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Water System Comprehensive Plan

Report

City of

Platteville, WI

June 2022

Revised December 2025

STRAND ASSOCIATES®

Report for

City of Platteville, Wisconsin

Water System Comprehensive Plan

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SECTION 1
INTRODUCTION

1.01 PURPOSE AND SCOPE

A. Purpose of Report

This report presents the comprehensive water system plan for the City of Platteville (City). The purpose of this study is to assess current distribution system performance, review existing supply and storage capacity, analyze the water system's ability to meet future demands, create and calibrate a hydraulic model of the system, and develop Opinions of Probable Construction Cost (OPCC) for water system improvements. This study will allow system improvements to be implemented in a practical and efficient manner.

B. Scope of Report

The scope of the comprehensive plan includes the following elements:

1. Summarize information from the *2006 Water System Master Plan*, the *Well No. 6 Site Investigation and Engineering Report,* and additional historical information of supply and storage facilities provided by the City.

2. Prepare a summary of the existing water supply and storage capacity of the water system.

3. Conduct a review of water system facilities to evaluate deficiencies and discuss potential improvements based on general observations.

4. Tabulate historical data from reports made to the Wisconsin Public Service Commission (PSC) dating back to 2006.

5. Use gathered data, population projections, and anticipated future growth areas to estimate current and 2040 design year demand.

6. Evaluate the existing ability of the wells and storage facilities to meet current and year 2040 maximum day demands and maximum day water plus fire demands.

7. Create a water system model in WaterGEMS that incorporates storage facility, pump, hydrant, valve, and supervisory control and data acquisition (SCADA) information.

8. Conduct up to ten field hydrant flow tests throughout the water system.

9. Perform a steady-state simulation calibration of the water model to industry-accepted standards using field hydrant flow testing results and SCADA information, and evaluate the system under current demand conditions. Evaluate the capacity of the system to meet current maximum day demand and fire flow needs.

10. Prepare a Capital Improvement Plan (CIP) that includes OPCC and implementation schedule for water system improvements developed from the system capacity and model analysis efforts.

1.02 ABBREVIATIONS AND DEFINITIONS

AWWA	American Water Works Association
CI	cast iron
CIP	Capital Improvement Plan
City	City of Platteville, Wisconsin
DI	ductile iron
GIS	geographical information system
gpcd	gallons per capita per day
gpm	gallons per minute
gpm/ft	gallons per minute per foot
HDPE	high density polyethylene
HVAC	heating, ventilation, and air conditioning
Hwy	Highway
I/O	input/output
ISO	Insurance Services Office
MCL	maximum contaminant levels
mg/L	milligrams per liter
MGD	million gallons per day
MSL	mean sea level
ND	non-detect
OPCC	Opinion of Probable Construction Cost
PSC	Wisconsin Public Service Commission
psi	pounds per square inch
PVC	polyvinyl chloride
SCADA	supervisory control and data acquisition
SCC	supervisor control center
SMCL	secondary maximum contaminant levels
USEPA	United States Environmental Protection Agency
WDNR	Wisconsin Department of Natural Resources
WDOA	Wisconsin Department of Administration
WEGS	*Water, Electric, Gas, and Sewer Annual Report*

SECTION 2
WATER FACILITIES EVALUATION

2.01 WATER DISTRIBUTION SYSTEM OVERVIEW

The City operates three wells and two elevated tanks that supply water through approximately 55 miles of water main ranging from 4 to 12 inches in diameter. There are two pressure zones in the system: a High Zone in the northeast and a Low Zone in the southwest. Table 2.01-1 summarizes the sizes and lengths of water main in the distribution system as reported to the PSC at the end of 2020. A map of the current distribution system with locations of key water facilities is shown on Figure 2.01-1.

Water Main Diameter (inches)	Length (feet)	Percentage of Total (%)
4	18,431	6.5
6	55,894	19.3
8	95,048	32.8
10	42,118	14.5
12	78,037	26.9
Total	289,807	100

Table 2.01-1 Existing Distribution System Water Main Inventory

