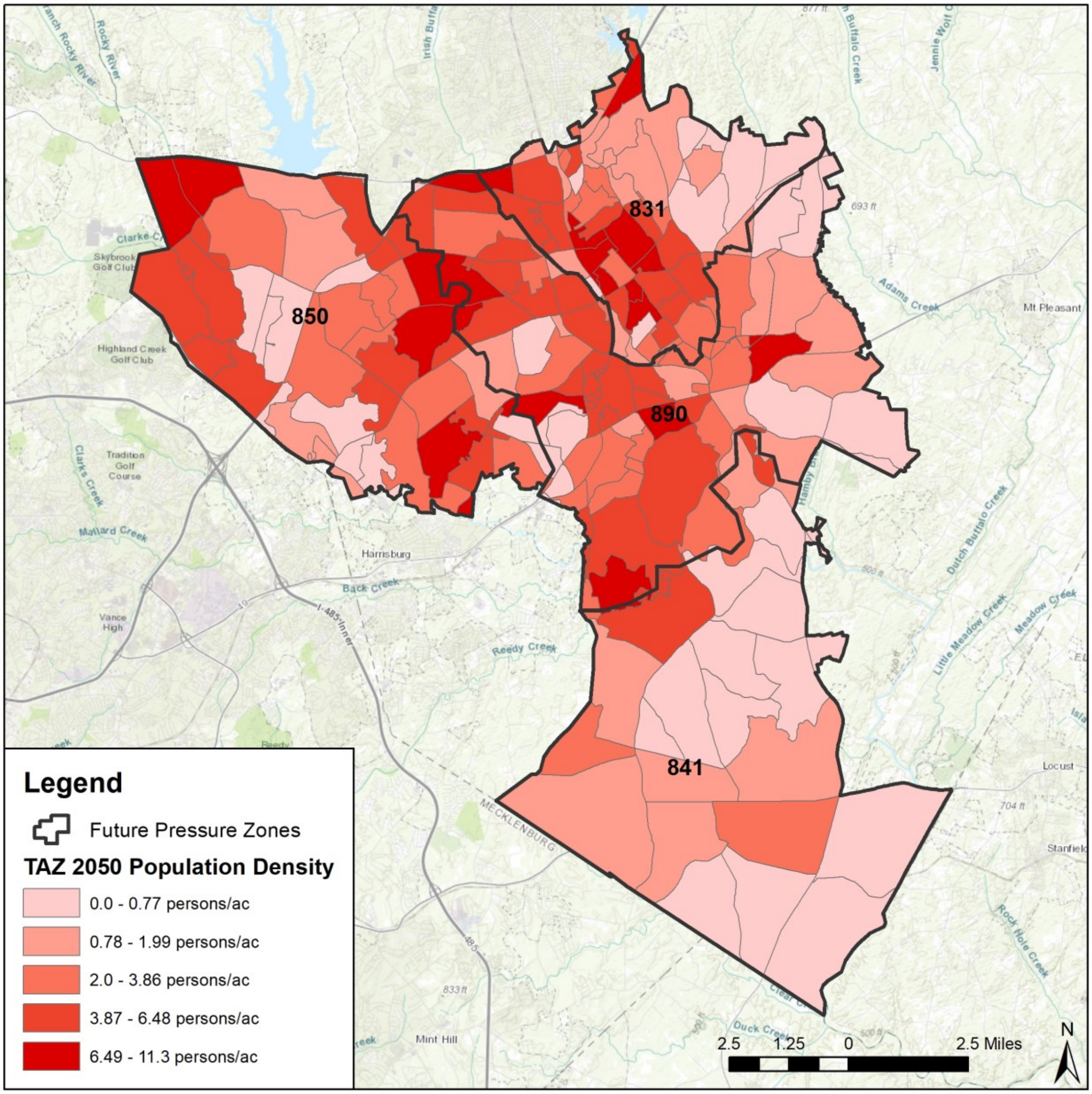


Figure 6-5: TAZ Population Density 2050

The service area boundaries, both present and future, include regions not reached by the distribution system at present. The circles in Figure 6-6 highlight several of these areas where the City may extend service in the future.

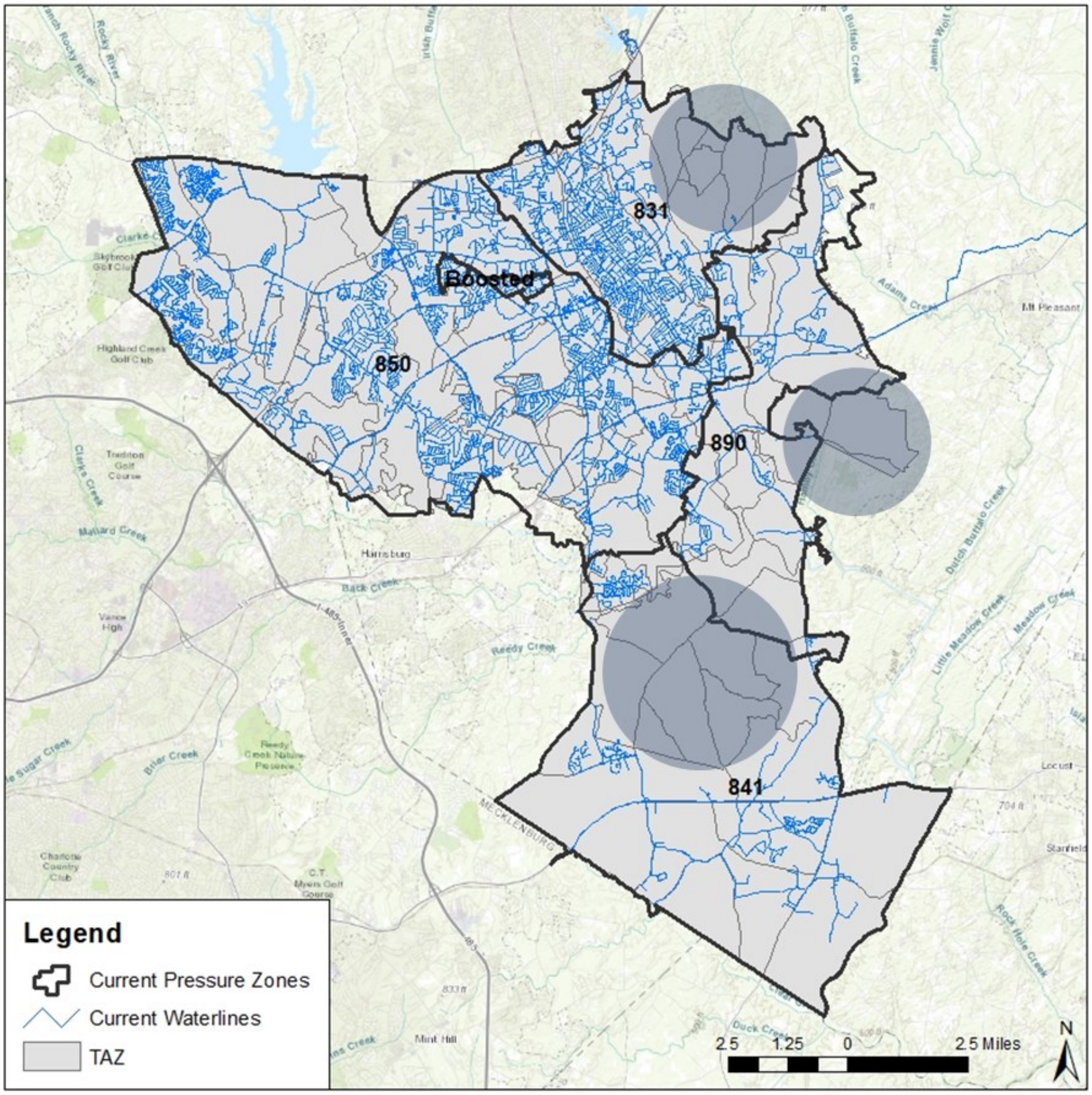


Figure 6-6: Current Pressure Zone Boundaries and Pipelines

Table 6-2 shows estimates of both the served population in 2018 from water meter data as well as an estimate of the total population within the 2018 service area boundary. The difference between the two is the best estimate of the population within the service area boundaries not currently being provided with City water.

Table 6-2: 2018 Population Adjustment

Source	Population
Meter Data	100,191
TAZ Data	121,959
Population Difference	**21,768**

Figure 6-7 shows the population projection for the total service area as well as by pressure zone. There are several pressure boundary adjustments that will be made in the next decade which would, if implemented, adjust the population served in each pressure zone. While Figure 6-7 makes use of the realignments as suggested in the 2017 Master Plan and adjusted during the process of updating this Master Plan, the timing and alignment of those adjustments is subject to change but is presented here as occurring over the next two years.

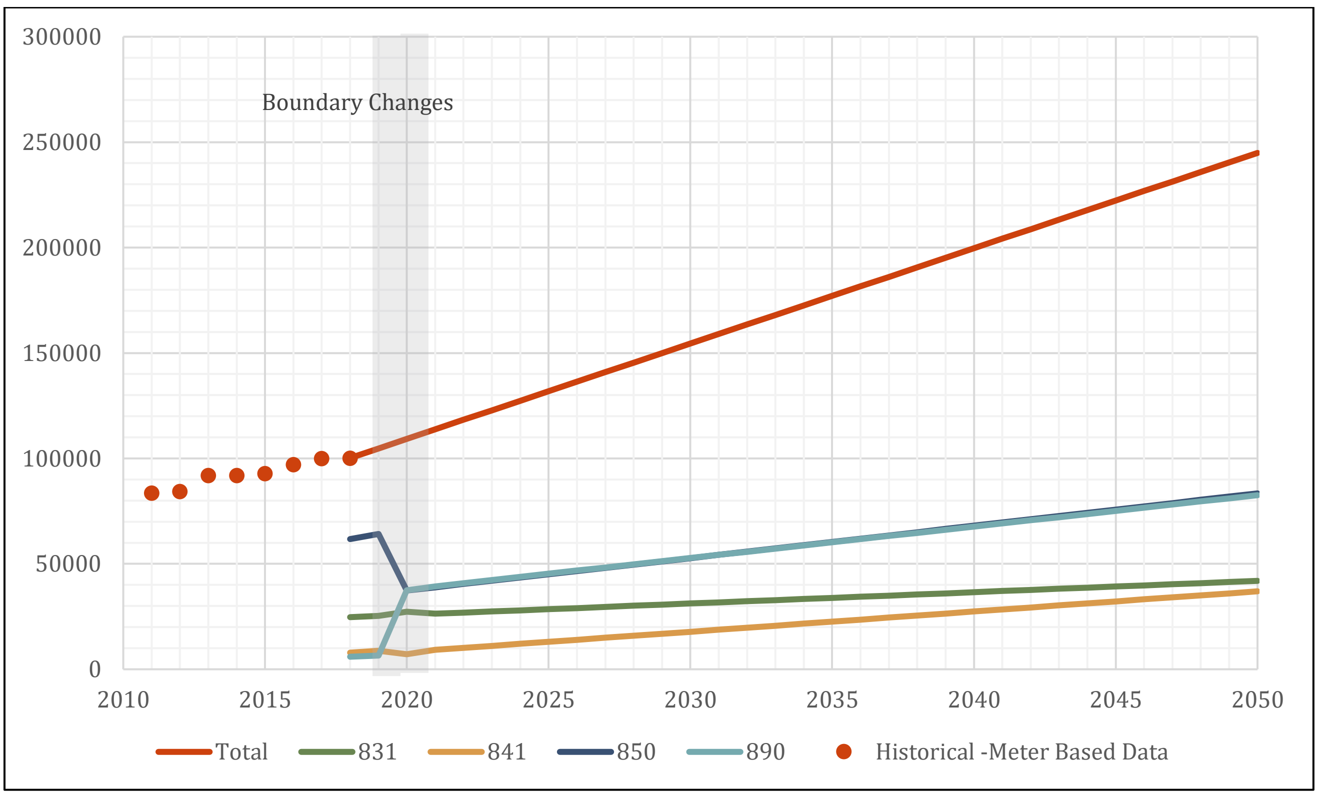


Figure 6-7: Service Area Population Projection

Employment growth was also used to develop the demand projections. The only source of employment projections is the CRMPO TAZ data. Figure 6-8 and Figure 6-9 show the employment density in the pressure zones in 2018 and 2050. Most of the employment is concentrated in the City of Concord with the 850 and upper 890 zone (post-realignment) anticipating the greatest increase in employment. Little to no employment growth is forecast for the unincorporated areas and Town of Midland.

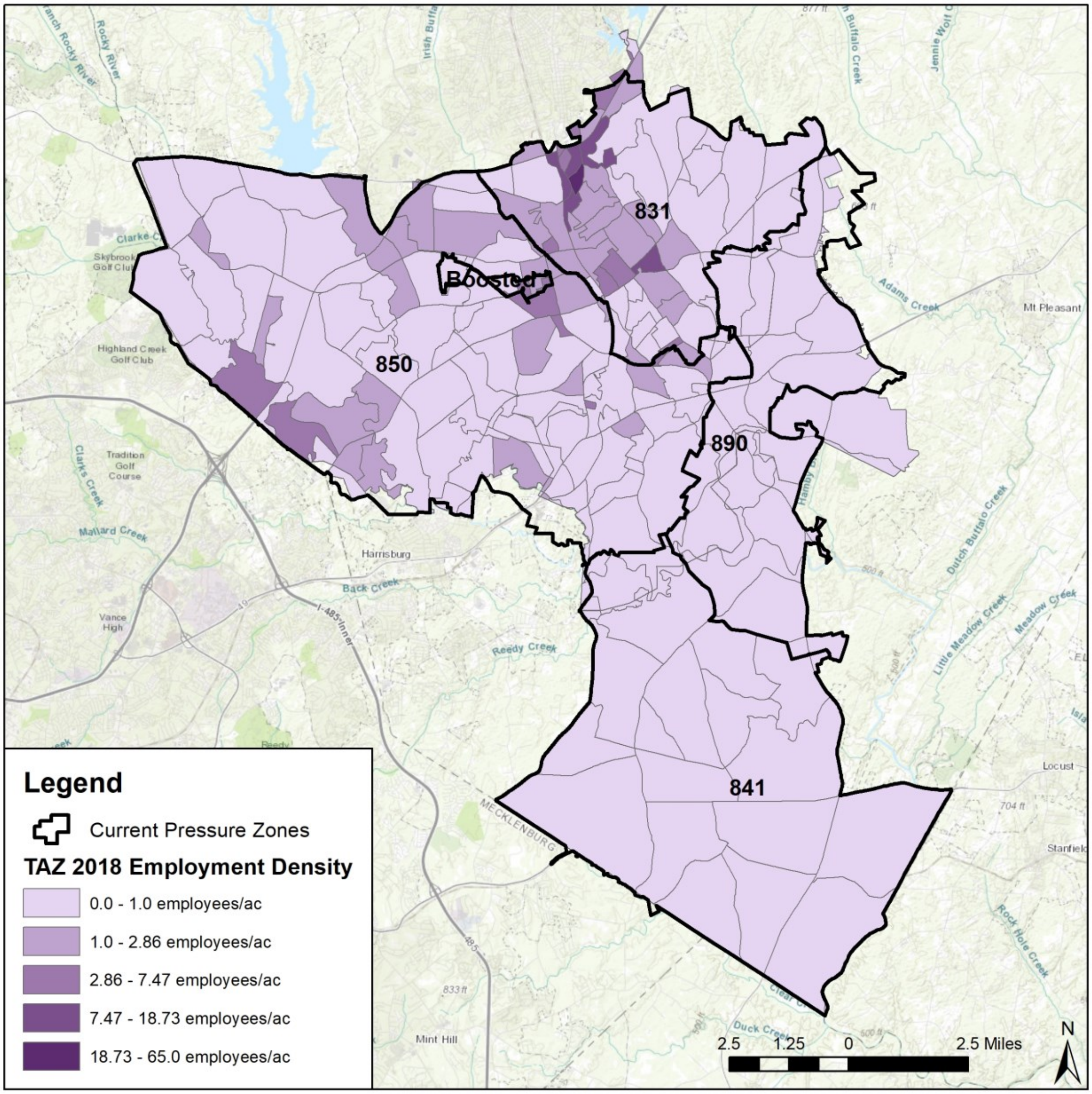


Figure 6-8: TAZ Employment Data 2018

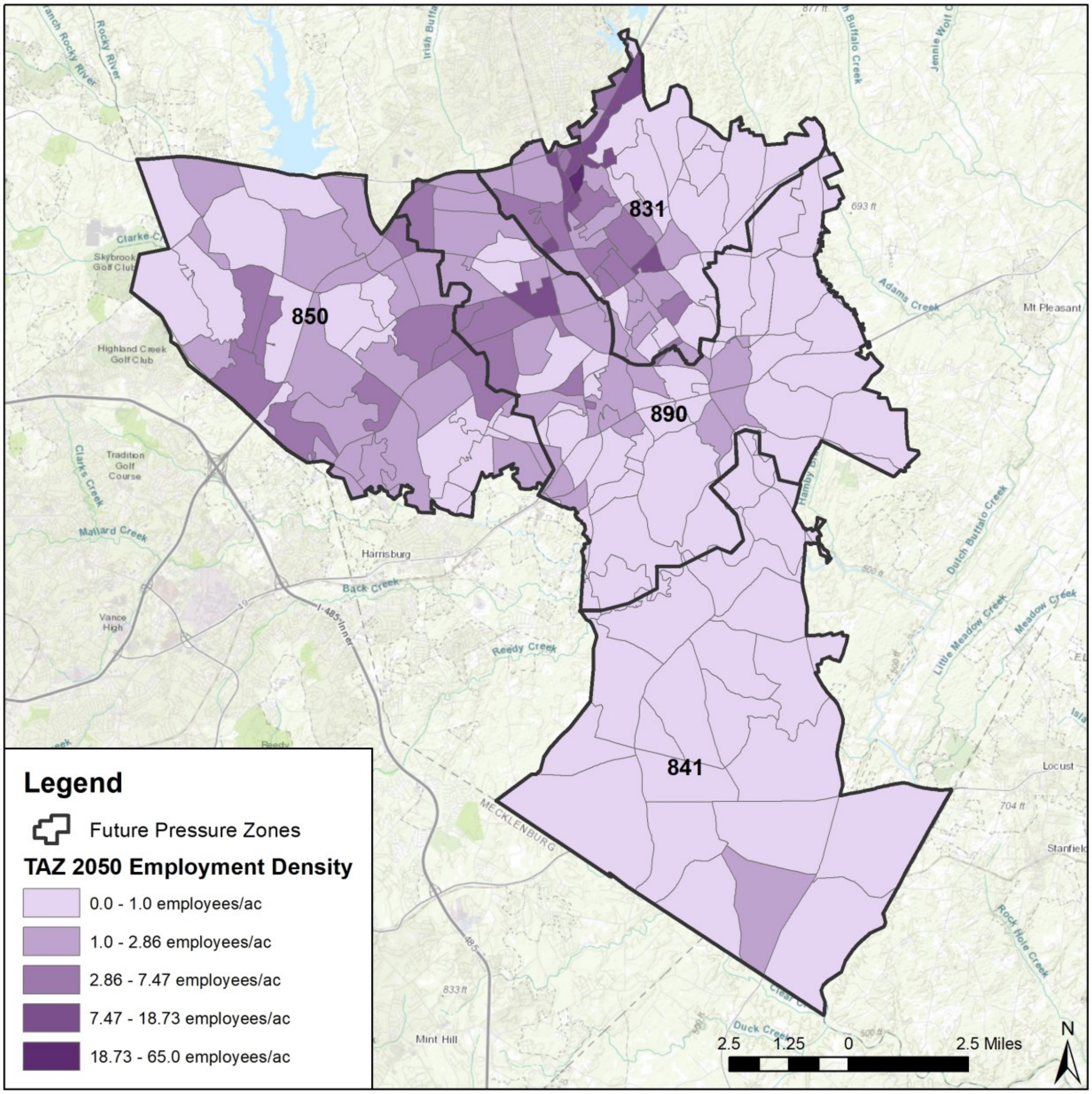


Figure 6-9: TAZ Employment Density 2050

B. Water Demand Intensity

Water demand for retail customers (i.e., non-wholesale customers) was divided into residential and non-residential use categories. The non-residential demand is comprised of the industrial, commercial, and institutional demand as reported on the City's Local Water Supply Plan (LWSP). Mean demand intensity was calculated for each use category. The residential and non-residential demand intensity were estimated from 2011 to 2019 LWSP data in conjunction with available calculated service area population employment data.

C. Residential Demand

The residential unit demand was calculated in units of gallons per capita per day (GPCD) and is displayed in Table 6-3. Though there is a slight negative trend in the unit usage, the trend is not statistically significant, and the City is not expecting unit residential demand to decrease over the planning horizon. Therefore, the average GPCD of 55.2 was used to develop the residential demand projections.

Table 6-3: Residential Unit Demand

Year	Residential Demand (GPCD)
2011	58.6
2012	58.6
2013	52.5
2014	55.1
2015	56.4
2016	54.6
2017	52.1
2018	53.7
2019	55.3
Average	**55.2**

D. Non-Residential Demand

The projection method for the non-residential demand relies on TAZ employment data and requires a compatible unit usage expressed in gallons per employee per day (GPED). However, since the TAZ employment data from CRMPO only includes two reference dates (2018 and 2050) , the non-residential demand was first analyzed using unit usage expressed in gallons per account per day (GPAD) which was developed for the last nine years. Figure 6-10 displays the unit consumption by year using this surrogate metric (GPAD). The non-residential demand intensity appears to have been relatively constant until 2017 and 2018 when the unit demand dropped by 200 GPAD. However, since water use rebounded in 2019, 2017 and 2018 unit usage is assumed to be an aberration.

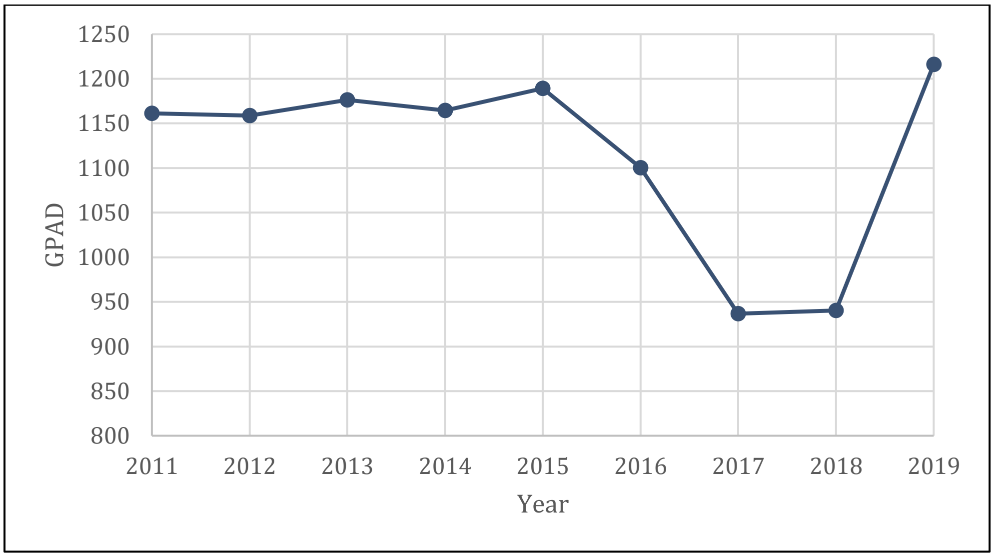


Figure 6-10: Non-Residential Unit Consumption Trend

To avoid using the anomalous 2018 non-residential demand in conjunction with the 2018 TAZ employment estimate, the service area employment for 2019 was estimated by linearly projecting between the 2018 and 2050 TAZ data as shown in the second column of Table 6-4. Table 6-4 also contains the resulting per employee non-residential unit demand in the desired GPED metric. The 2019 calculated value of 51.9 GPED was selected for use in developing non-residential demand projections.

Table 6-4: Unit Demand by Employment

Year	TAZ Number of Employees	Demand (MGD)	GPED
2018	59,759	2.44	40.8
2019	**61,513**	**3.19**	**51.9**
2050	116,509	-	-

In contrast to the assumption with residential population, it is assumed the City supplies water to all existing and future employees in the TAZ zones. The service area is projected to see a 95% increase in employment through 2050.

E. Site Specific Industrial Projects

In addition to the steady non-residential growth anticipated for the region and described in Section D, there are two industrial development sites expected by the City to create a significant increase in demand over the planning horizon. The locations of these two areas are displayed in Figure 6-11.

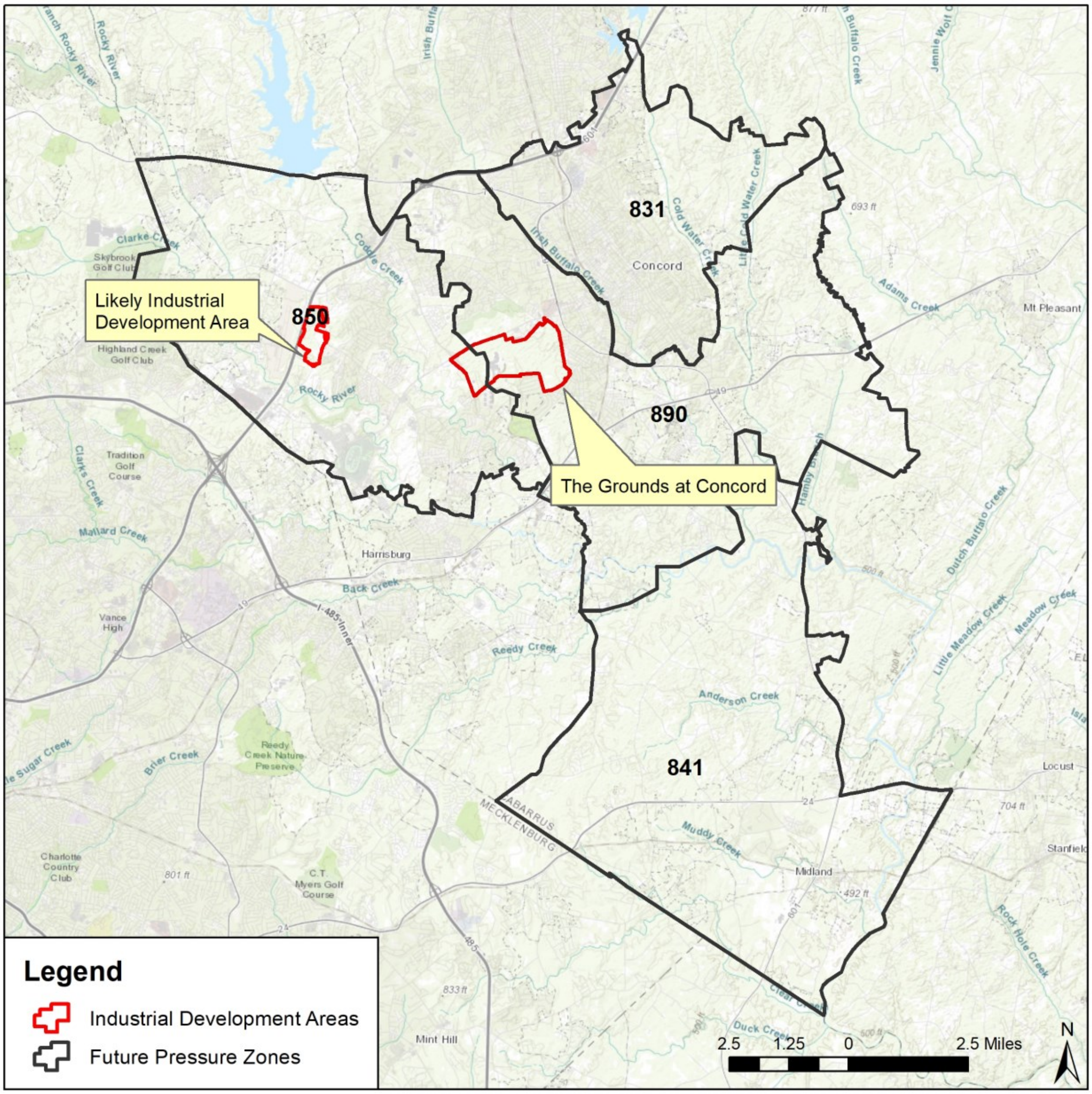


Figure 6-11: Industrial Development Area Locations

The City of Concord Planning Department identified the location shaded in yellow in Figure 6-12 as an area of likely future industrial growth based off future zoning and internal discussion. Using unit demand based off available square-footage for industrial buildings from the 2016 Water and Sewer Authority of Cabarrus County (WSACC) Master Plan of 150 gal/ksf, the average daily demand was estimated to be 0.75 MGD when fully developed. The City estimated this area may begin to develop in 2025 starting with half of the expected demand and would be in full service by 2030. A maximum day peaking factor for industrial demand of 2.5 was used as a conservative estimate for the max day demand forecast.

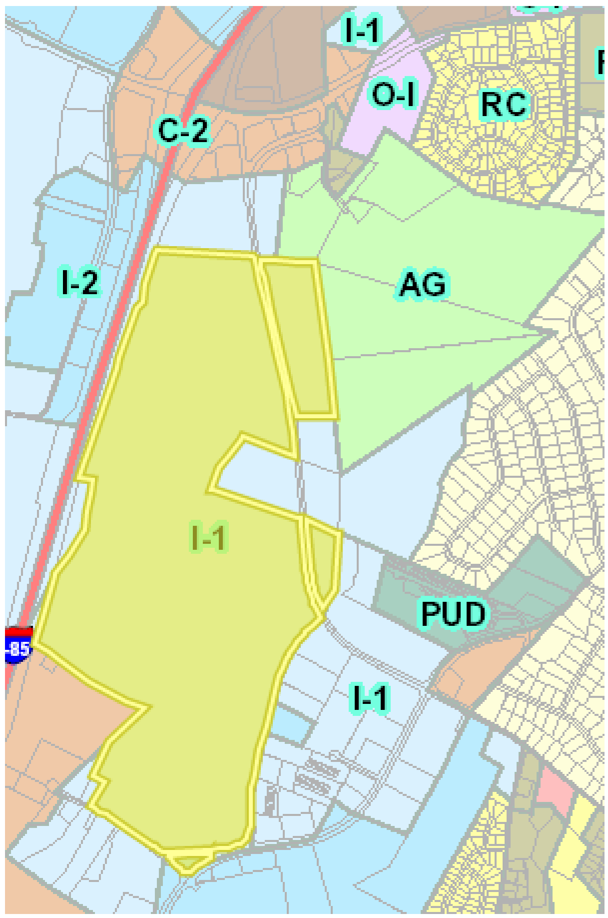


Figure 6-12: Likely Industrial Development Area

Additionally, a second major industrial development project known as The Grounds at Concord is under planning. The available development area is at the site displayed in Figure 6-13. Using the same methods based off the 2016 WSACC Master plan, the estimated average day demand was calculated to be 1.4 MGD with 3.5 MGD for the max day demand forecast. The City anticipates The Grounds at Concord will be fully developed and in service by 2025.

Figure 6-13: The Grounds at Concord Development Area

F. Wholesale Users

The City of Concord supplies water to several wholesale customers. Table 6-5 displays the current sales contract demands expected to remain through the 2050 planning period. The City does not expect any additional requests from the three municipalities and therefore the sales contract volumes have been added to the City's demand projections at their present rates. The maximum day demand corresponds to the contract value for each municipal customer. The maximum day peaking factor of 1.4 developed for the service area was used to estimate the average day demand based on the maximum contract for the purpose of the demand projections.

Table 6-5: Wholesale Demand

Utility	ADD (MGD) Estimated	MDD (MGD) Contract value
City of Kannapolis	2.5	3.5
Town of Harrisburg	2.1	3.0
Town of Mt Pleasant	0.14	0.2

G. Average Day Water Demand Projections

I. Methodology

The average and maximum day demand projections were developed by multiplying the unit usages by change in population and employment projections from 2019 and then adding the identified additional industrial area and wholesale customer demands. Several assumptions were made in developing these calculations:

- Residential and non-residential unit consumption will remain relatively constant through 2050.
- Based on the actual non-revenue water percentage of 8% in 2019, a conservative 9% of internal demand was added to account for non-revenue water. However, the City does not expect any changes in the non-revenue water fraction through the planning horizon.
- The yearly ADD demands were calculated using the change in population and employment projections (described in Section A) and added to the actual 2019 demand because this data was corrected to include missing billing data.

H. ADD Projections

Average day demand projections for the City's service area are shown in the red series in Figure 6-14. The ADD including wholesale contracts is shown in the blue series in Figure 6-14. The ADD forecast by pressure zone is shown in Table 6-6. The demands by pressure zone incorporate the boundary changes as described in Section 4 and only includes the internal system demand (i.e., does not include wholesale demand that may pass through the given pressure zone).

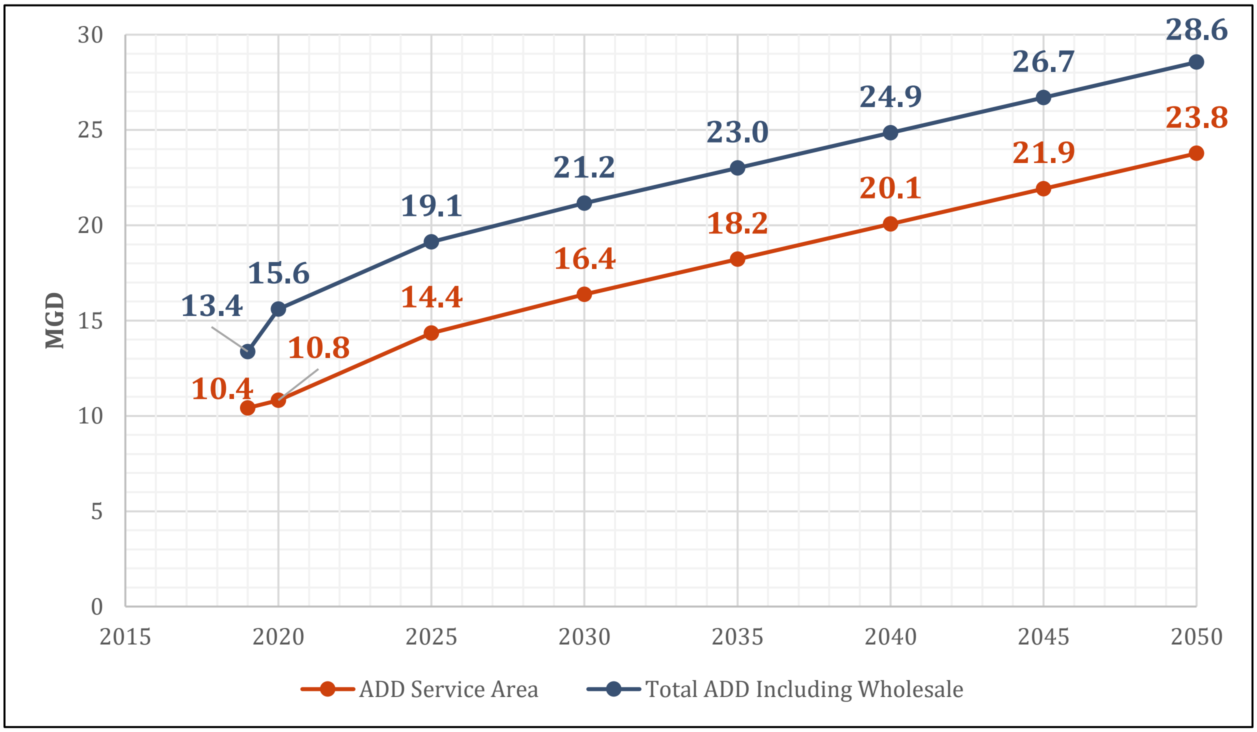


Figure 6-14: Average Day Demand Projections

Table 6-6: Average Day Demand by Pressure Zone[1]

Year	831 Zone	841 Zone	850 Zone	890 Zone	Total
2019	2.67	1.05	6.30	0.40	10.42
2020	2.83	0.95	4.03	3.01	10.82
2025	3.04	1.35	4.84	5.12	14.35
2030	3.34	1.69	5.65	5.69	16.37
2035	3.64	2.03	6.29	6.26	18.22
2040	3.94	2.37	6.93	6.83	20.07
2045	4.24	2.71	7.57	7.40	21.92
2050	4.54	3.05	8.21	7.97	23.77

[1]Totals do not include wholesale demands pumped through these pressure zones.

I. Maximum Day Water Demand Projections

I. Maximum Day Peaking Factor

A system-wide maximum day peaking factor was selected based on an analysis of historical demand. Figure 6-15 shows average day demand, maximum day demand, and maximum day peaking factors from 2007 to 2019. The actual maximum day from 2017 was removed from this analysis due to a pipe break. The highest maximum day peaking factor in the last ten years was approximately 1.4 and was therefore used to develop the future maximum day demands.

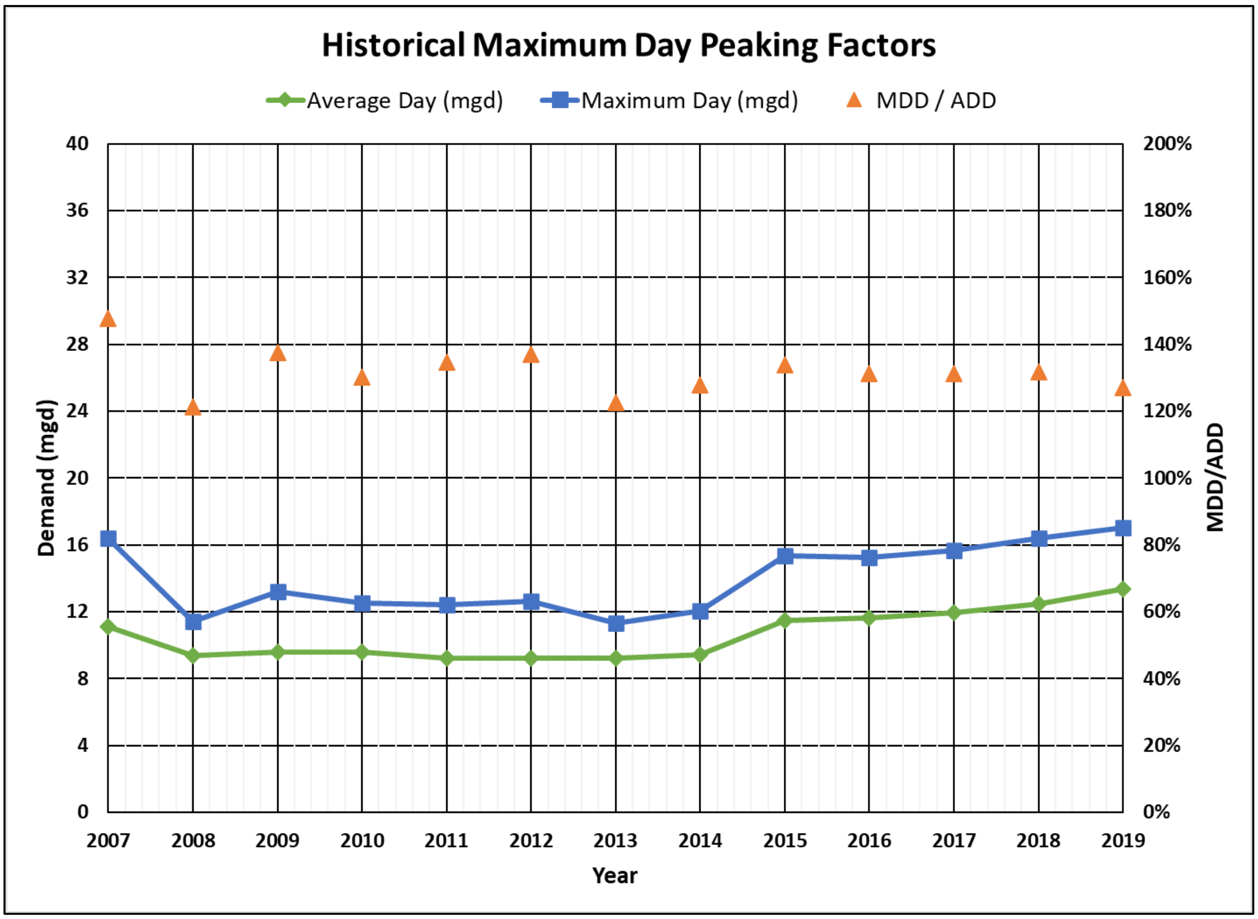


Figure 6-15: Historical Max Day Factors

II. MDD Projections

The total maximum day projections (including maximum wholesale contract values) are shown in Figure 6-16 alongside the complete average day projections. The sharp increase in MDD between 2019 and 2020 is due to the fact the 2019 value is an actual MDD for last year and for 2020 and beyond the line represents an upper bound projected value. The MDD projection includes the sum of maximum contract values for wholesale users and the historical high-end peaking factor of 1.4 within the service area. Therefore, the MDD projection shown in Figure 6-16 represents an upper bound that could occur, and that the City should plan to satisfy, but in a typical year the MDD will be several MGD less than this line indicates.

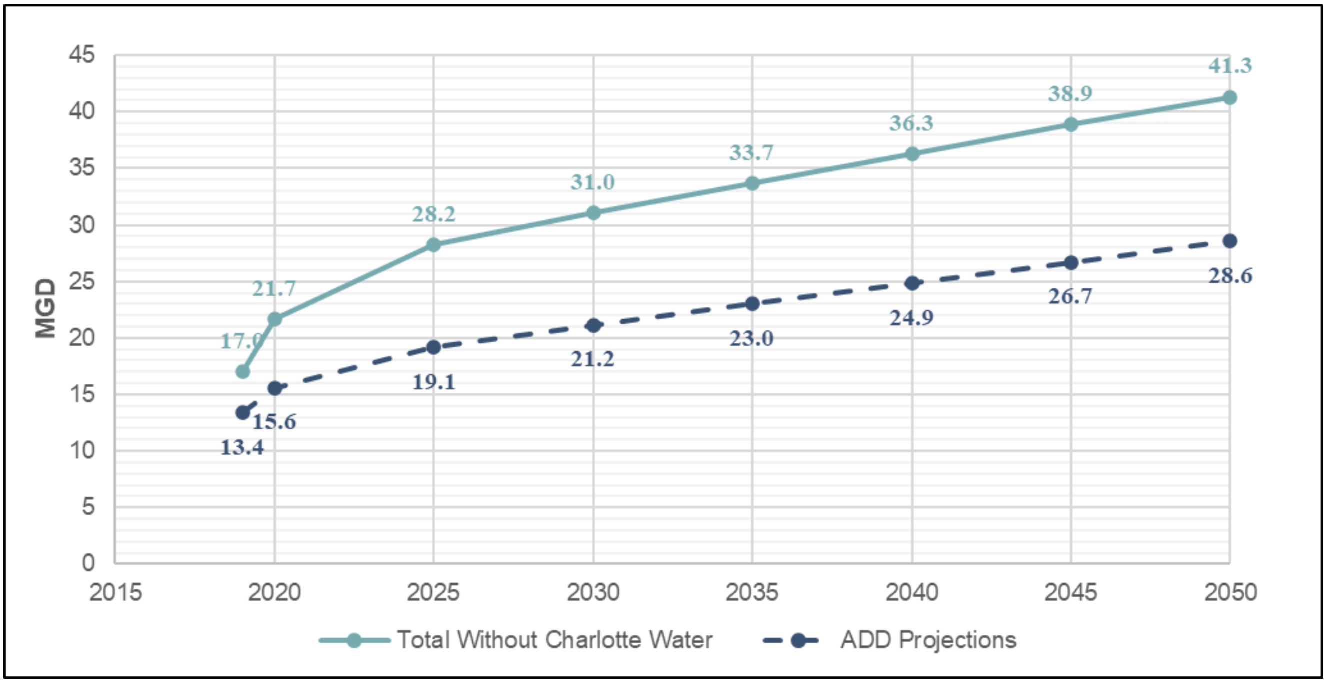


Figure 6-16: Maximum and Average Day Demand Projections

Table 6-7 and Table 6-6 shows the expected water supply for 2050 and Table 6-8 compares this to the projected 2050 maximum day demand. The total available supply and the maximum day demand for 2050 suggest there is sufficient supply capacity to meet future demand.

Table 6-7: 2050 Water Supply

Source	Capacity (MGD)
Hillgrove WTP	12 (permitted capacity)
Coddle Creek WTP	12 (permitted capacity)
Yadkin Purchase	10
Catawba Purchase	10
Total	**44**

Table 6-8: Total Supply vs. Max Day Demand

2050 Supply Total (MGD)	2050 Max Demand Total (MGD)
44	41.3

Section 7 Evaluate Future Demand Conditions

To improve system pressure in the 850 Pressure Zone, the 2016 master plan recommend enlarging the 890 Pressure Zone, as shown Figure 7-1. For this zone shift to occur, the following recommendations from the 2016 master plan are required:

- 30-inch water main along NC 49 crossing Cold Water Creek (completed)
- 24-inch water main along Polar Tent Rd (under construction)
- Upgrade Corban Pump Station (completed)
- Construction of New 2 MG Tank (under construction)

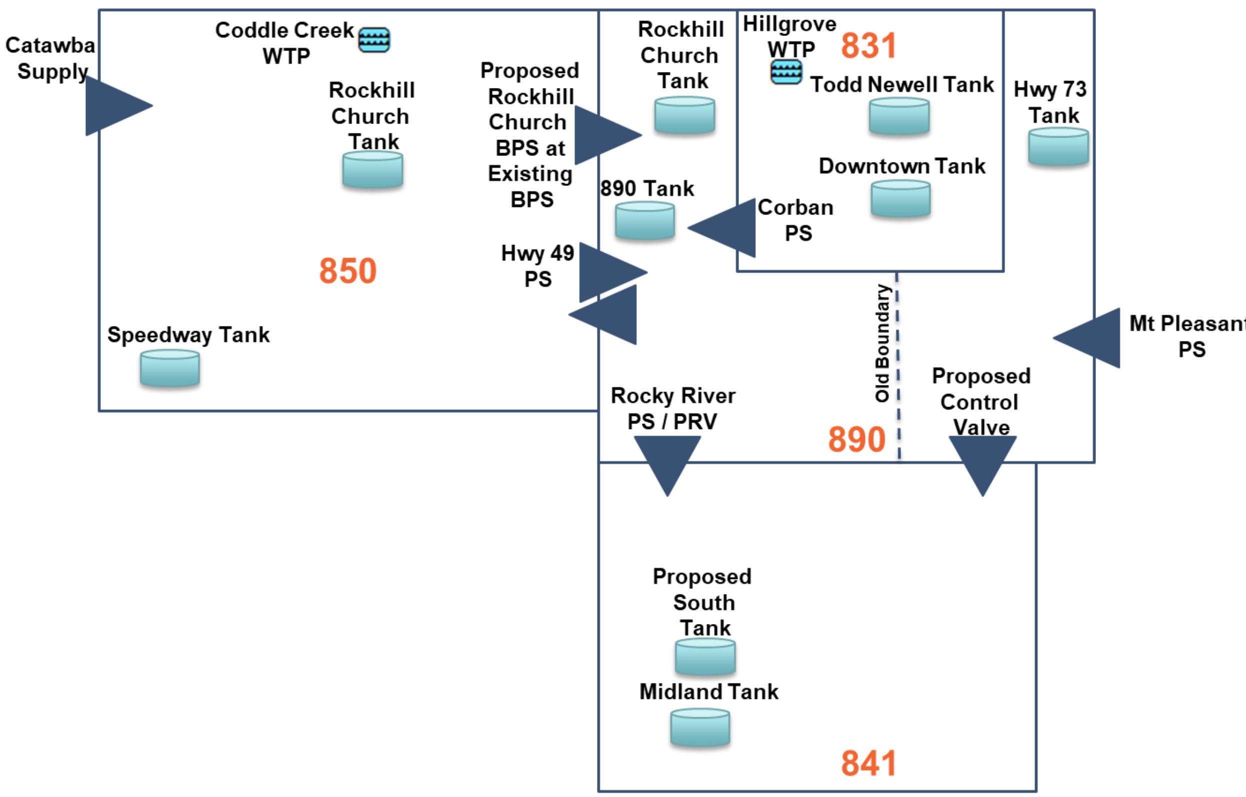


Figure 7-1: Future Pressure Zones

System pressures in the 850 Pressure Zone are above 30 psi after these improvements and boundary shift are complete.

A. Pump Capacity Evaluation

Figure 7-1 evaluates 2050 pump capacity by pressure zone by comparing projected maximum day demands to existing firm capacity (the firm capacity of the pump station is the total rated pump capacity) assuming the largest pump in each pump station is out of service. This table considers the proposed shifts in pressure zone boundaries as well as abandoning HWY 73 (left in place for emergencies) and HWY 601 pump stations. This analysis shows deficiencies in the 841 and 890 pressure zones. We recommend new booster pump stations and control valve as shown in the lower half of the table. The Catawba supply of 10 mgd supplements Coddle Creek supply to the full 890 pressure zone demand will be required before 2050. In addition, the smaller pumps at Coddle Creek will need to be replaced with pumps having at minimum similar capacity as the existing large pumps on VFDs to accommodate a larger ranger of demands. Demands in the table include wholesale customers.

Pump capacity evaluation tables for other years are shown in Appendix A.

Table 7-1: 2050 Pump Capacity Evaluation

2050 System	831	841	850	890
Pump Capacity				
Maximum Day – MGD	6.30	4.22	15.83	14.99
Supply to Other Zones	4.60	0.00	9.00	4.22
Total Pumping Requirement	10.90	4.22	24.83	19.21
Existing Firm Pump Capacity – MGD				
Hillgrove WTP	15.20			
Coddle Creek WTP			16.00[1]	
Hwy 601 PS Abandoned				
Rocky River PS (VFD)		2.80		
Highway 73 PS (Emergency)				
Mt Pleasant PS (VFD)				7.00
Corban PS (VFD)				4.60
Catawba Supply			10.00	
Total Pump Capacity – MGD	15.20	2.80	26.00	11.60
Pump Capacity Surplus/ Deficit	4.30	-1.42	1.17	-7.61
Proposed Improvements				
New Pumps at Existing BPS (850 into 890)				5.00
New HWY 49 PS (850 into 980)				4.00
New PRV/FCV (890 into 841)		3.33		
Total Pump Capacity – MGD	15.20	6.13	26.00	20.60
Pump Capacity Surplus/ Deficit	4.30	1.91	1.17	1.39

1. Assuming smaller existing pumps are replaced with similar size large pumps with VFD's (4 duty pumps at 4 mgd each)

B. Storage Capacity Evaluation

Table 7-2 evaluates storage capacity for 2050. The table calculates equalizing and fire storage in each pressure zone for comparison with existing storage capacity. Equalizing storage to allow constant pumping rates was calculated by multiplying maximum day demand without wholesale customers by a percentage that was derived from the diurnal demand pattern in each zone. This assumes wholesale customers provide their own equalizing storage. Fire storage requirements were calculated by multiplying the maximum need fire flow in each pressure zone by the durations recommended by the American Water Works Association (AWWA). The table then adds equalizing and fire storage requirements and compares it with the existing capacity of floating storage in each pressure zone.

The PWS requirement of half the average day demand without wholesale, approximately 11.9 MG, was also checked. Existing storage capacity including the WTP clearwells (4 MG each) and Mt Pleasant ground storage (1 MG) meets the requirement.

The equalizing and fire storage evaluation shows deficits in the 841 and expanded 890 Pressure Zones. We recommend a new elevated tank to be installed by 2035 to eliminate the deficit in the 841 Pressure Zone. The deficit in the 890 Pressure Zone can be compensated by the proposed booster pumps from the 850 Pressure Zone into the 890 Pressure Zone. The size of the new tank and booster pump stations are discussed in the next section.

Storage evaluation tables for other years are shown in Appendix B.

Table 7-2: 2050 Storage Evaluation

2050 System with All Service Areas	831	841	850	890
Equalizing Storage				
Average Day Demand – MGD[1]	4.54	3.05	8.21	7.97
Maximum Day Demand – MGD	6.30	4.22	11.48	12.69
Equalizing Percentage	13%	16%	14%	20%
Equalizing Volume – MG	0.82	0.67	1.61	2.54
Fire Storage				
Needed Fire Flow – GPM	3000	2500	3000	3000
Duration – Hours	3	2	3	3
Fire Storage Volume – MG	0.54	0.30	0.54	0.54
Equalizing + Fire Storage Volume – MG	1.36	0.97	2.15	3.08
Existing Storage - MG				
Clearwell - Hillgrove WTP	4.00			
Clearwell – Coddle Creek WTP			4.00	
Midland		0.50		
Speedway			1.00	
Rockhill Church			2.00	
Downtown	0.50			
Todd Newell	1.00			
HWY 73				0.75
HWY 29[2]				2.00
New Midland South Tank		1.00		
Total Floating Storage – MG	1.50	1.50	3.00	2.75
Equalizing & Fire Storage - MG				
Storage Capacity Surplus/Deficit	0.14	0.53	0.85	-0.33

[1] Without wholesale demand
[2] Construction complete in 2021

C. Proposed Improvements to Eliminate Deficiencies

To understand what is needed in the future, the model simulated the existing system along with the 2016 master plan CIP recommendations supplying future maximum day demand of 41.3 mgd. Figure 7-2 shows the resulting headloss though the system. Each tick mark represents 1 foot of headloss or approximately 0.4 psi. This helps us visualize where larger transmission mains are required and where the new 841 Pressure Zones storage tank would be most advantageous.

Major improvements are described below and include:

- New transmission main (7 miles) and new tank in 841 Pressure Zone

- New pump station between 850 and 890 Pressure Zones (at existing Rockhill Church PS)
- New pump station between 850 and 890 Pressure Zones (Hwy 49)
- New Control valve on Hwy 601

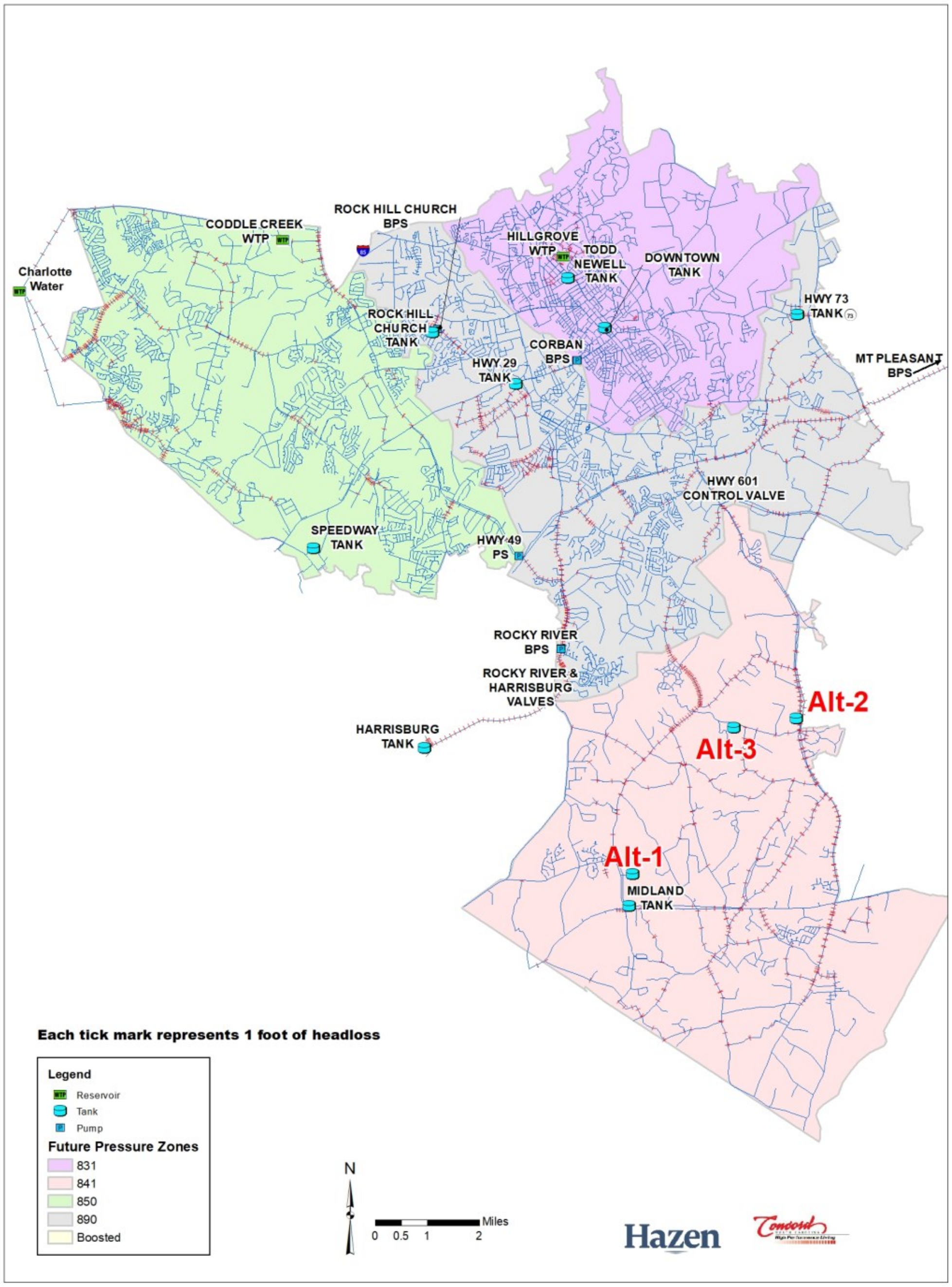


Figure 7-2: Headloss under 2050 conditions

A map of the major 2016 master plan improvements and new major improvements from this study are shown in Figure 7-3.

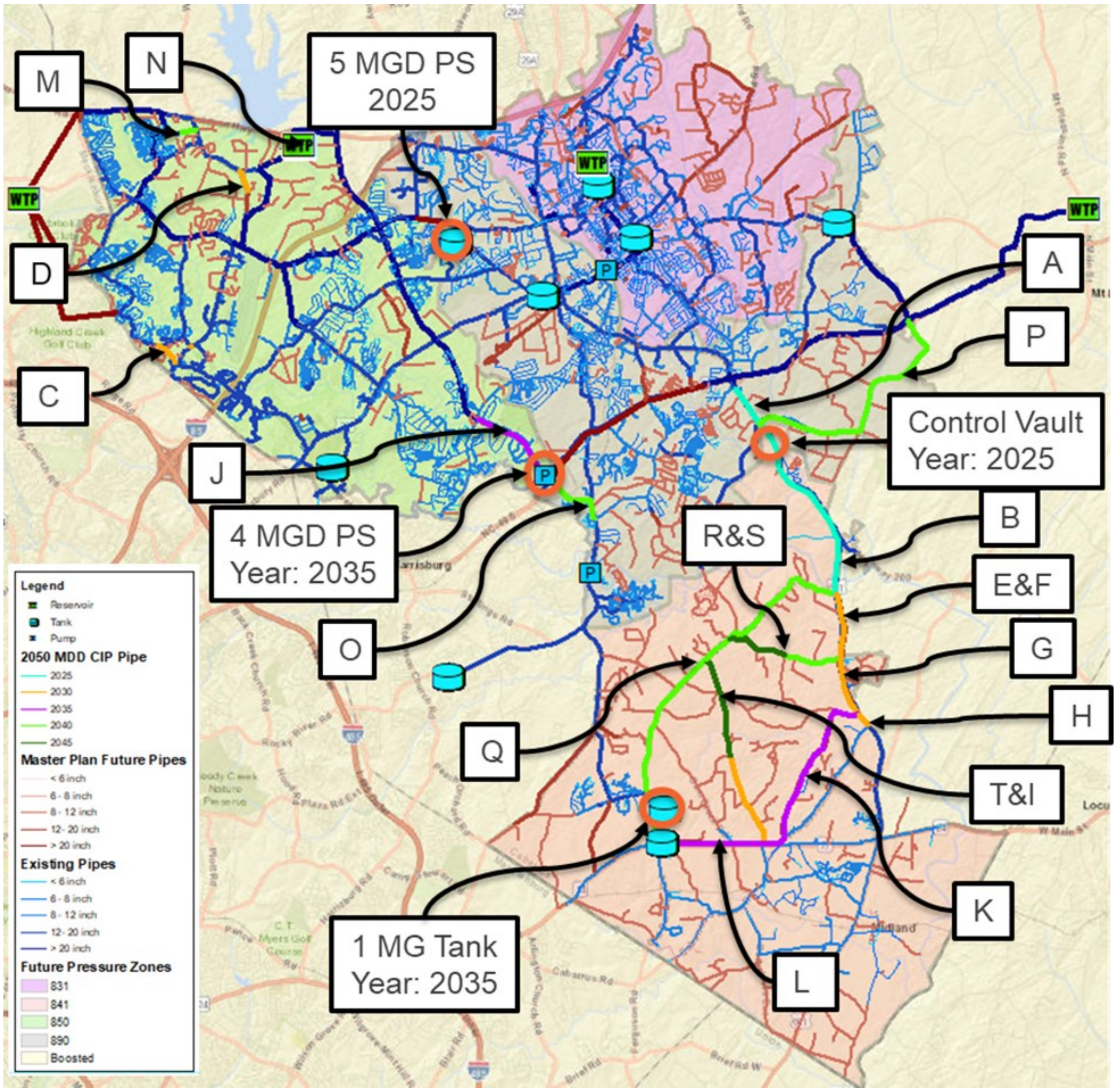


Figure 7-3: CIP Major Recommendations

Major transmission mains are needed to carry water into the 841 Pressure Zone is required to meet the Midland growth. A new tank in the 841 Pressure Zone is to meet equalizing and fire storage volume. Three locations for the 1 MG tank were based on a call with the city staff. During this call, the city staff preferred a 1 MG tank vs. the 0.5 MG storage required to meet storage deficiency.

- Alternative-1: South location off Flowes Stores Rd with a 122 ft tank.
- Alternative-2: North location off US Hwy 601 with a 162 ft tank
- Alternative-3: North location off Flowes Store Rd with a 111 ft tank.

The different heights for each option are needed based on ground elevation to achieve an overflow elevation of 841 feet.

Locating the new north 162 ft tank off US Hwy 601 and 111 ft tank off Flowes Store Rd required approximately 7 miles of 24-inch pipe from the proposed tank to Midland tank for an adequate hydraulic connection. Placing the new south 122 ft tank off Flowes Store Rd reduced the required pipe to 16-inch pipe providing a saving of $5.2 million. We recommend the alternative 1 site or a site near this location due to reduced transmission main cost and a lower height (122 ft compared to 162 ft).

The new control valve on Hwy 601 will supply water from 890 Pressure Zone to 841 Pressure Zone to help meet demand and supply the Pressure Zone 841 tanks. We recommend a valve that opens, and closes based on Midland tanks from overflowing.

The new pump station in the existing Rockhill BPS building will utilize the existing 12- and 8-inch water mains along Rockhill Church Rd. These mains will be in the expanded 890 Pressure Zone on the discharge side of the pump station. For this pump station to work, new suction piping will be required connecting to the fill line from the Rockhill Church Tank that is still within the 850 Pressure Zone. This connection will help pump performance. The site is already owned by the city, allowing quicker construction. We recommend a 5 mgd pump station at 65 ft TDH to meet 2025 demands and beyond. The driver for this pump station is the projected demand of the Grounds at Concord. We recommended two duty pumps on VFDs and one standby pump.

The new pump station on Hwy 49 will serve two purposes. For normal operation, it will feed the new 16-inch line supplying Rocky River BPS and the existing PRVs that send water to the 841 Pressure Zone. We also recommend configuring the pump station so that it can pump from the 890 Pressure Zone to 850 Pressure Zone in case Coddle Creek WTP is out of service. We recommend a 4 mgd pump station at 44 ft TDH to meet the 2035 demands. This pump station relies on the construction of the 24-inch along the George Liles PKWY extension from Roberta Rd to Hwy 49, as shown in Figure 7-3.

Other water mains to improve fire flow are explained in the next section of this report.

With these major improvements in service, the model predicted satisfactory tank performance in an extended period simulation of 2050 maximum day demands. The tanks balance as shown in Figure 7-4. The recommended improvements provide a robust system to meet the future demands.

Figure 7-5 shows predicted peak hour pressures with proposed improvements and zone boundary shifts. Predicted pressures meet PWS criteria in all pressure zones, except near Todd Newell Tank.

The model was also used to test proposed improvements to increase deficient fire flows. Proposed improvements provide major reductions in the number of hydrants with deficient fire flows, as shown in Figure 7-6 and Figure 7-7.

A water age map with proposed improvements for average day demand is shown in Figure 7-8. Water age has improved from the current system primarily due to demand increase.

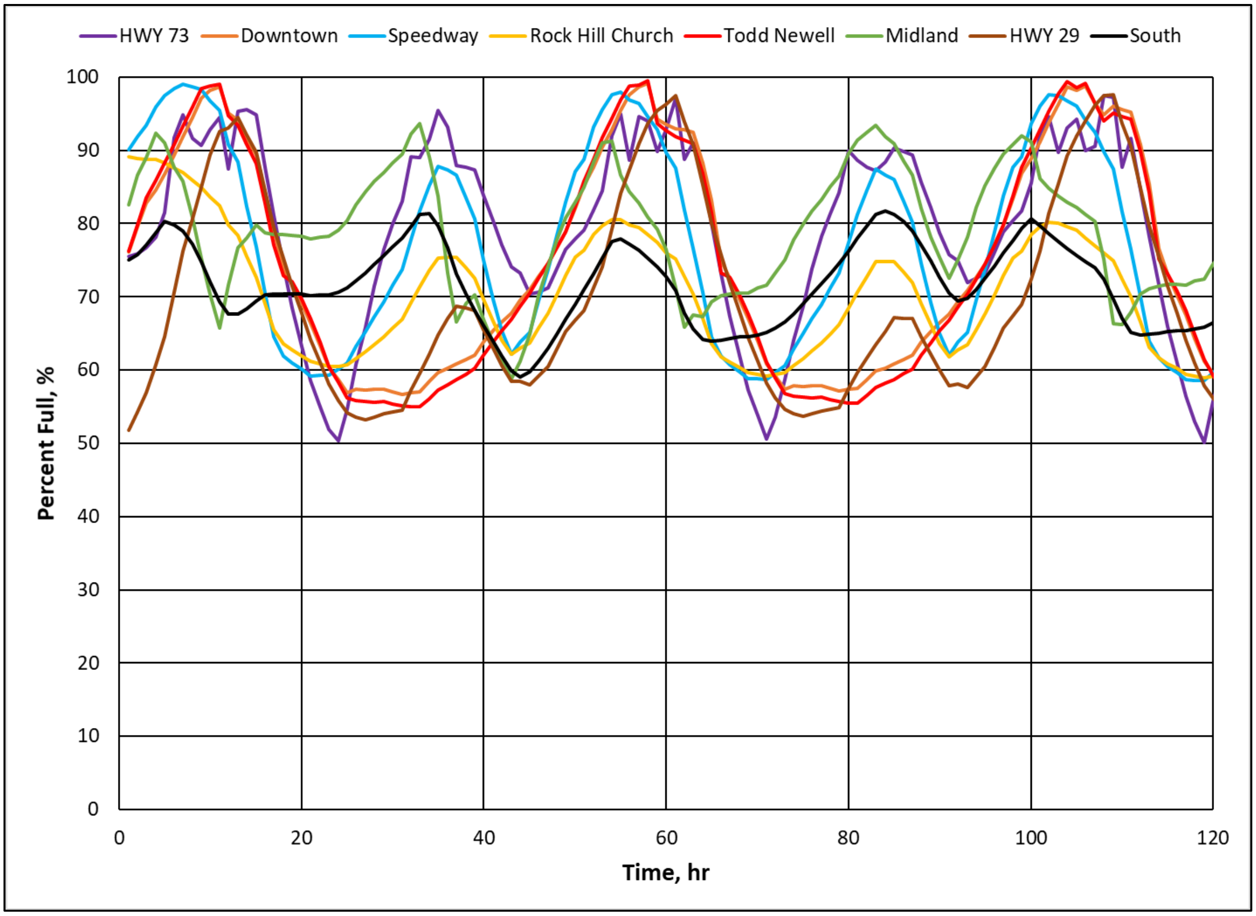


Figure 7-4: Model Predicted Tank Water Levels with 2050 Maximum Day Demands and Proposed Improvements

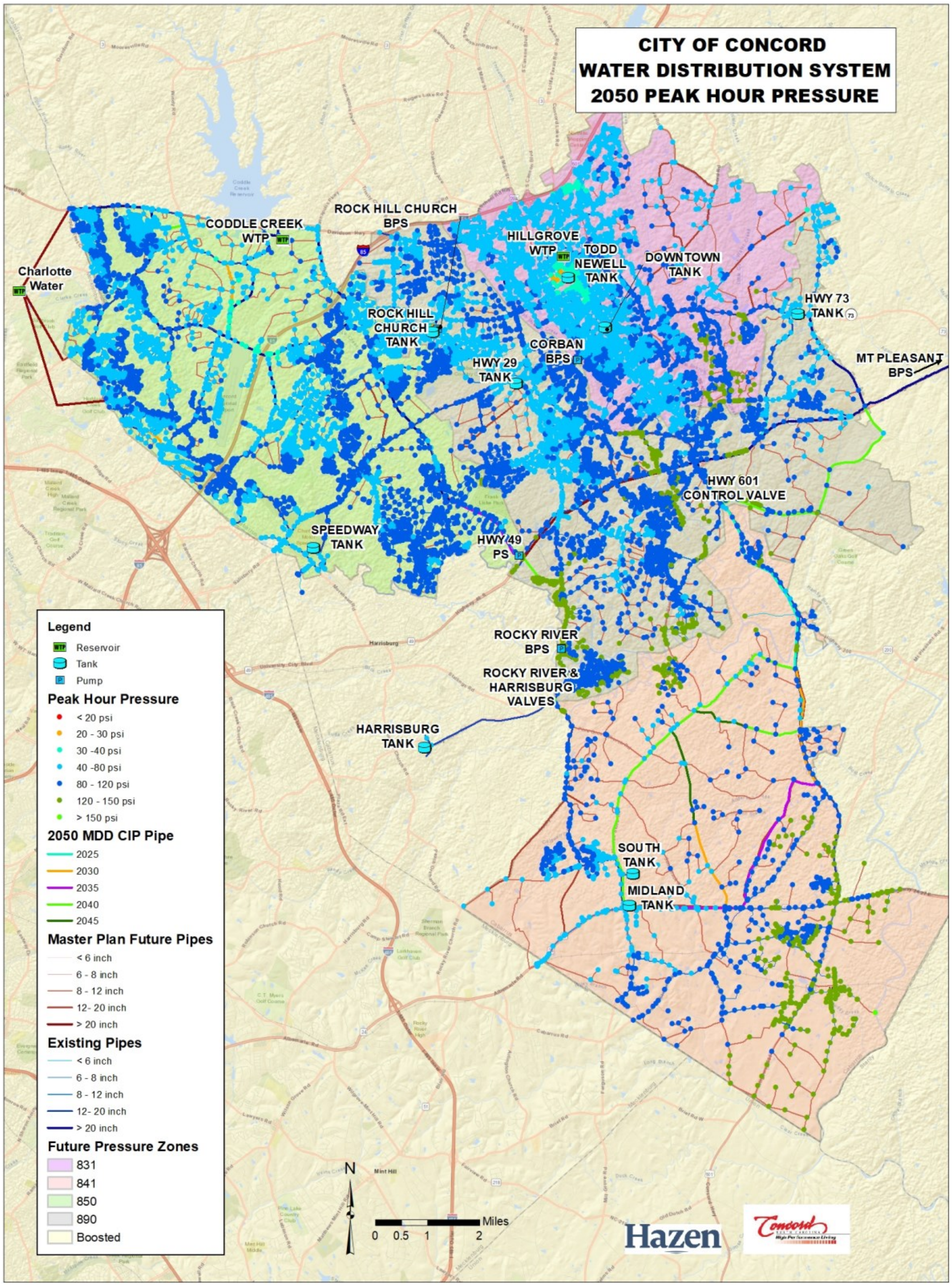


Figure 7-5: Predicted 2050 Peak Hour Pressures with Proposed Improvements

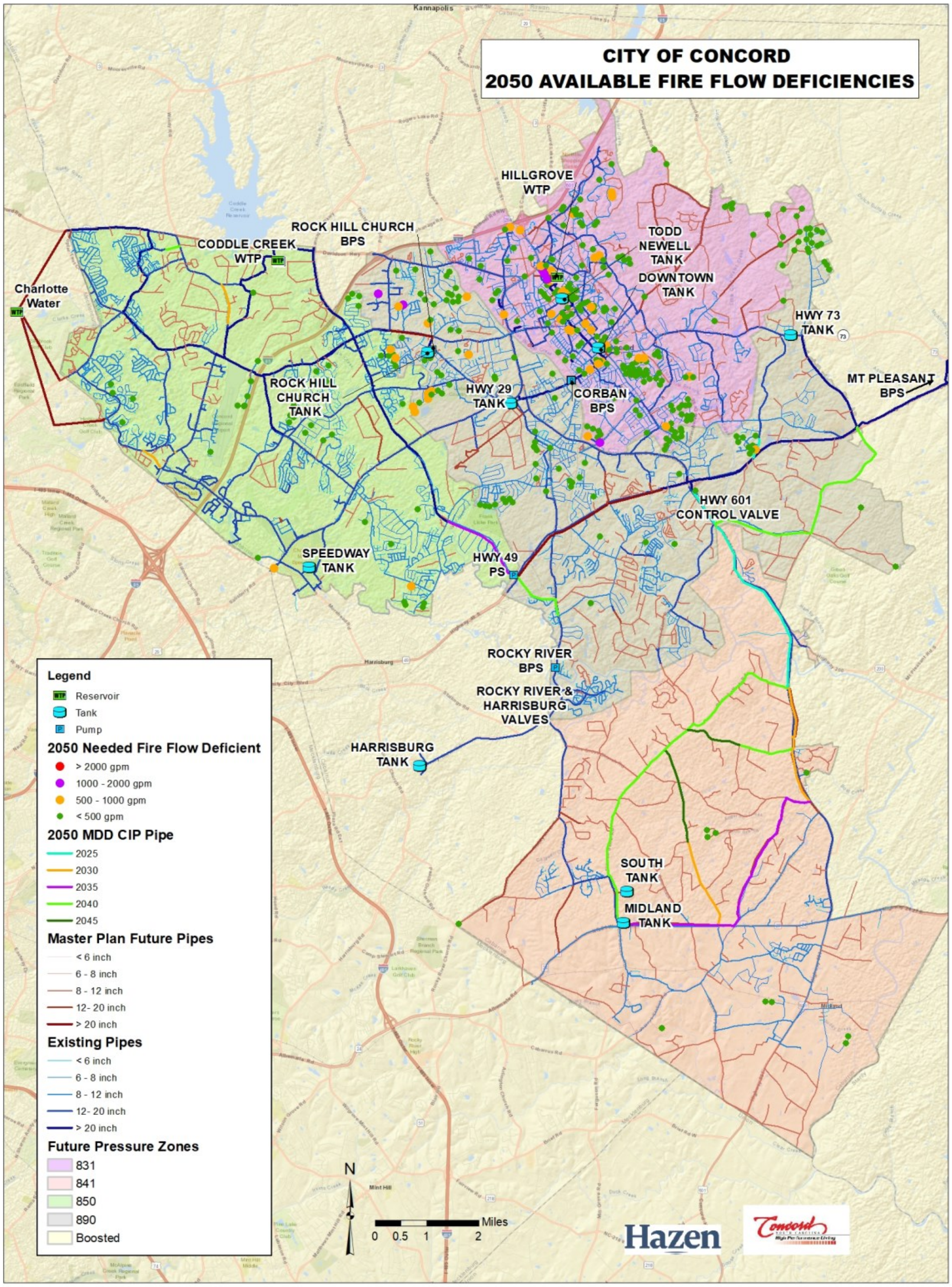


Figure 7-6: Predicted 2050 Fire Flow Deficiencies with Proposed Improvements

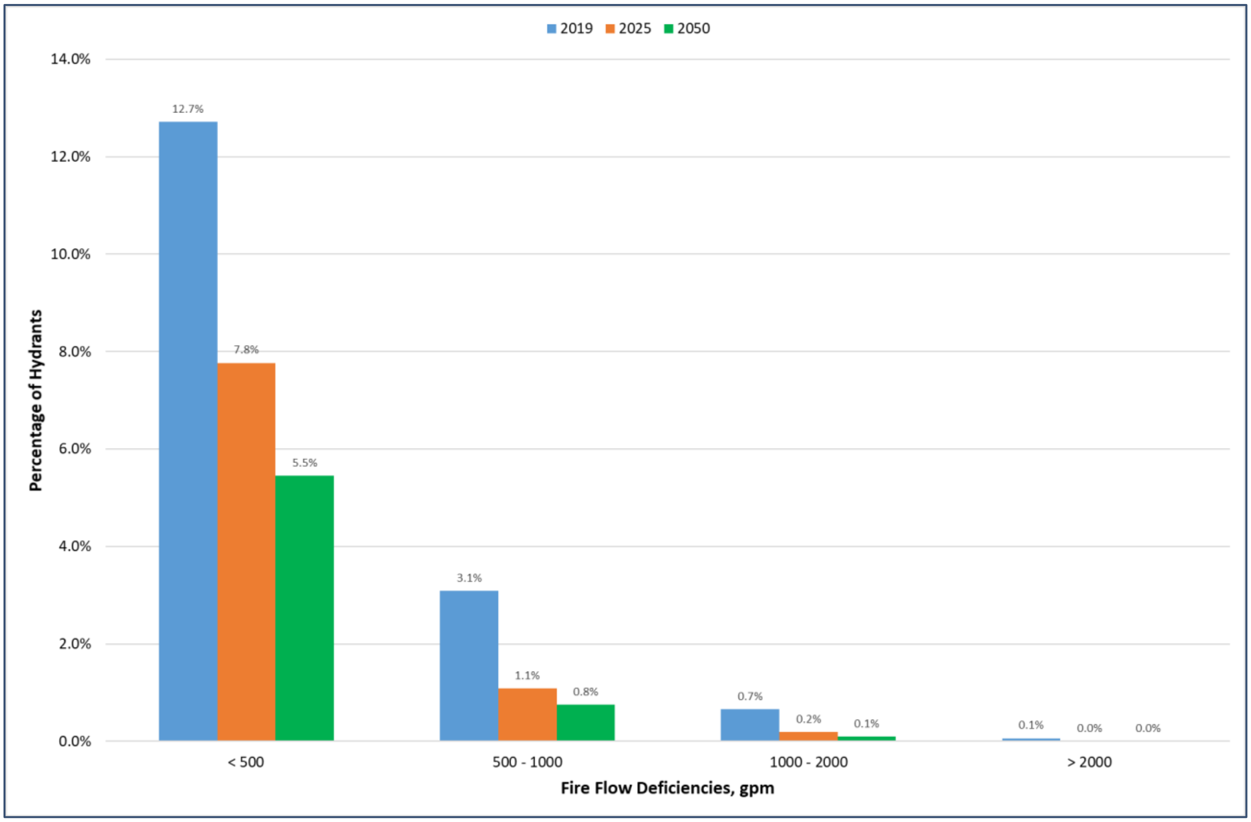


Figure 7-7: Fire Flow Deficiencies (Needed Fire Flow Minus Available Fire Flow)

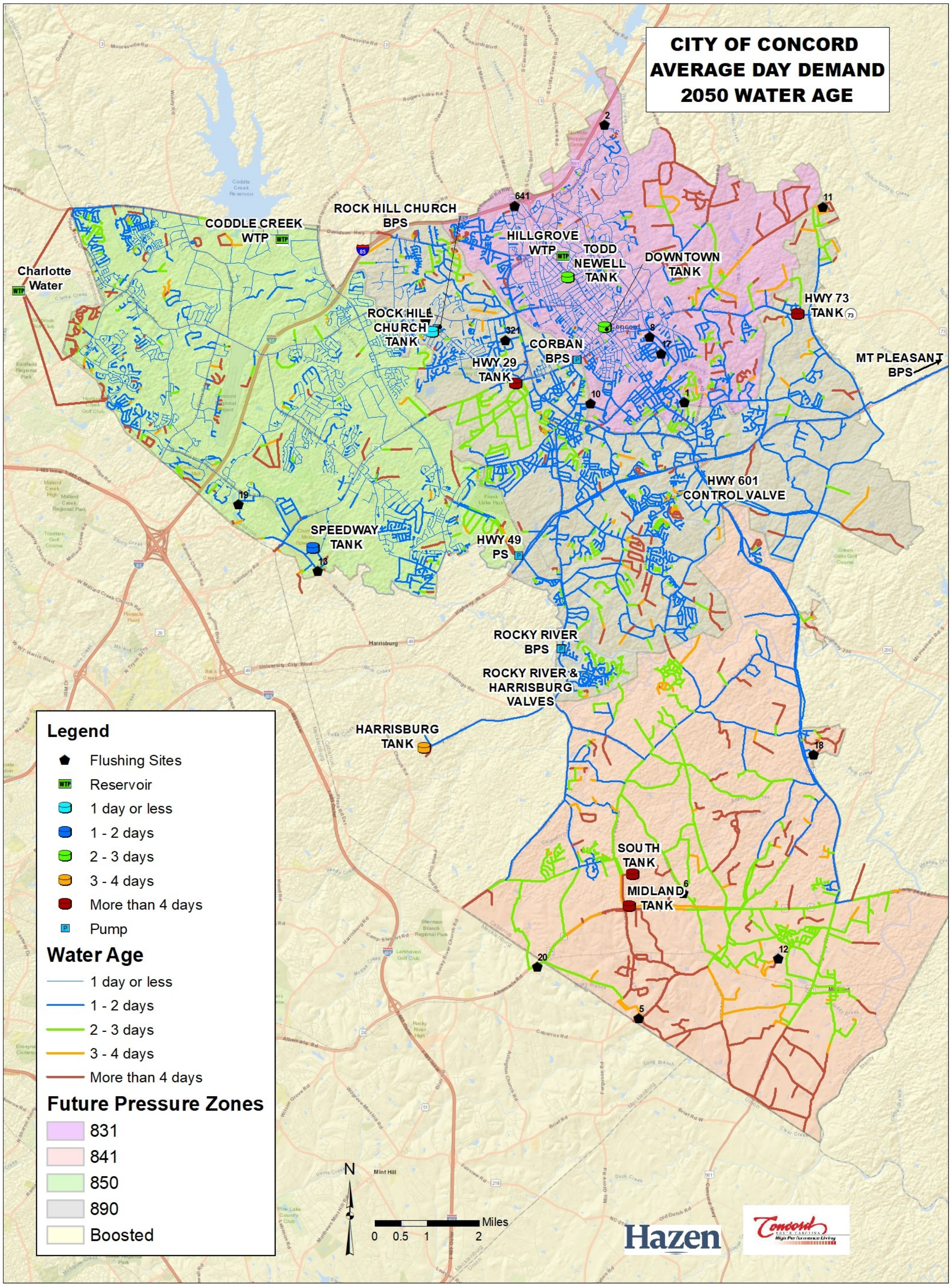


Figure 7-8: Model Predicted Water Age for 2050 Average Day Demand

Section 8 Capital Improvements Plan

Table 8-1 summarizes the Capital Improvement Plan. Twenty-four major improvements were recommended to provide required hydraulic capacity and adequate pressures. Once maximum day demand reaches 80% of plant capacity, plant upgrades should be considered, which Hazen considers practice. The major recommendations are shown in Figure 8-1. Appendixes C and D provide details including improvements listing CIP priority, CIP year, reason for improvement, improvement ID number, pressure zone, location, pipe size, length, estimated construction cost, and related improvements.

Fifty-six (56) smaller recommendations to improve fire flows have a total estimated construction cost of $6.8 million, as shown in Appendix D.

The total estimated construction cost for all recommendations shown in Figure 8-1 is $95.7 million, 93% ($88.9 million) of which are the 24 major improvements.

Table 8-1: Summary and Schedule of Recommended Improvements

CIP Year	Improvement ID	Estimated Construction Cost ($)[1]
2025	PS 1; FC Vault; A; B	24,500,000
2030	C; D; E; F; G; H; I	11,440,000
2035	J, PS 2, Elevated Tank; K; L	24,970,000
2040	M; N; O; P; Q; R	23,894,000
2045	S; T	4,104,000
Total		**88,908,000**

[1] 2020$$

Figure 8-1: CIP Major Recommendations – Large Map


CITY OF CONCORD
CIP MAJOR RECOMMENDATIONS
Catawba Water
CIP - M
CODDLE CREEK WTP
CIP - N
CIP - D
ROCK HILL CHURCH BPS
HILLGROVE WTP
TODD NEWELL TANK
DOWNTOWN TANK
HWY 73 TANK
MT PLEASANT BPS
CIP - S MG PS1
ROCK HILL CHURCH TANK
CORBAN BPS
HWY 29 TANK
CIP - C
CIP - C
CIP - P
CIP - A
HWY 601 CONTROL VALVE
CIP - P
CIP - J
CIP - J
SPEEDWAY TANK
HWY 49 PS
CIP - O
CIP - B
ROCKY RIVER BPS
ROCKY RIVER & HARRISBURG VALVES
HARRISBURG TANK
CIP - E
CIP - S
CIP - R
CIP - F
CIP - Q
CIP - T
CIP - G
CIP - H
CIP - K
CIP - I
SOUTH TANK
MIDLAND TANK
CIP - L
Legend
Reservoir
Tank
Pump
2050 MDD CIP Pipe
2025
2030
2035
2040
2045
Master Plan Future Pipes
< 6 inch
6 - 8 inch
8 - 12 inch
12- 20 inch
> 20 inch
Existing Pipes
< 6 inch
6 - 8 inch
8 - 12 inch
12- 20 inch
> 20 inch
Future Pressure Zones
831
841
850
890
Boosted
N
0 0.75 1.5 3
Miles
Hazen
Concord
NORTH CAROLINA
High Performance Living

A. Improvements Schedule

I. Phase 1 Improvements: by 2025 (MDD demand up to 26.6 mgd)

The City's goals, between 2020 to 2025 or maximum day demand up to 20.7 mgd, are to maintain Hillgrove WTP supply to 6 mgd, Mt Pleasant PS to 4 mgd, and Catawba supply to 4 mgd. Coddle Creek WTP was not limited for this scenario.

In addition, we recommend expanding the 890 Pressure Zone expansion to utilize the new 2 MG Hwy 29 tank and to improve current low pressures. It can be done by reactivating a 12-inch connection from Concord Pkwy to Union Cemetery (on the property of The Grounds at Concord) and expanding the original Pressure Zone 890 west to include areas south of NC 49 and East Rocky River Rd. There is an existing 12-inch line on Union Cemetery Road that is a mostly dedicated water line going south on Jackson Terrace SW towards NC 49. This line will need to be isolated to make this phasing work, shown in Figure 8-2. It is necessary to verify that isolation valves are in the correct locations for this improvement to be effective. However, new valves may be inserted if necessary.

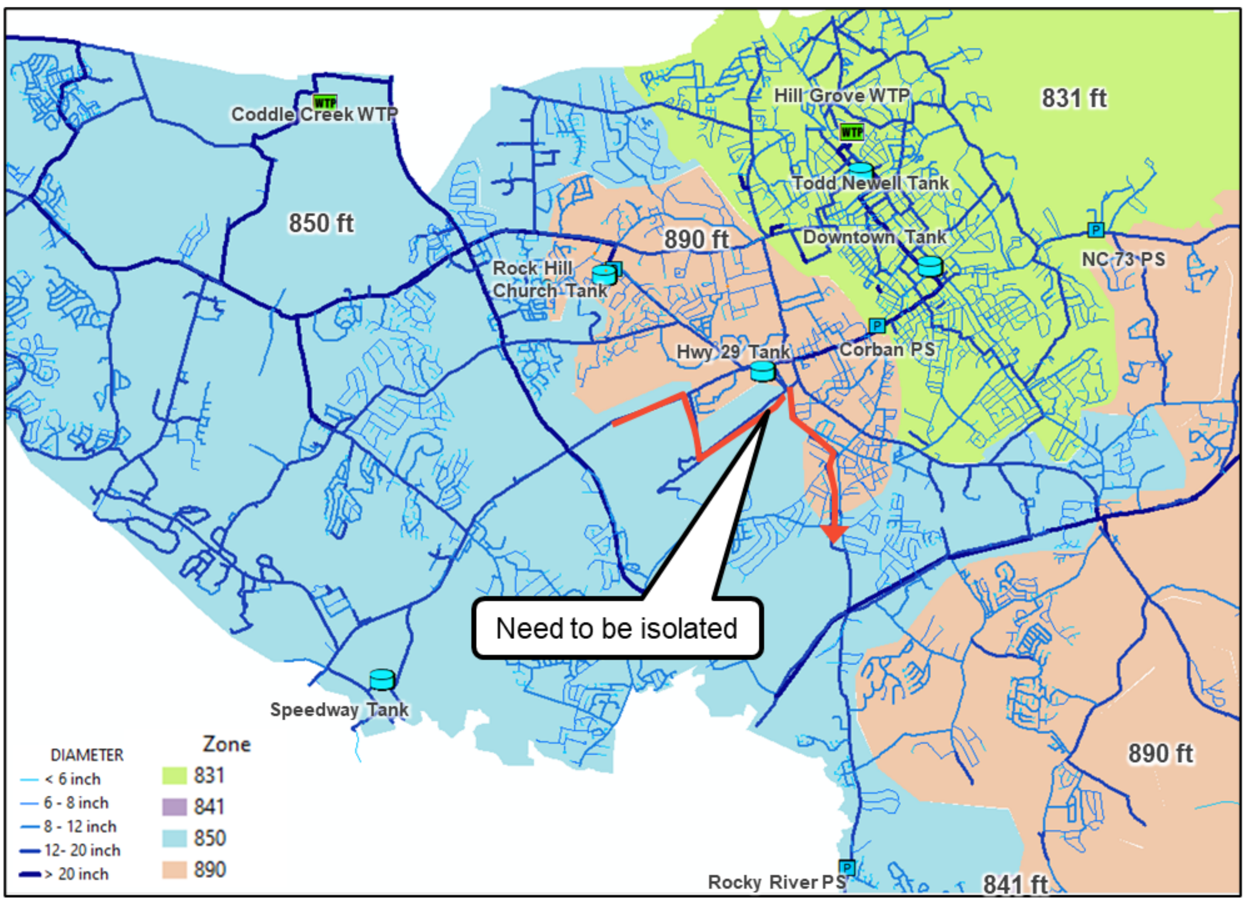


Figure 8-2: 890 Pressure Zone Expansion

By 2025 (28.2 mgd), we recommend building a 5 mgd PS in the existing Rockhill Church PS building. The driver for this pump station is the "Grounds at Concord" proposed maximum day demand of 3.5 mgd and to maintain Hillgrove WTP around 6 mgd until raw water line and WTP upgrades can occur.* Other recommendations, as shown in Figure 8-1, are:

- Flow control (FC) vault: near the intersection of Flowes Store Rd, Miami Church Rd, and US Hwy 60. The purpose of this valve is to help meet demand and supply the Pressure Zone 841 tanks.
- CIP A: 24-inch 4,400 LF water main along North US Hwy 601. Required to meet design criteria without Hwy 601 PS, due to the PS is under capacity.
- CIP B: 24-inch 20,000 LF water main along US Hwy 601 between Miami Church Rd and Parks Lafferty Rd. Required to meet design criteria without Hwy 601 PS.

To improve pressures in the 831 Pressure Zone near the Todd Newell Tank, we recommend operating the tanks above 23 feet if water quality permits. We recommend water quality sampling at the tank between and after the revised operations to confirm the proposed levels do not significantly deteriorate water quality.

The model also predicted that as system hydraulics change, the Coddle Creek smaller pumps will begin to run out on their curves. It is recommended for better pump performance that the 3.5-inch diameter orifice plate be added, as detailed on the Crane Deming Pump Test Data Sheet for these pumps.

This phase of improvements also addresses 56 existing hydrants with deficient fire flow. Priorities were assigned to eliminate the higher deficiencies first. These improvements were previously identified in the 2016 master plan.

Figure 8-3 summarizes these improvements. Appendix D provides details including the priority, deficiency in gallons per minute, CIP Year, diameter, length of repair, and construction cost. Unit costs of construction shown in Table 8-2 assume working in areas with many other utilities and significant traffic. The total estimated cost for fire flow improvements is approximately $6.8 million.

*As noted previously, the Grounds demands may change based on actual development.

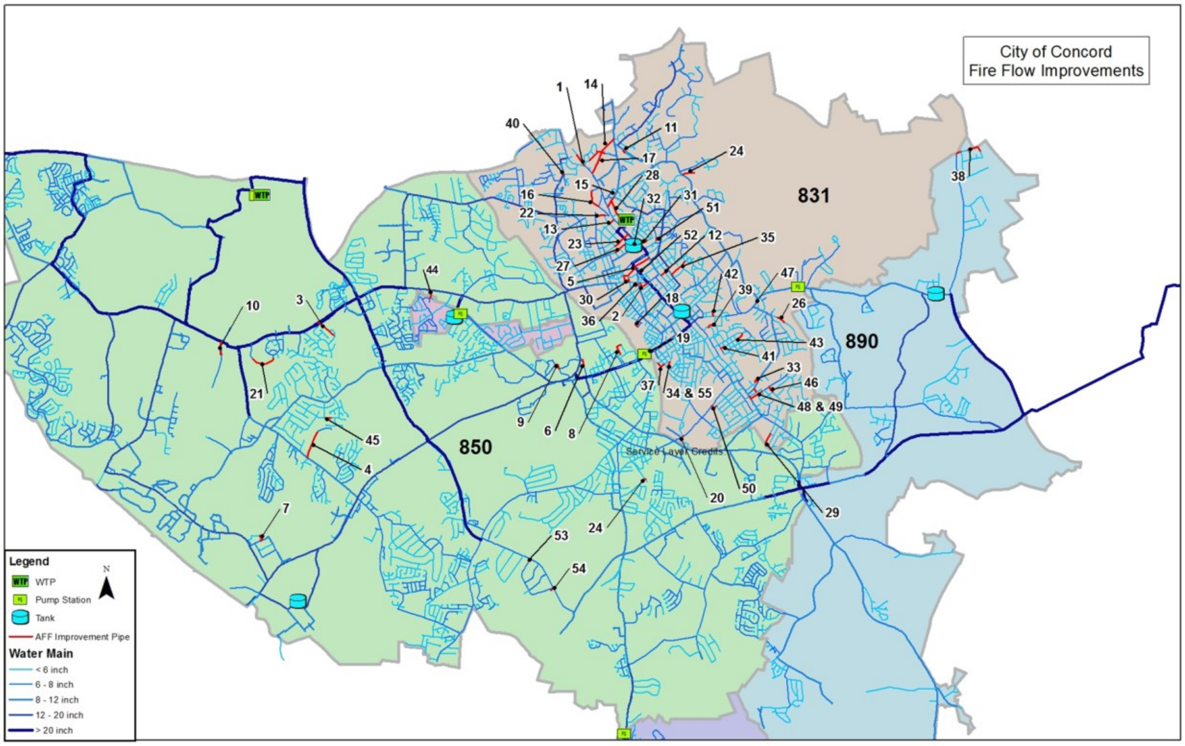


Figure 8-3: Fire Flow Improvements (Figure 36 from 2016 Master Plan)

Table 8-2: Unit Costs for Fire Flow Improvements

Construction Unit Costs	
Water Main:	$20/in-dia/LF
Engineering	15%
Contingency	30%

Figure 8-4 to Figure 8-7 show details of the major Phase 1 improvements.

PLANNING LEVEL COST ESTIMATE	ROCKHILL CHURCH RD BPS
Mobilization (3%)	$66,000
Piping	$125,000
Tie-ins/Hydrants/Services	$38,000
Erosion & Traffic Control, Restoration	$27,000
Pump Station (with VFDs)	$1,934,000
Construction Cost	**$2,190,000**
Contingency (30%)	$657,000
Construction Subtotal	**$2,847,000**
Design @ 15%	$329,000
Limited Construction Admin @ 5%	$110,000
Engineering Subtotal	**$439,000**
TOTAL	**$3,286,000**

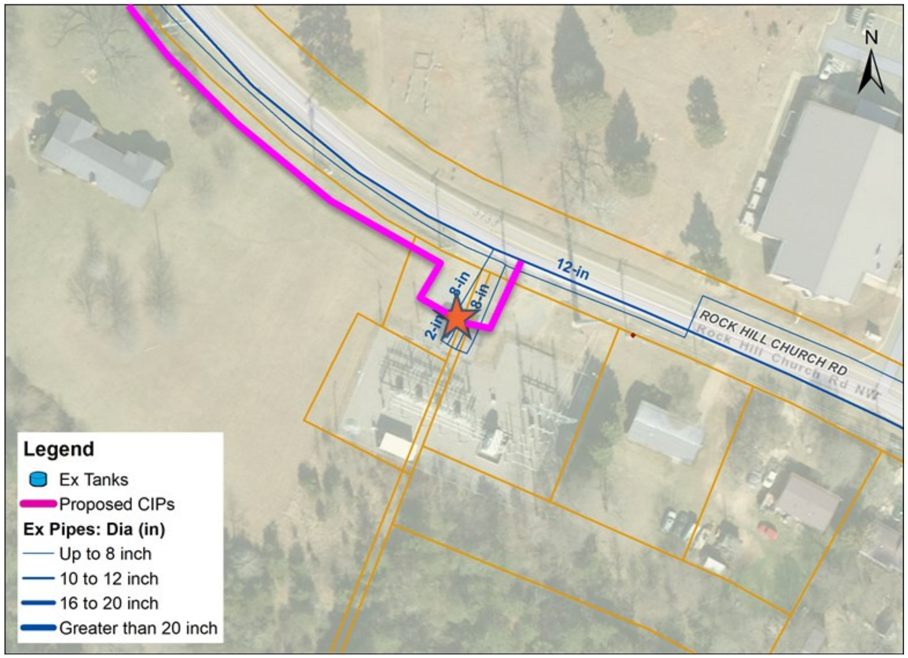


Figure 8-4: CIP–Pump Station 1-Along Rock Hill Church Road at the Existing Pump Station Site

PLANNING LEVEL COST ESTIMATE	US HWY 601 FLOW CONTROL VAULT
Mobilization (3%)	$6,000
Vault Structure	$25,000
Property Acquisition	$15,000
Valves	$50,000
Sitework, Piping and Telemetry	$104,000
Construction Cost	**$200,000**
Contingency (30%)	$60,000
Construction Subtotal	**$260,000**
Design @ 15%	$30,000
Limited Construction Admin @ 5%	$10,000
Engineering Subtotal	**$40,000**
TOTAL	**$300,000**

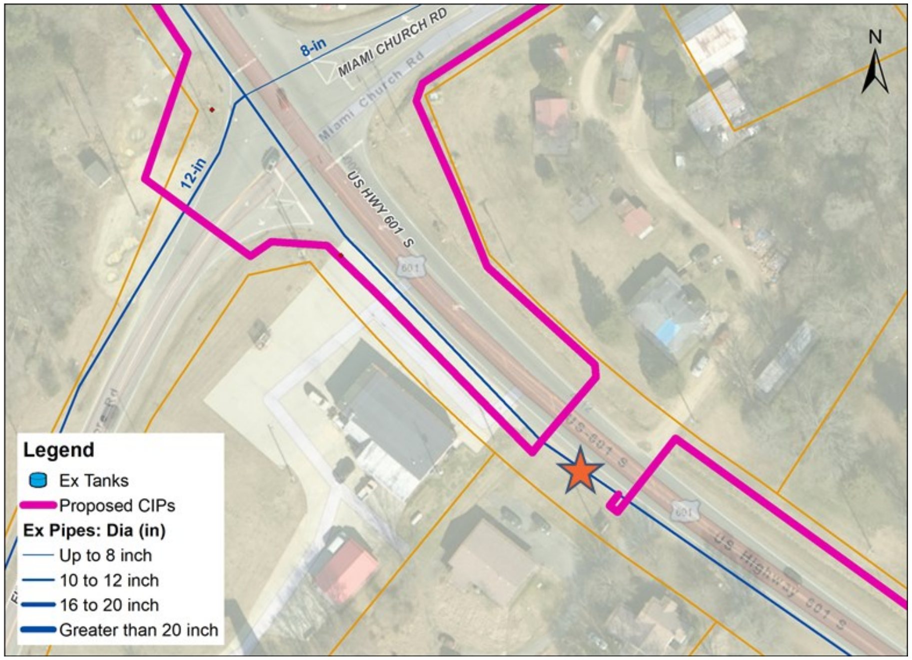


Figure 8-5: CIP–Flow Control Vault-Near the intersection of Flowes Store Rd, Miami Church Rd, and US Hwy 601

PLANNING LEVEL COST ESTIMATE	A: 24-in 4,400 LF along North US Hwy 601
Mobilization (3%)	$72,000
Piping	$1,750,000
Road & Stream Crossings	$340,000
Tie-ins/Hydrants/Services	$48,000
Erosion & Traffic Control, Restoration	$190,000
Construction Cost	**$2,400,000**
Rock Contingency (15%)	$360,000
Contingency (30%)	$720,000
Construction Subtotal	**$3,480,000**
Design @ 15%	$360,000
Limited Construction Admin @ 5%	$120,000
Engineering Subtotal	**$480,000**
TOTAL	**$3,960,000**

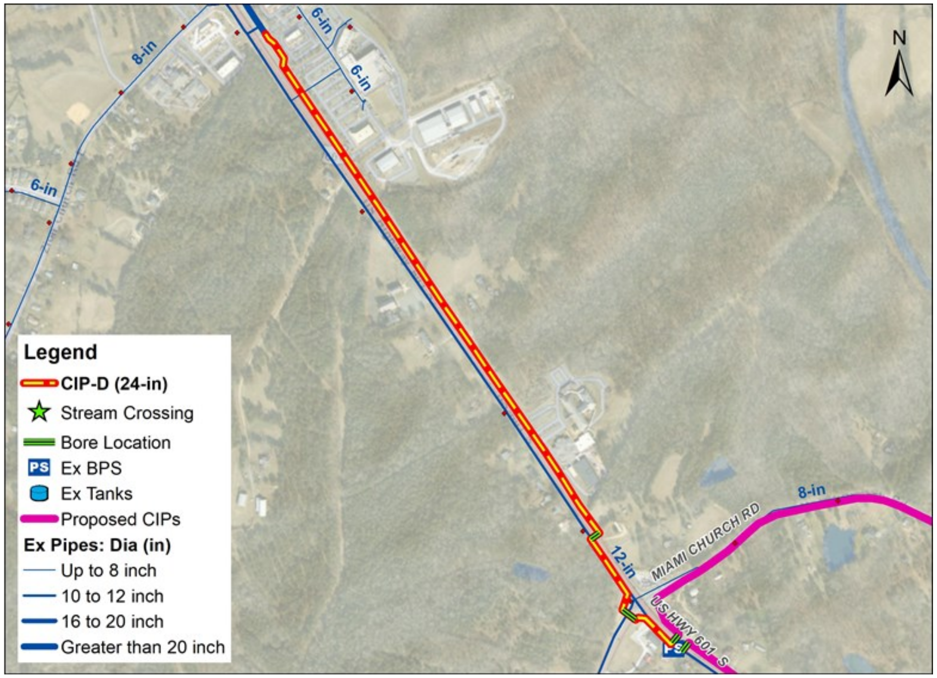


Figure 8-6: CIP–A - 24-in 4,400 LF along North US Hwy 601

PLANNING LEVEL COST ESTIMATE	B: 24-in 20,000 LF along US Hwy 601 (between Miami Church Rd and Parks Lafferty Rd)
Mobilization (3%)	$308,000
Piping	$7,821,000
Road & Stream Crossings	$915,000
Tie-ins/Hydrants/Services	$350,000
Erosion & Traffic Control, Restoration	$881,000
Construction Cost	**$10,275,000**
Rock Contingency (15%)	$1,541,000
Contingency (30%)	$3,083,000
Construction Subtotal	**$14,899,000**
Design @ 15%	$1,541,000
Limited Construction Admin @ 5%	$514,000
Engineering Subtotal	**$2,055,000**
TOTAL	**$16,954,000**

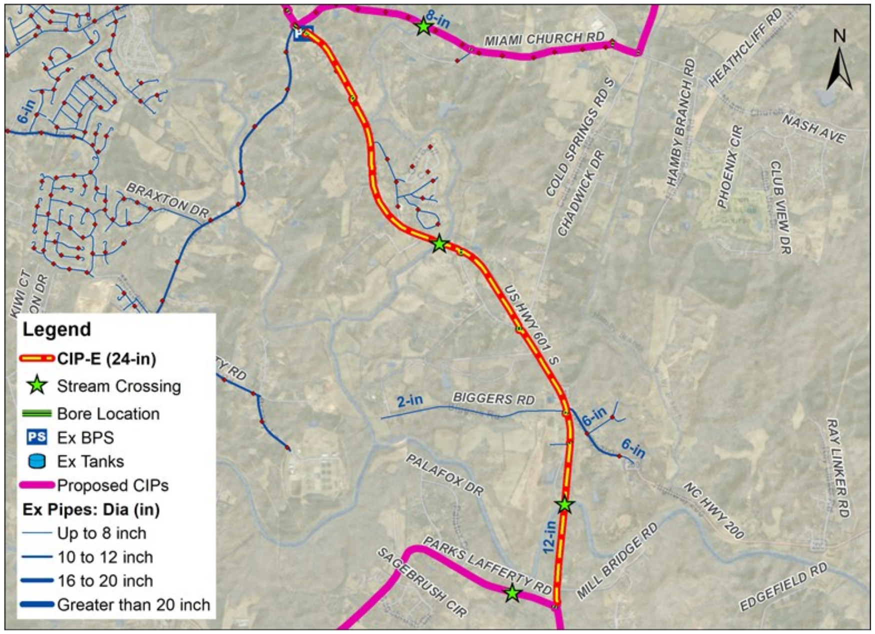


Figure 8-7: CIP–B - 24-in 20,000 LF along US Hwy 601 (between Miami Church Rd and Parks Lafferty Rd)

II. Phase 2: Improvements by 2030 (MDD demand: 31.0 mgd)

By 2030, CIP-E, F, G & H water lines are required in Pressure Zone 841 to meet design criteria, lower headloss and increase available fire flow until the new tank is built, shown in Figure 8-1. CIP-I can be constructed as development occurs in this area; however, under projected 2050 demands, this pipe must be 12-inch to meet design criteria.

Project sheets with cost estimates for Phase 2 improvements are shown in Figure 8-8 to Figure 8--14.

PLANNING LEVEL COST ESTIMATE	C: 12-in 3,350 LF along Christenbury Parkway
Mobilization (3%)	$19,000
Piping	$302,000
Road & Stream Crossings	$160,000
Tie-ins/Hydrants/Services	$55,000
Erosion & Traffic Control, Restoration	$87,000
Construction Cost	**$623,000**
Rock Contingency (15%)	$93,000
Contingency (30%)	$187,000
Construction Subtotal	**$903,000**
Design @ 15%	$93,000
Limited Construction Admin @ 5%	$31,000
Engineering Subtotal	**$124,000**
TOTAL	**$1,027,000**

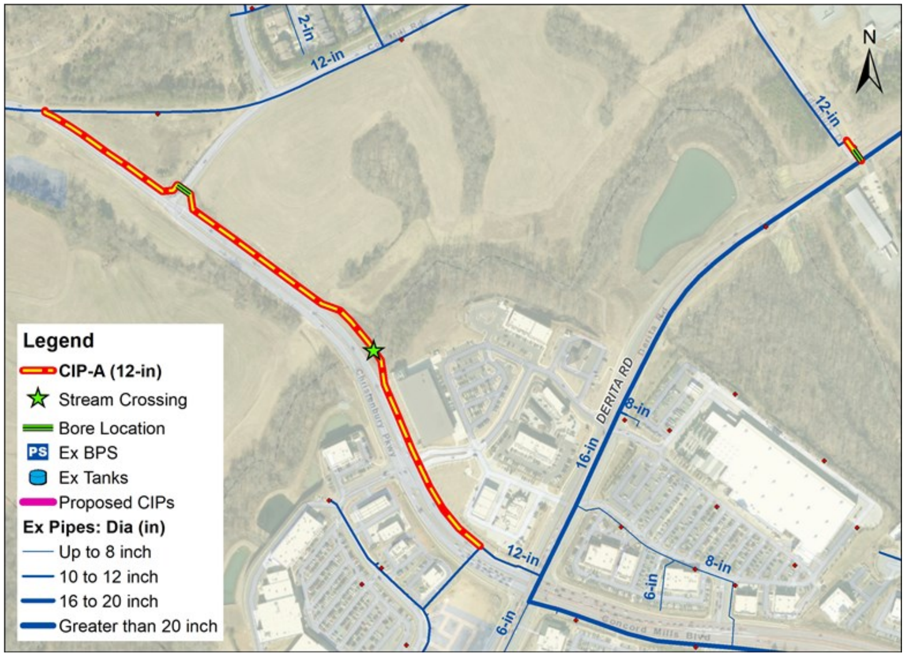


Figure 8-8: CIP–C - 12-in 3,350 LF along Christenbury Parkway

PLANNING LEVEL COST ESTIMATE	D: 12-in 2,050 LF along Odell School Rd
Mobilization (3%)	$10,000
Piping	$169,000
Road & Stream Crossings	$60,000
Tie-ins/Hydrants/Services	$50,000
Erosion & Traffic Control, Restoration	$54,000
Construction Cost	**$343,000**
Rock Contingency (15%)	$51,000
Contingency (30%)	$103,000
Construction Subtotal	**$497,000**
Design @ 15%	$51,000
Limited Construction Admin @ 5%	$17,000
Engineering Subtotal	**$68,000**
TOTAL	**$565,000**

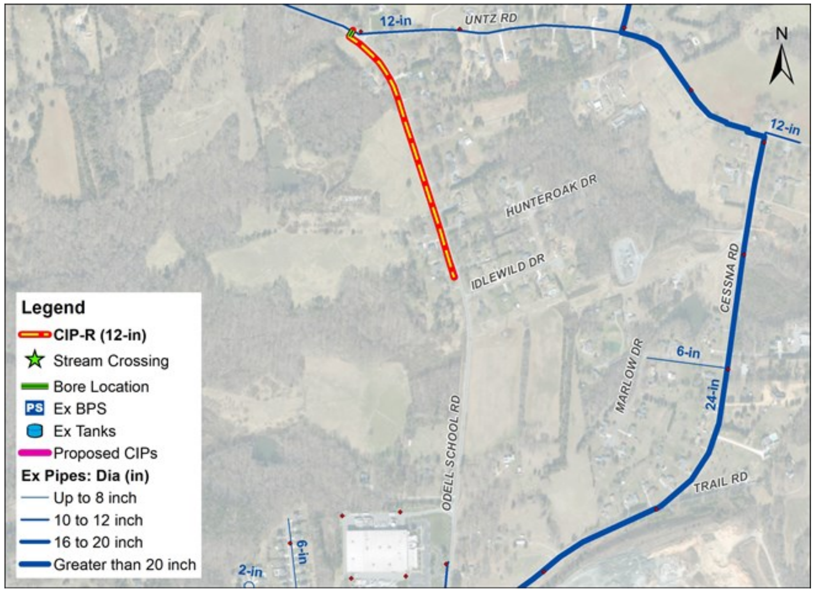


Figure 8-9: CIP–D - 12-in 2,050 LF along Odell School Rd

PLANNING LEVEL COST ESTIMATE	E: 24-in 5,200 LF along US Hwy 601 Cal Bost Rd
Mobilization (3%)	$47,000
Piping	$1,164,000
Road & Stream Crossings	$150,000
Tie-ins/Hydrants/Services	$55,000
Erosion & Traffic Control, Restoration	$138,000
Construction Cost	**$1,554,000**
Rock Contingency (15%)	$233,000
Contingency (30%)	$466,000
Construction Subtotal	**$2,253,000**
Design @ 15%	$233,000
Limited Construction Admin @ 5%	$78,000
Engineering Subtotal	**$311,000**
TOTAL	**$2,564,000**

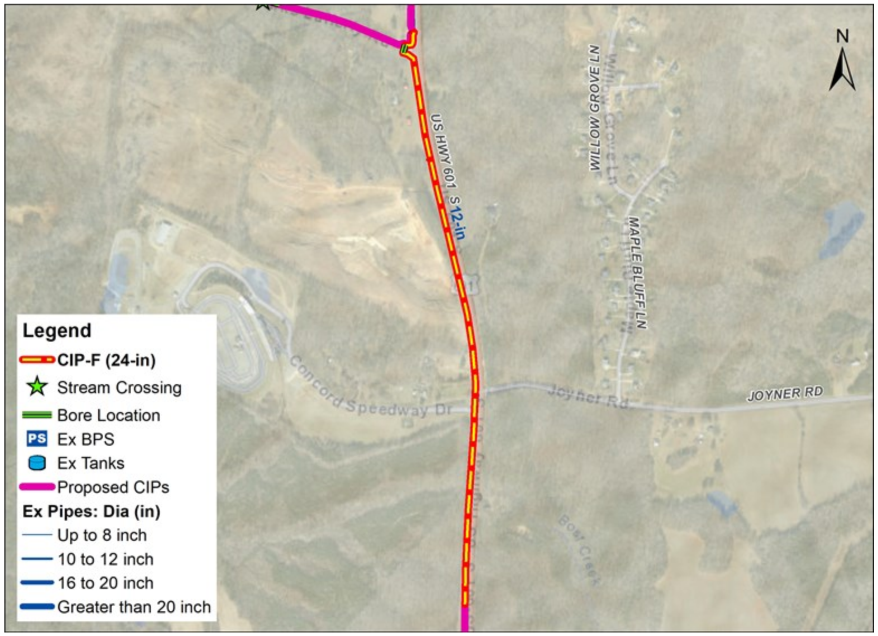


Figure 8-10: CIP–E - 24-in 5,200 LF along US Hwy 601 Cal Bost Rd

PLANNING LEVEL COST ESTIMATE	F: 24-in 1,300 LF along US Hwy 601
Mobilization (3%)	$15,000
Piping	$278,000
Road & Stream Crossings	$150,000
Tie-ins/Hydrants/Services	$25,000
Erosion & Traffic Control, Restoration	$33,000
Construction Cost	**$501,000**
Rock Contingency (15%)	$75,000
Contingency (30%)	$150,000
Construction Subtotal	**$726,000**
Design @ 15%	$75,000
Limited Construction Admin @ 5%	$25,000
Engineering Subtotal	**$100,000**
TOTAL	**$826,000**

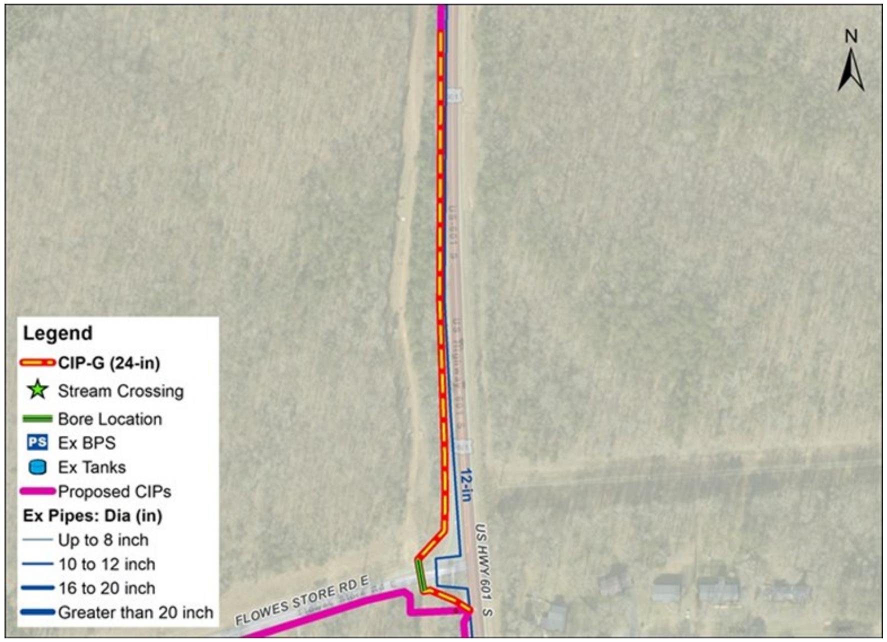


Figure 8-11: CIP–F - 24-in 1,300 LF along US Hwy 601

PLANNING LEVEL COST ESTIMATE	G : 16-in 5,800 LF along US Hwy 601
Mobilization (3%)	$35,000
Piping	$793,000
Road & Stream Crossings	$94,000
Tie-ins/Hydrants/Services	$85,000
Erosion & Traffic Control, Restoration	$155,000
Construction Cost	**$1,162,000**
Rock Contingency (15%)	$174,000
Contingency (30%)	$349,000
Construction Subtotal	**$1,685,000**
Design @ 15%	$174,000
Limited Construction Admin @ 5%	$58,000
Engineering Subtotal	**$232,000**
TOTAL	**$1,917,000**

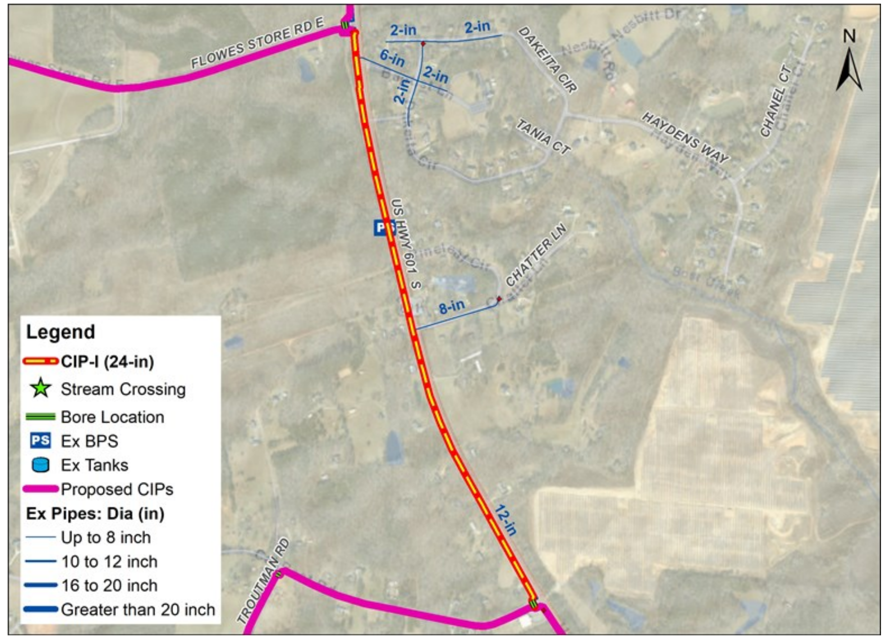


Figure 8-12: CIP–G - 16-in 5,800 LF along US Hwy 601

PLANNING LEVEL COST ESTIMATE	H: 12-in 1,500 LF along US Hwy 601
Mobilization (3%)	$28,000
Piping	$132,000
Road & Stream Crossings	$725,000
Tie-ins/Hydrants/Services	$35,000
Erosion & Traffic Control, Restoration	$21,000
Construction Cost	**$941,000**
Rock Contingency (15%)	$141,000
Contingency (30%)	$282,000
Construction Subtotal	**$1,364,000**
Design @ 15%	$141,000
Limited Construction Admin @ 5%	$47,000
Engineering Subtotal	**$188,000**
TOTAL	**$1,552,000**

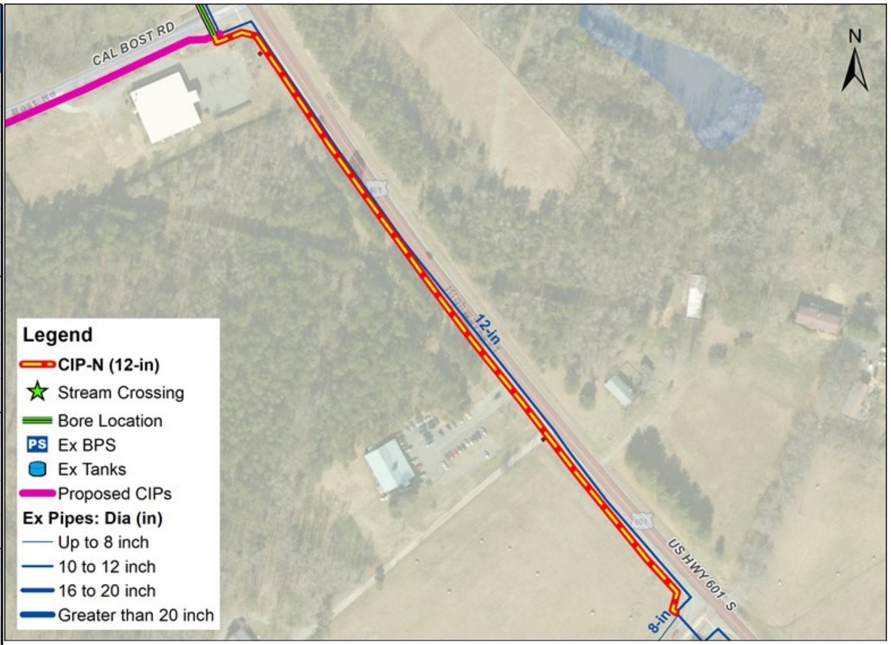


Figure 8-13: CIP–H - 12-in 1,500 LF along US Hwy 601

PLANNING LEVEL COST ESTIMATE	I: 12-in 9,500 LF along Bethel Church Rd to NC 24-27
Mobilization (3%)	$54,000
Piping	$924,000
Road & Stream Crossings	$500,000
Tie-ins/Hydrants/Services	$90,000
Erosion & Traffic Control, Restoration	$243,000
Construction Cost	**$1,811,000**
Rock Contingency (15%)	$272,000
Contingency (30%)	$543,000
Construction Subtotal	**$2,626,000**
Design @ 15%	$272,000
Limited Construction Admin @ 5%	$91,000
Engineering Subtotal	**$363,000**
TOTAL	**$2,989,000**

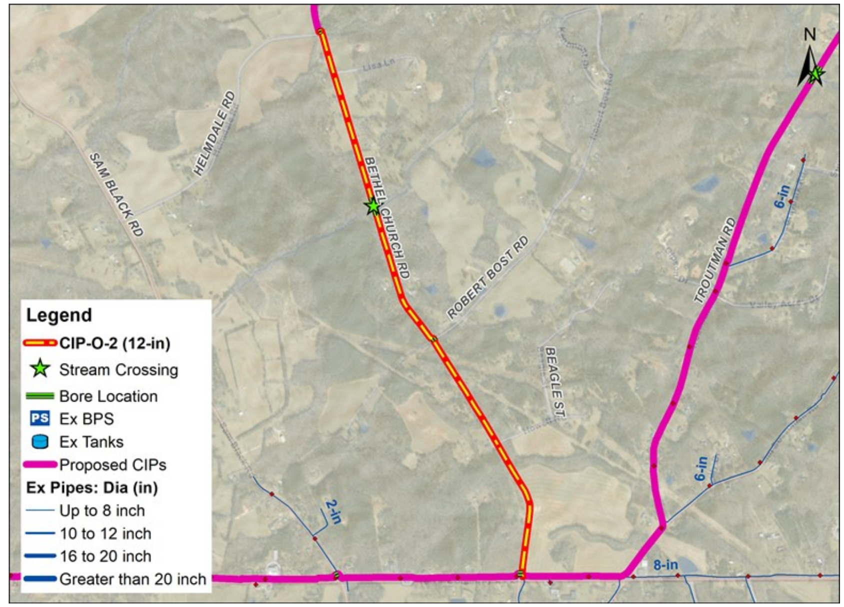


Figure 8--14: CIP–I - 12-in 9,500 LF along Bethel Church Rd to NC 24-27

III. Phase 3: Improvements by 2035 (MDD demand: 33.7 mgd)

By 2035, we recommended a 4 mgd Booster Pump Station (BPS) (**CIP-B**) from 850 Pressure Zone to 980 Pressure Zone. This station will meet demands in both Pressures Zones 890 and 841. This requires a 24-inch pipe along Future George Lilies Pkwy (**CIP-J**) to meet design criteria for headloss and pressure.

To address the equalizing and fire storage shortage in the 841 Pressure Zone, we recommend a **1 MG tank** and corresponding 16-inch water line (**CIP-K)** for hydraulic balance in the zone. Figure 8-1 shows all the improvements by 2035.

To meet 2035 MDD, all 3 of the larger pumps at Coddle Creek WTP are required. We recommend that at least one of the smaller pumps be replaced with a fourth large pump on a VFD to accommodate a larger ranger of demands.

Project sheets with cost estimates for Phase 3 improvements are shown in Figure 8-15 to Figure 8-19.

PLANNING LEVEL COST ESTIMATE	J: 24-in 9,750 LF along Stough Rd (Direct Replace)
Mobilization (3%)	$150,000
Piping	$3,788,000
Road & Stream Crossings	$440,000
Tie-ins/Hydrants/Services	$195,000
Erosion & Traffic Control, Restoration	$430,000
Construction Cost	**$5,003,000**
Rock Contingency (15%)	$750,000
Contingency (30%)	$1,501,000
Construction Subtotal	**$7,254,000**
Design @ 15%	$750,000
Limited Construction Admin @ 5%	$250,000
Engineering Subtotal	**$1,000,000**
TOTAL	**$8,254,000**

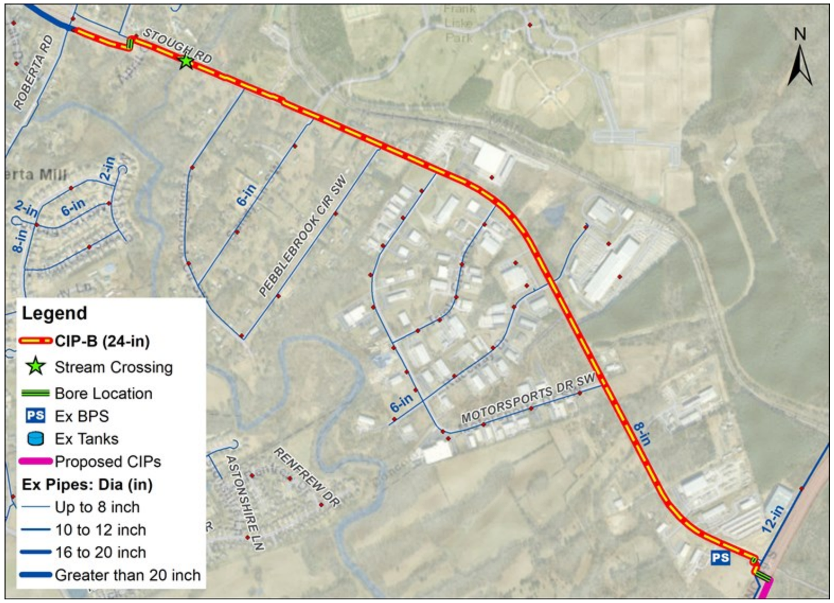


Figure 8-15: CIP–J - 24-in 9,750 LF along Stough Rd (Direct Replace)

PLANNING LEVEL COST ESTIMATE	NC HWY 49 BPS
Mobilization (3%)	$49,000
Piping	$23,000
Property Acquisition	$50,000
Erosion & Traffic Control, Restoration	$5,000
Pump Station (with VFDs)	$1,501,000
Construction Cost	**$1,628,000**
Contingency (30%)	$488,000
Construction Subtotal	**$2,116,000**
Design @ 15%	$245,000
Limited Construction Admin @ 5%	$82,000
Engineering Subtotal	**$327,000**
TOTAL	**$2,443,000**

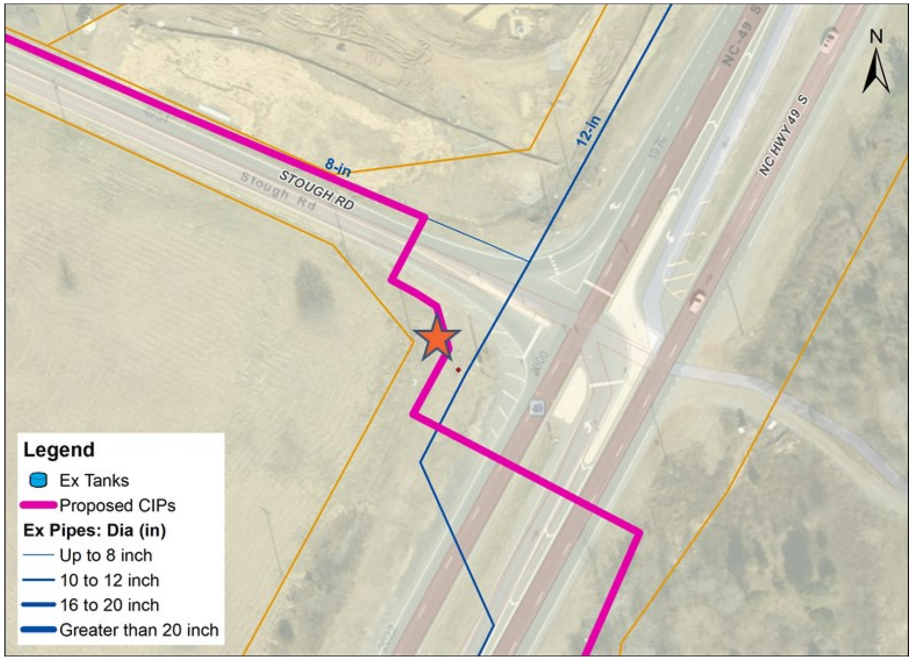
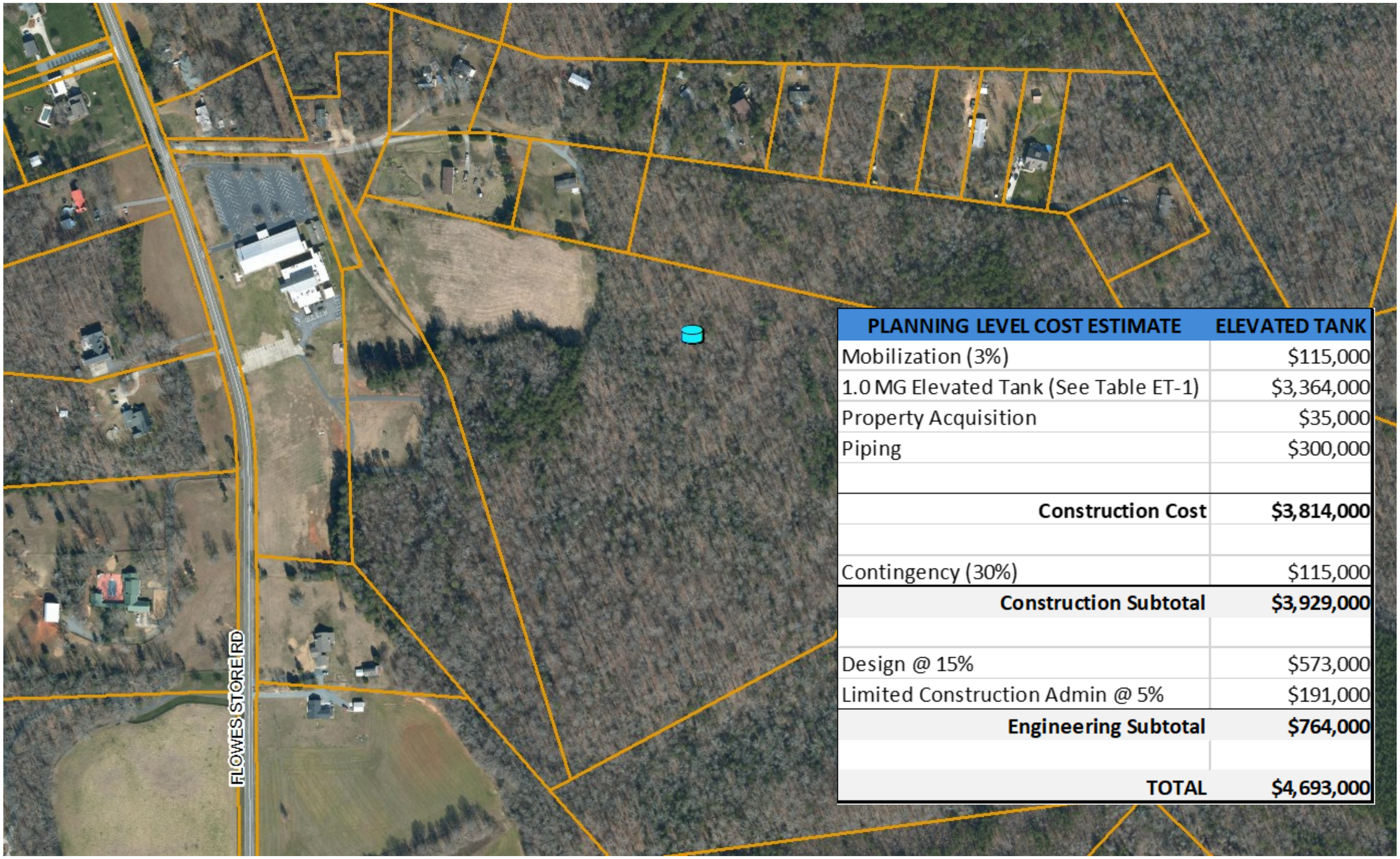


Figure 8-16: Pump Station 2 - At the Intersection of Stough Road and NC Hwy 49



PLANNING LEVEL COST ESTIMATE	ELEVATED TANK
Mobilization (3%)	$115,000
1.0 MG Elevated Tank (See Table ET-1)	$3,364,000
Property Acquisition	$35,000
Piping	$300,000
Construction Cost	**$3,814,000**
Contingency (30%)	$115,000
Construction Subtotal	**$3,929,000**
Design @ 15%	$573,000
Limited Construction Admin @ 5%	$191,000
Engineering Subtotal	**$764,000**
TOTAL	**$4,693,000**

Figure 8-17: CIP Elevated Tank in 841 Pressure Zone

PLANNING LEVEL COST ESTIMATE	K: 16-in 17,100 LF along Troutman Rd between US Hwy 601 and NC Hwy 24-27
Mobilization (3%)	$105,000
Piping	$2,430,000
Road & Stream Crossings	$363,000
Tie-ins/Hydrants/Services	$160,000
Erosion & Traffic Control, Restoration	$452,000
Construction Cost	**$3,510,000**
Rock Contingency (15%)	$527,000
Contingency (30%)	$1,053,000
Construction Subtotal	**$5,090,000**
Design @ 15%	$527,000
Limited Construction Admin @ 5%	$176,000
Engineering Subtotal	**$703,000**
TOTAL	**$5,793,000**

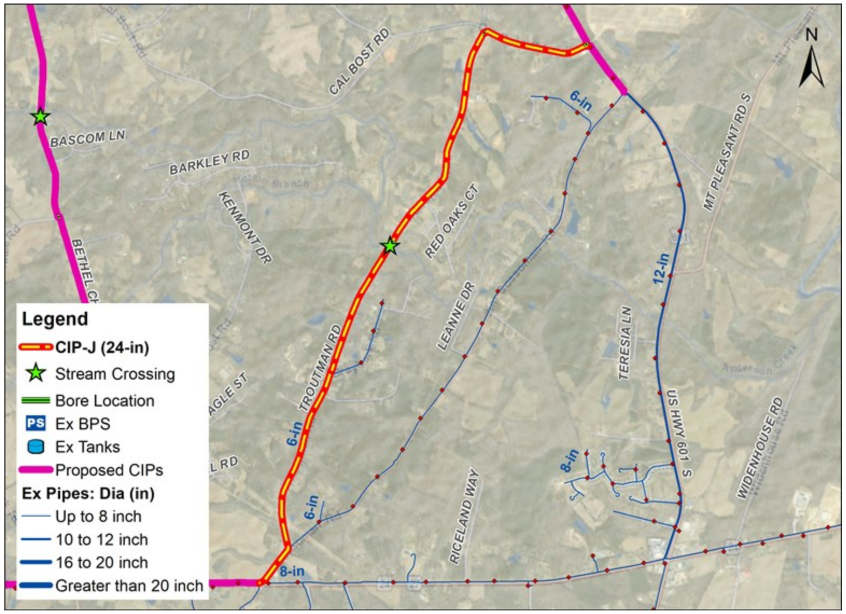


Figure 8-18: CIP–K - 16-in 17,100 LF along Troutman Rd between US Hwy 601 and NC Hwy 24-27

PLANNING LEVEL COST ESTIMATE	L: 16-in 11,550 LF along NC Hwy 24-27
Mobilization (3%)	$69,000
Piping	$1,634,000
Road & Stream Crossings	$169,000
Tie-ins/Hydrants/Services	$115,000
Erosion & Traffic Control, Restoration	$308,000
Construction Cost	**$2,295,000**
Rock Contingency (15%)	$344,000
Contingency (30%)	$689,000
Construction Subtotal	**$3,328,000**
Design @ 15%	$344,000
Limited Construction Admin @ 5%	$115,000
Engineering Subtotal	**$459,000**
TOTAL	**$3,787,000**

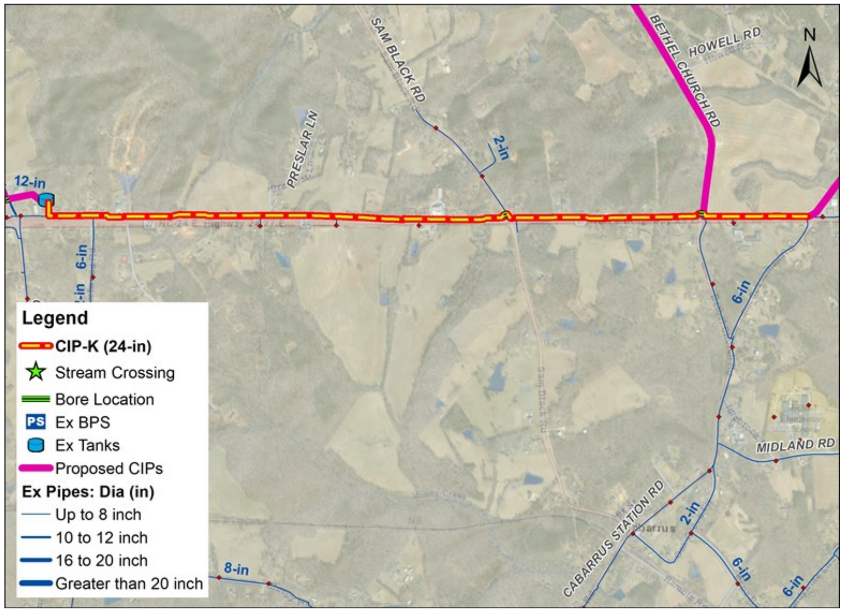


Figure 8-19: CIP–L - 16-in 11,550 LF along NC Hwy 24-27

IV. Phase 4: Improvements by 2040 (MDD demand: 36.3 mgd)

2040 improvements are shown in Figure 8-1, the Rocky River BPS will be needed to maintain Harrisburg Tank level and Midland Tank levels.

Project sheets with cost estimates for Phase 4 improvements are shown in Figure 8-20 to Figure 8-25.

PLANNING LEVEL COST ESTIMATE	M: 12-in 1,600 LF between Odell School Rd and Moss Plantation Ave
Mobilization (3%)	$9,000
Piping	$136,000
Road & Stream Crossings	$65,000
Tie-ins/Hydrants/Services	$35,000
Erosion & Traffic Control, Restoration	$41,000
Construction Cost	**$286,000**
Rock Contingency (15%)	$43,000
Contingency (30%)	$86,000
Construction Subtotal	**$415,000**
Design @ 15%	$43,000
Limited Construction Admin @ 5%	$14,000
Engineering Subtotal	**$57,000**
TOTAL	**$472,000**

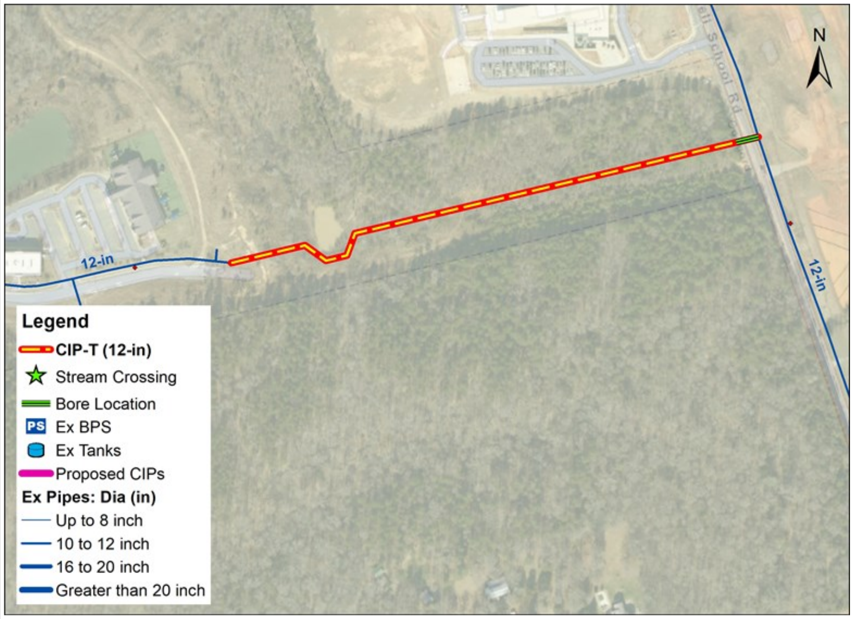


Figure 8-20: CIP–M - 12-in 1,600 LF between Odell School Rd and Moss Plantation Ave

PLANNING LEVEL COST ESTIMATE	N: 12-in 600 LF at Coddle Creek WTP
Mobilization (3%)	$2,000
Piping	$25,000
Road & Stream Crossings	$0
Tie-ins/Hydrants/Services	$40,000
Erosion & Traffic Control, Restoration	$16,000
Construction Cost	**$83,000**
Rock Contingency (15%)	$12,000
Contingency (30%)	$25,000
Construction Subtotal	**$120,000**
Design @ 15%	$12,000
Limited Construction Admin @ 5%	$4,000
Engineering Subtotal	**$16,000**
TOTAL	**$136,000**

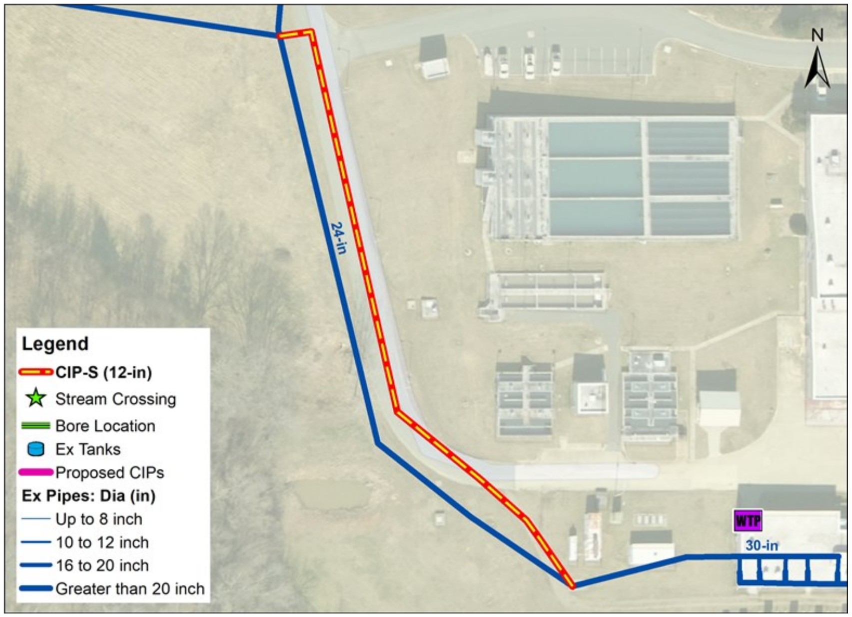


Figure 8-21: CIP–N - 12-in 600 LF at Coddle Creek WTP

PLANNING LEVEL COST ESTIMATE	O: 16-in 8,200 LF cross-country between Stough Rd and Rocky River Rd
Mobilization (3%)	$78,000
Piping	$1,136,000
Road & Stream Crossings	$1,102,000
Tie-ins/Hydrants/Services	$105,000
Erosion & Traffic Control, Restoration	$190,000
Construction Cost	**$2,611,000**
Rock Contingency (15%)	$392,000
Contingency (30%)	$783,000
Construction Subtotal	**$3,786,000**
Design @ 15%	$392,000
Limited Construction Admin @ 5%	$131,000
Engineering Subtotal	**$523,000**
TOTAL	**$4,309,000**

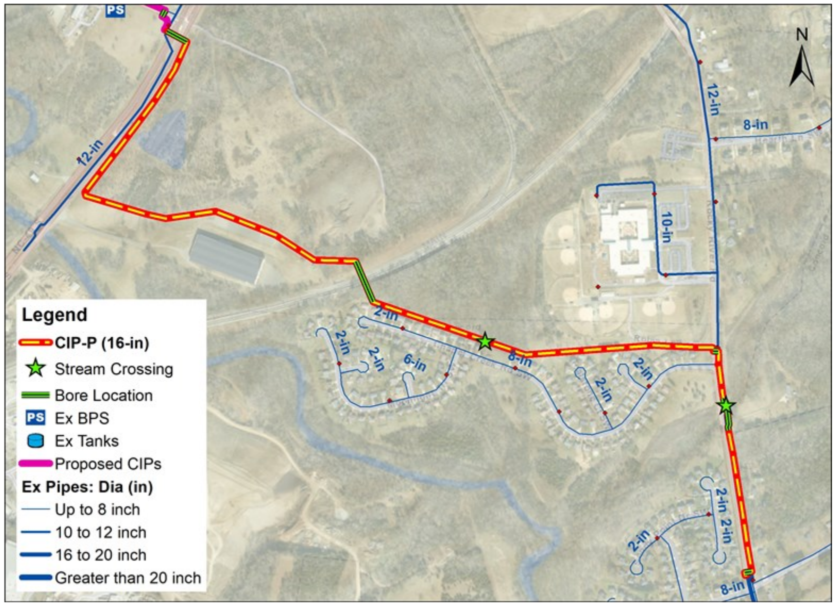


Figure 8-22: CIP–O - 16-in 8,200 LF cross-country between Stough Rd and Rocky River Rd

PLANNING LEVEL COST ESTIMATE	P: 12-in 26,250 LF along Cold Springs Rd between NC Hwy 49 and US Hwy 601
Mobilization (3%)	$131,000
Piping	$2,543,000
Road & Stream Crossings	$730,000
Tie-ins/Hydrants/Services	$260,000
Erosion & Traffic Control, Restoration	$689,000
Construction Cost	**$4,353,000**
Rock Contingency (15%)	$653,000
Contingency (30%)	$1,306,000
Construction Subtotal	**$6,312,000**
Design @ 15%	$653,000
Limited Construction Admin @ 5%	$218,000
Engineering Subtotal	**$871,000**
TOTAL	**$7,183,000**

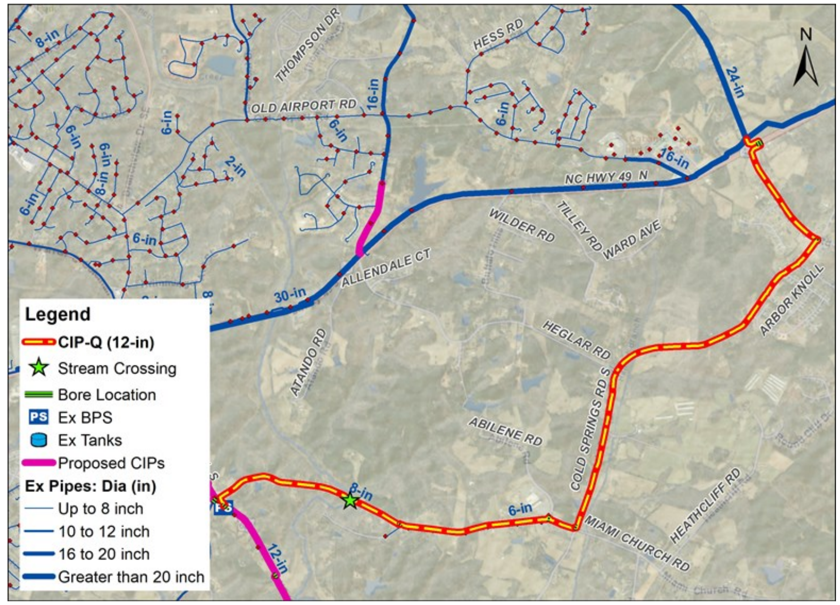


Figure 8-23: CIP–P - 12-in 26,250 LF along Cold Springs Rd between NC Hwy 49 and US Hwy 601

PLANNING LEVEL COST ESTIMATE	Q: 12-in 37,500 LF along Parks Lafferty Rd and Flowes Store Rd between US Hwy 601 and NC Hwy 24-27
Mobilization (3%)	$184,000
Piping	$3,873,000
Road & Stream Crossings	$725,000
Tie-ins/Hydrants/Services	$350,000
Erosion & Traffic Control, Restoration	$993,000
Construction Cost	**$6,125,000**
Rock Contingency (15%)	$919,000
Contingency (30%)	$1,838,000
Construction Subtotal	**$8,882,000**
Design @ 15%	$919,000
Limited Construction Admin @ 5%	$306,000
Engineering Subtotal	**$1,225,000**
TOTAL	**$10,107,000**

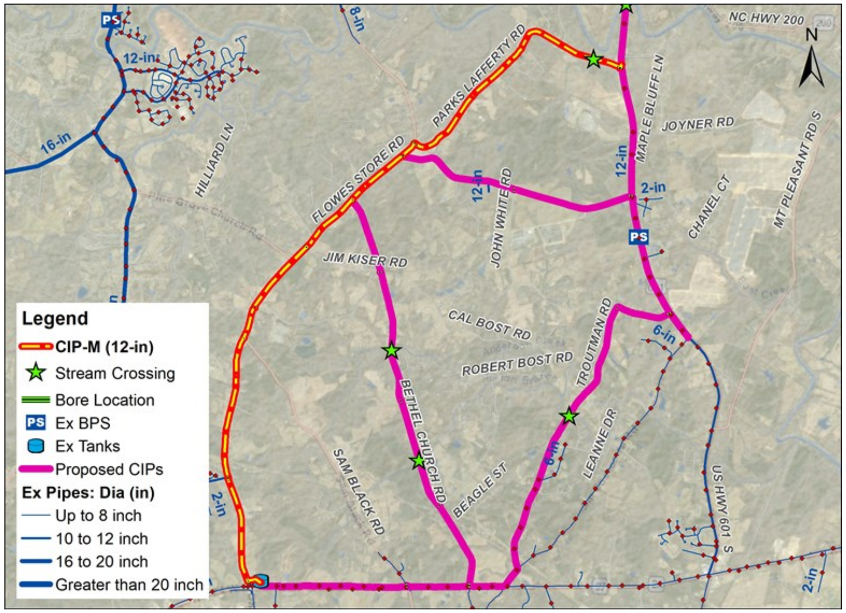


Figure 8-24: CIP–Q - 12-in 37,500 LF along Parks Lafferty Rd and Flowes Store Rd between US Hwy 601 and NC Hwy 24-27

PLANNING LEVEL COST ESTIMATE	R: 12-in 7,050 LF along Flowes Store Rd
Mobilization (3%)	$31,000
Piping	$683,000
Road & Stream Crossings	$50,000
Tie-ins/Hydrants/Services	$70,000
Erosion & Traffic Control, Restoration	$189,000
Construction Cost	**$1,023,000**
Rock Contingency (15%)	$153,000
Contingency (30%)	$307,000
Construction Subtotal	**$1,483,000**
Design @ 15%	$153,000
Limited Construction Admin @ 5%	$51,000
Engineering Subtotal	**$204,000**
TOTAL	**$1,687,000**

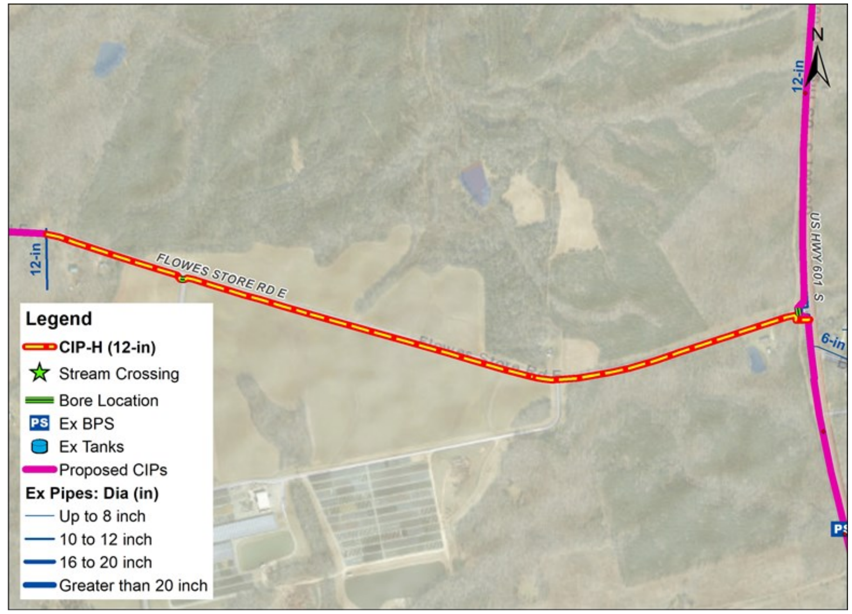


Figure 8-25: CIP–R - 12-in 7,050 LF along Flowes Store Rd

V. Phase 5: Improvements by 2045 (MDD demand: 38.9 mgd)

Figure 8-1 shows the phase of improvements by 2045. These projects complete can be constructed as development occurs and provides the backbone infrastructure to balance hydraulics and meet fire flows.

Coddle Creek WTP pumps capacity will need to be increased to meet maximum days. We recommend replacing the second smaller pumps with similar or larger capacity than the existing large pumps and operate them on VFDs to accommodate a larger ranger of demands.

Project sheets with cost estimates for Phase 5 improvements are shown in Figure 8-26 and Figure 8-27.

PLANNING LEVEL COST ESTIMATE	S: 12-in 4,500 LF along Flowes Store Rd
Mobilization (3%)	$19,000
Piping	$432,000
Road & Stream Crossings	$0
Tie-ins/Hydrants/Services	$48,000
Erosion & Traffic Control, Restoration	$122,000
Construction Cost	**$621,000**
Rock Contingency (15%)	$93,000
Contingency (30%)	$186,000
Construction Subtotal	**$900,000**
Design @ 15%	$93,000
Limited Construction Admin @ 5%	$31,000
Engineering Subtotal	**$124,000**
TOTAL	**$1,024,000**

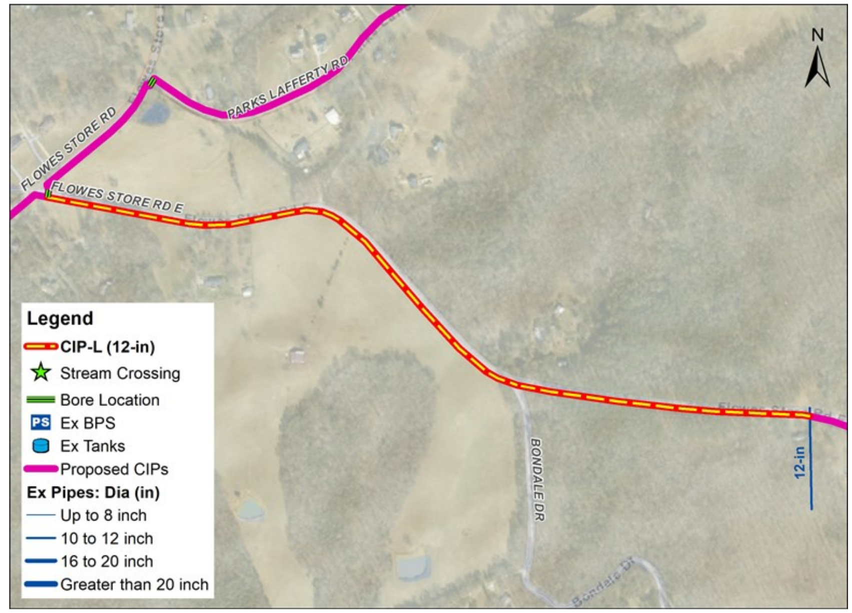


Figure 8-26: CIP–S - 12-in 4,500 LF along Flowes Store Rd

PLANNING LEVEL COST ESTIMATE	T: 12-in 9,900 LF along Bethel Church Rd to Helmdale Rd
Mobilization (3%)	$56,000
Piping	$972,000
Road & Stream Crossings	$500,000
Tie-ins/Hydrants/Services	$85,000
Erosion & Traffic Control, Restoration	$254,000
Construction Cost	**$1,867,000**
Rock Contingency (15%)	$280,000
Contingency (30%)	$560,000
Construction Subtotal	**$2,707,000**
Design @ 15%	$280,000
Limited Construction Admin @ 5%	$93,000
Engineering Subtotal	**$373,000**
TOTAL	**$3,080,000**

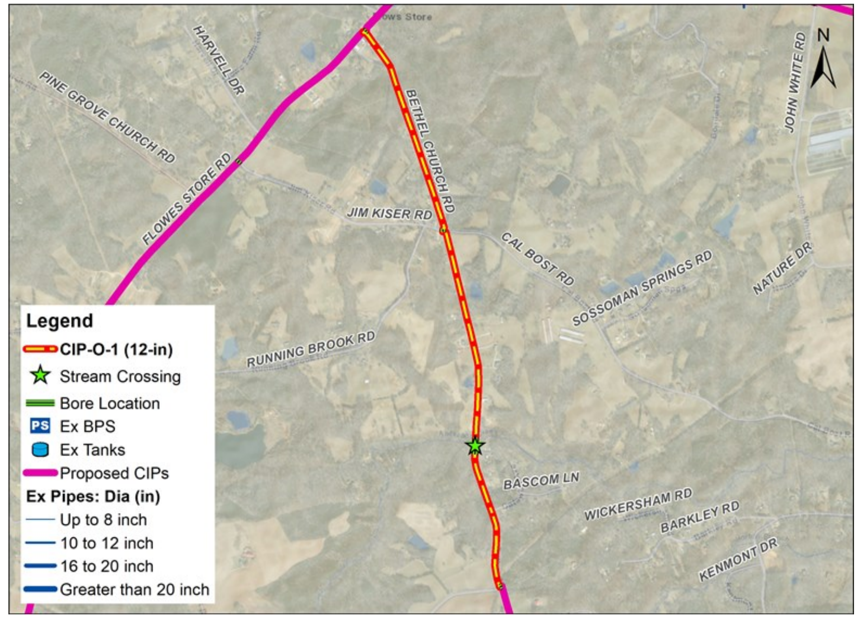


Figure 8-27: CIP-T - 12-in 9,900 LF along Bethel Church Rd to Helmdale Rd

Appendix A: Pump Capacity Evaluation

Existing System	2020-2025					2025				2030				2035				2040				2045			
	831	841	850	890E	890W	831	841	850	890	831	841	850	890	831	841	850	890	831	841	850	890	831	841	850	890
Pump Capacity																									
Maximum Day - MGD	3.98	2.67	10.84	1.60	1.62	4.22	1.73	11.12	9.54	4.64	2.20	12.25	11.91	5.05	2.79	13.13	12.71	5.47	3.26	14.03	13.51	5.89	3.74	14.93	14.30
Supply to Other Zones	1.62	0.00	1.52	1.20	0.00	5.80	0.00	5.00	2.20	5.80	0.00	9.00	2.79	5.80	0.00	9.00	3.26	5.80	0.00	9.00	3.74	4.60	0.00	9.00	4.22
Total Pumping Requirement	5.60	2.67	12.36	2.80	1.62	10.44	2.20	17.25	14.11	10.85	2.79	22.13	15.49	11.27	3.26	23.03	16.77	11.69	3.74	23.03	18.04	10.90	4.22	24.83	19.21
Existing Firm Pump Capacity - MGD																									
Hillgrove WTP	6.00					12.60				12.60				12.60				12.60				12.60			
Coddle Creek WTP			12.00					13.20				13.20				13.20				13.20				16.00	
Hwy 601 PS Abandoned		0.70																							
Rocky River PS (VFD)		2.80					2.7				2.7				2.7				2.7				2.7		
Highway 73 PS (Emergency)									1.20				1.20				1.20				1.20				1.20
Rockhill Church Road PS																									
Mt Pleasant PS (VFD)				4.00					7.00				7.00				7.00				7.00				7.00
Corban PS (VFD)					1.62				4.60				4.60				4.60				4.60				4.60
Catawba Supply			4.00					4.00				7.00				9.00				10.00				10.00	
Total Pump Capacity - MGD	6.00	3.50	16.00	4.00	1.62	12.60	2.70	20.20	12.80	12.60	2.70	21.20	12.80	12.60	2.70	21.20	12.80	12.60	2.70	23.20	12.80	15.20	2.80	26.00	11.60
Pump Capacity Surplus/ Deficit	0.40	0.83	3.64	1.20	0.00	2.58	0.97	1.08	1.53	2.16	0.50	2.95	-1.31	1.75	-0.09	-0.93	-2.69	1.33	-0.56	0.17	-3.97	0.91	-1.04	2.07	-5.24
New Pumps at Existing PS (850 into 890)									5.00				5.00				5.00				5.00				5.00
New HWY 49 PS (850 into 890)																	4.00				4.00				4.00
New PRV/FCV (890 into 841)							1.00				1.33				2.76				3.05				3.53		
Total Pump Capacity - MGD	6.00	3.50	16.00	4.00	1.62	12.60	3.70	17.20	17.80	12.60	4.03	20.20	17.80	12.60	5.46	22.20	21.80	12.60	5.75	23.20	21.80	12.60	6.23	26.00	21.80
Pump Capacity Surplus/Deficit	0.40	0.83	3.64	1.20	0.00	2.58	1.97	1.08	6.53	2.16	1.82	2.95	3.69	1.75	2.68	0.07	6.31	1.33	2.48	0.17	5.03	0.91	2.49	2.07	3.76

Appendix B: Storage Evaluation

Existing System	2020				2025				2030				2035				2040				2045			
	831	841	850	890	831	841	850	890	831	841	850	890	831	841	850	890	831	841	850	890	831	841	850	890
EQUALIZING STORAGE																								
AVERAGE DAY DEMAND - MGD	2.83	0.95	4.03	3.01	3.04	1.35	4.84	5.12	3.34	1.69	5.65	5.69	3.64	2.03	6.29	6.26	3.94	2.37	6.93	6.83	4.24	2.71	7.57	7.40
MAXIMUM DAY DEMAND - MGD	3.91	1.28	5.63	5.74	4.20	1.84	6.77	8.70	4.62	2.31	7.90	9.50	5.04	2.79	8.80	10.29	5.46	3.26	9.69	11.09	5.88	3.74	10.59	11.89
EQUALIZING PERCENTAGE	13%	16%	14%	20%	13%	16%	14%	20%	13%	16%	14%	20%	13%	16%	14%	20%	13%	16%	14%	20%	13%	16%	14%	20%
EQUALIZING VOLUME -MG	0.51	0.20	0.79	1.15	0.55	0.29	0.95	1.74	0.60	0.37	1.11	1.90	0.66	0.45	1.23	2.06	0.71	0.52	1.36	2.22	0.76	0.60	1.48	2.38
FIRE STORAGE																								
NEEDED FIRE FLOW - GPM	3000	2500	3000	3000	3000	2500	3000	3000	3000	2500	3000	3000	3000	2500	3000	3000	3000	2500	3000	3000	3000	2500	3000	3000
DURATION - HOURS	3	2	3	3	3	2	3	3	3	2	3	3	3	2	3	3	3	2	3	3	3	2	3	3
FIRE STORAGE VOLUME - MG	0.54	0.30	0.54	0.54	0.54	0.30	0.54	0.54	0.54	0.30	0.54	0.54	0.54	0.30	0.54	0.54	0.54	0.30	0.54	0.54	0.54	0.30	0.54	0.54
EQUALIZING + FIRE STORAGE VOLUME - MG	1.05	0.50	1.33	1.69	1.09	0.59	1.49	2.28	1.14	0.67	1.65	2.44	1.20	0.75	1.77	2.60	1.25	0.82	1.90	2.76	1.30	0.90	2.02	2.92
EXISTING STORAGE																								
CLEARWELL - HILLGROVE WTP	4.00				4.00				4.00				4.00				4.00				4.00			
CLEARWELL - CODDLE CREEK WTP			4.00				4.00				4.00				4.00				4.00				4.00	
MIDLAND		0.50				0.50				0.50				0.50				0.50				0.50		
SPEEDWAY			1.00				1.00				1.00				1.00				1.00				1.00	
ROCKHILL CHURCH HILL			2.00				2.00				2.00				2.00				2.00				2.00	
DOWNTWON	0.50				0.50				0.50				0.50				0.50				0.50			
TODD NEWELL	1.00				1.00				1.00				1.00				1.00				1.00			
HWY 73				0.75				0.75				0.75				0.75				0.75				0.75
HWY 29								2.00				2.00				2.00				2.00				2.00
NEW MIDLAND														1.00				1.00				1.00		
TOTAL SYSTEM STORAGE - MG	1.50	0.50	3.00	0.75	1.50	0.50	3.00	2.75	1.50	0.50	3.00	2.75	1.50	1.50	3.00	2.75	1.50	1.50	3.00	2.75	1.50	1.50	3.00	2.75
EQUALIZING & FIRE STORAGE - MG																								
Existing	1.50	0.50	3.00	0.75	1.50	0.50	3.00	2.75	1.50	0.50	3.00	2.75	1.50	1.50	3.00	2.75	1.50	1.50	3.00	2.75	1.50	1.50	3.00	2.75
Surplus/Deficit	0.45	0.00	1.67	-0.94	0.41	-0.09	1.51	0.47	0.36	-0.17	1.35	0.31	0.30	0.75	1.23	0.15	0.25	0.68	1.10	-0.01	0.20	0.60	0.98	-0.17
EMERGENCY STORAGE (1/2 ADD) -MG																								
Existing	13.75				15.75				15.75				16.75				16.75				16.75			
Surplus/Deficit	8.34				8.58				7.57				7.64				6.72				5.79			

Appendix C: CIP

CIP Priority	CIP Year	Reason for Improvement	Improvement ID Number	Improvement	Pressure Zone	Location / Description	Rural/ Urban	Size	Length (LF)	Estimated Construction Cost	Related Improvement	Driver for Improvement
1	2025	Capacity	PS 1	Pump Station Upgrade	890	along Rock Hill Church Rd at the existing PS site	-	-	-	$ 3,286,000		Industrial Demand
1	2025	Capacity	FC Vault	FC Vault	890	near the intersection of Flowes Store Rd, Miami Church Rd, and US HWY 601	-	-	-	$ 300,000	FC Vault, B	System connectivity, vulnerability, and reliability
1	2025	Capacity	A	Transmission Main	890	24-inch 4400 lf along North US HWY 601	Urban	24-inch	4400	$ 3,960,000		Improve hydraulic capacity and solve demand issues
1	2025	Capacity	B	Transmission Main	841	24-inch 20000 lf along US HWY 601(between Miami Church Rd and Parks Lafferty Rd)	Urban	24-inch	20000	$ 16,954,000	FC Vault, B	Improve hydraulic capacity and solve demand issues
2	2030	Capacity	C	Transmission Main	850	12-inch 3350 lf along Christenbury Parkway	Rural	12-inch	3350	$ 1,027,000		Improve hydraulic capacity and system connectivity
2	2030	Capacity	D	Transmission Main	850	12-inch 2050 lf along Odell School Rd	Rural	12-inch	2050	$ 565,000		Improve hydraulic capacity and system connectivity
2	2030	Capacity	E	Transmission Main	841	24-inch 5200 lf along US HWY 601 Cal Bost Rd	Rural	24-inch	5200	$ 2,564,000	E, F, G, H	Improve hydraulic capacity and solve demand issues
2	2030	Capacity	F	Transmission Main	841	24-inch 1300 lf along US HWY 601	Rural	24-inch	1300	$ 826,000	E, F, G, H	Improve hydraulic capacity and solve demand issues
2	2030	Capacity	G	Transmission Main	841	16-inch 5800 lf along US HWY 601	Rural	16-inch	5800	$ 1,917,000	E, F, G, H	Improve hydraulic capacity and solve demand issues
2	2030	Capacity	H	Transmission Main	841	12-inch 1500 lf along US HWY 601	Rural	12-inch	1500	$ 1,552,000	E, F, G, H	Improve hydraulic capacity and solve demand issues
2	2030	Capacity	I	Transmission Main	841	12-inch 9500 lf along Bethel Church Rd to NC 24-27	Rural	12-inch	9500	$ 2,989,000	I, T	Improve hydraulic capacity and solve demand issues
3	2035	Capacity	J	Transmission Main	850	24-inch 9750 lf along Stough Rd (Direct Replace)	Urban	24-inch	9750	$ 8,254,000		Improve hydraulic capacity
3	2035	Capacity /reliability	PS 2	New Pump Station	850/890	at the intersection of Stough Rd and NC HWY 49	-	-	-	$ 2,443,000		Required equalizing and fire storage
3	2035	Storage	Elevated Tank	1 MG Tank	841	122-ft tank off Flowes Store Rd	-	-	-	$ 4,693,000		Provides System Tank balance; reliability

CIP Priority	CIP Year	Reason for Improvement	Improvement ID Number	Improvement	Pressure Zone	Location / Description	Rural/ Urban	Size	Length (LF)	Estimated Construction Cost	Related Improvement	Driver for Improvement
3	2035	Capacity	K	Transmission Main	841	16-inch 17100 lf along Troutman Rd between US HWY 601 and NC HWY 24-27	Rural	16-inch	17100	$ 5,793,000	K, L	Improve hydraulic capacity
3	2035	Capacity	L	Transmission Main	841	16-inch 11500 lf along NC HWY 24-27	Rural	16-inch	11500	$ 3,787,000	K, L	Improve hydraulic capacity
4	2040	Capacity	M	Transmission Main	850	12-inch 1600 lf between Odell School Rd and Moss Plantation Ave	Rural	12-inch	1600	$ 472,000		Improve hydraulic capacity and system connectivity
4	2040	Capacity	N	Transmission Main	850	12-inch 600 lf at Coddle Creek WTP	Rural	12-inch	600	$ 136,000		Improve hydraulic capacity
4	2040	Capacity	O	Transmission Main	890	16-inch 8200 lf cross-country between Stough Rd and Rocky River Td	Rural	16-inch	8200	$ 4,309,000		Improve hydraulic capacity
4	2040	Capacity	P	Transmission Main	890	12-inch 26250 lf along Cold Springs Rd between NC HWY 49 and US HWY 601	Rural	12-inch	26250	$ 7,183,000		Solve demand issues
4	2040	Capacity	Q	Transmission Main	841	12-inch 37500 lf along Parks Lafferty Rd and Flowes Store Rd between US HWY 601 and NC HWY 24-27	Rural	12-inch	37500	$ 10,107,000		Improve hydraulic capacity and solve demand issues
4	2040	Capacity	R	Transmission Main	841	12-inch 7050 lf along Flowes Store Rd	Rural	12-inch	7050	$ 1,687,000	R, S	Improve hydraulic capacity and solve demand issues
5	2045	Capacity	S	Transmission Main	841	12-inch 4500 lf along Flowes Store Rd	Rural	12-inch	4500	$ 1,024,000	R, S	Improve hydraulic capacity and solve demand issues
5	2045	Capacity	T	Transmission Main	841	12-inch 9900 lf along Bethel Church Rd to Helmdale Rd	Rural	12-inch	9900	$ 3,080,000	I, T	Improve hydraulic capacity and solve demand issues

Appendix D: Fifty-six (56) Smaller Recommendations

Priority	NFF Deficient by gpm	CIP Year	Diameter	Length	Location	Construction Cost ($)	Cost of Construction + Engineering ($)	Cost of Construction + Engineering + Contingencies ($)
1	>2000	2017	8	1000	North of Kannaplis Hwy on S Ridge Ave	$96,000	$110,400	$143,520
2	>2000	2017	6	360	South of Mcgill Ave NW and North of Buffalo Ave NW on Bill St NW	$25,920	$29,808	$38,750
2	1500-2000	2017	8	500	East of Bill St NW and southwest of Spring St NW on Buffalo Ave NW	$48,000	$55,200	$71,760
3	1500-2000	2017	8	1550	South of Poplar Tent Rd on Education Way NW	$148,800	$171,120	$222,456
4	1500-2000	2017	8	2150	North of Pitts School Rd NW on Plantation Rd NW	$206,400	$237,360	$308,568
5	1500-2000	2017	8	3100	Southwest of Church St N and north of Mcgill Ave NW on Douglas Ave NW, Freeze Ave NW, and ST. James St NW	$297,600	$342,240	$444,912
6	1500-2000	2017	8	480	South of Liske Ave NW and North of Cararrus Ave W on Warren C Coleman Blvd N	$46,080	$52,992	$68,890
7	1500-2000	2017	12	600	North of Bruton Smith Blvd and Concord Pkwy S crossing	$86,400	$99,360	$129,168
8	1500-2000	2017	6	1100	Connect Pipes on Prior Dr to Grace Ave across behind Building to Ramdin Ct	$79,200	$91,080	$118,404
9	1500-2000	2017	8	600	Connecting pipe on Cloister Ct to 8-inch at The Village shopping center	$57,600	$66,240	$86,112
10	1000-1500	2018	8	1150	South of Poplar Tent Rd on Ney Cline Rd	$110,400	$126,960	$165,048
11	1000-1500	2018	6	260	Connect 12-inch on Branchview Dr to 6-inch on Lee-Ann Dr	$18,720	$21,528	$27,986
12	1000-1500	2018	12	700	Southwest of Simpson Dr NW and northeast of Church St N on Mckimmon Ave NE	$100,800	$115,920	$150,696
13	1000-1500	2018	8	440	North of Wilmar St NW on Gibson Dr NW	$42,240	$48,576	$63,149
14	1000-1500	2018	12	2750		$396,000	$455,400	$592,020

Priority	NFF Deficient by gpm	CIP Year	Diameter	Length	Location	Construction Cost ($)	Cost of Construction + Engineering ($)	Cost of Construction + Engineering + Contingencies ($)
15.1	1000-1500	2018	6	20		$1,440	$1,656	$2,153
15.2	1000-1500	2019	8	10	Southwest of Lake Concord Rd NE and Martin St NE crossing (Connecting 12-inch to 8-inch)	$960	$1,104	$1,435
16	1000-1500	2018	8	1450	Davison Dr between Beechwood Ave NW & Concord Pkwy	$139,200	$160,080	$208,104
17	1000-1500	2018	8	2160	North of Concord PKWY N on Country Club Dr NE and Kingsport Dr NE	$207,360	$238,464	$310,003
18	1000-1500	2018	8	600	Southwest of Duval St NW on Buck Pl NW	$57,600	$66,240	$86,112
19	1000-1500	2018	8	340	Connect Cabarrus Ave W and Corban Ave SW near Corban PS	$32,640	$37,536	$48,797
20	1000-1500	2018	8	250	South of Brunting Ln SW and East of Wilshire Ct SW on Wilshire Ave SW	$24,000	$27,600	$35,880
21	500-1000	2019	8	1900	South of Poplar Tent Rd on Shelton Rd NW	$182,400	$209,760	$272,688
22	500-1000	2019	12	740	East of Harris Pl NW on Morton Ave NW	$106,560	$122,544	$159,307
23	500-1000	2019	12	1400	Southwest of Church St N and Northeast of Forest St NW on Spencer Ave NW	$201,600	$231,840	$301,392
24	500-1000	2019	6	90	Southwest of Faith Dr SW across Barbee Rd SW	$6,480	$7,452	$9,688
25	500-1000	2019	8	950	Williamsburg Ct	$91,200	$104,880	$136,344
26	500-1000	2019	8	670	Southeast of Conifer Pl SE and Northeast of Woodend Dr SE on Briarwood Pl SE	$64,320	$73,968	$96,158
27	500-1000	2019	8	1270	Southwest of Wilmart St NW and Northeast of Buford St NW on Winecoff Ave NW	$121,920	$140,208	$182,270
28	500-1000	2019	12	2970	South of Shady St NE and North of Church St N on Lake Concord Rd NE+ South of Lake Concord Rd NE and North of Plalaside Dr NE on Arbor St NE	$427,680	$491,832	$639,382

Priority	NFF Deficient by gpm	CIP Year	Diameter	Length	Location	Construction Cost ($)	Cost of Construction + Engineering ($)	Cost of Construction + Engineering + Contingencies ($)
29	500-1000	2019	6	1360	South of Blackwelder St SW and north of Paddington Dr SW on Rosegaye Ave SW connecting to Paddington Dr SW	$97,920	$112,608	$146,390
30	500-1000	2019	6	305	North of Mcgill Ave NW on Ney St NW	$21,960	$25,254	$32,830
31	500-1000	2019	12	690	Northeast of Chrch St N and Southwest of Englewood St NW on Todd Dr NE	$99,360	$114,264	$148,543
32	500-1000	2019	12	30	on the cross of Newell St and Todd Dr NW	$4,320	$4,968	$6,458
33	500-1000	2019	6	320	North of Kay Ol SE connecting Patton CT SE and Lawndale Ave SE	$23,040	$26,496	$34,445
34	500-1000	2019	12	190	Southwest of Powder St SW and South of Pecan Ave SW on Melrose Dr SW	$27,360	$31,464	$40,903
35	500-1000	2019	6	1280	Southwest of Homerine St NE and Northeast of Simpson Dr NE on Long Ave NE	$92,160	$105,984	$137,779
36	500-1000	2019	6	350	South of Mcgill Ave NW on Eudy Dr NW	$25,200	$28,980	$37,674
37	500-1000	2019	6	670	Southwest of Robbins St SW and Southeast of Sycamore Ct SW on Pecan Ct SW and Glen Rae St SW	$48,240	$55,476	$72,119
38	500-1000	2019	8	1515	East of Irish Potato Rd on Gold Hill Rd and Liberty Ridge Rd	$145,440	$167,256	$217,433
39	500-1000	2019	6	680	South of Corban Ave SE and North of Louise Dr SE connecting Washington Ln SE and Cirginia St SE	$48,960	$56,304	$73,195
40	500-1000	2019	8	85	North of Walnut Ave NW and East of Central Dr NW	$8,160	$9,384	$12,199
41	500-1000	2019	8	280	Southwest of Rabon St SE and Northeast of Sunset Dr SE on Glendale Ave SE	$26,880	$30,912	$40,186
42	500-1000	2019	6	370	South of Wilkinson Ct SE and North of Corban Ave SE on Hopkins St SE	$26,640	$30,636	$39,827
43	500-1000	2019	6	730	South of Hillcrest Ave SE and Northeast of Claymnt St SE on Glendale Ave SE	$52,560	$60,444	$78,577
44	500-1000	2019	8	660	North of Jameson Dr NW from Poplar Tent Rd	$63,360	$72,864	$94,723

Priority	NFF Deficient by gpm	CIP Year	Diameter	Length	Location	Construction Cost ($)	Cost of Construction + Engineering ($)	Cost of Construction + Engineering + Contingencies ($)
45	500-1000	2019	6	430	East of Alamance Dr NW on Monticello Dr NW	$30,960	$35,604	$46,285
46	0-500	2020	8	460	South of Eastcliff Dr SE and North of Arlington Ave SE on Marble St SE	$44,160	$50,784	$66,019
47	0-500	2020	8	115	Southeast of Corban Ave SE and Northeast of Crestside Dr SE on Mountview Ct SE	$11,040	$12,696	$16,505
48	0-500	2020	8	1200	South of East Cliff Dr SE on Union St S and Shephard Aly SE	$115,200	$132,480	$172,224
49	0-500	2020	6	300	South of Lawndale Ave SE and North of East Cliff Dr SE on State St SE	$21,600	$24,840	$32,292
50	0-500	2020	6	500	Southeast of Wilshire Ave SW and West of Sedgefield St SW on Hillside Ave SW	$36,000	$41,400	$53,820
52	0-500	2020	6	180	South of Camrose Cir NE on Carolina Ave NE	$12,960	$14,904	$19,375
53	0-500	2020	6	12	Northeast of Spring St NW across ST Mary Ave NW	$864	$994	$1,292
54	0-500	2020	8	230	North of Triple Crown Dr SW and on the cross of Stough Rd and Motorsports Dr SW	$22,080	$25,392	$33,010
55	0-500	2020	8	240	South of Raceway Dr SW on the cross of Stough Rd and Motosport Dr SW	$23,040	$26,496	$34,445
56	0-500	2020	6	110	Northeast of Powder St SW and South of Sycamore Ave SW on Melrose Dr SW	$7,920	$9,108	$11,840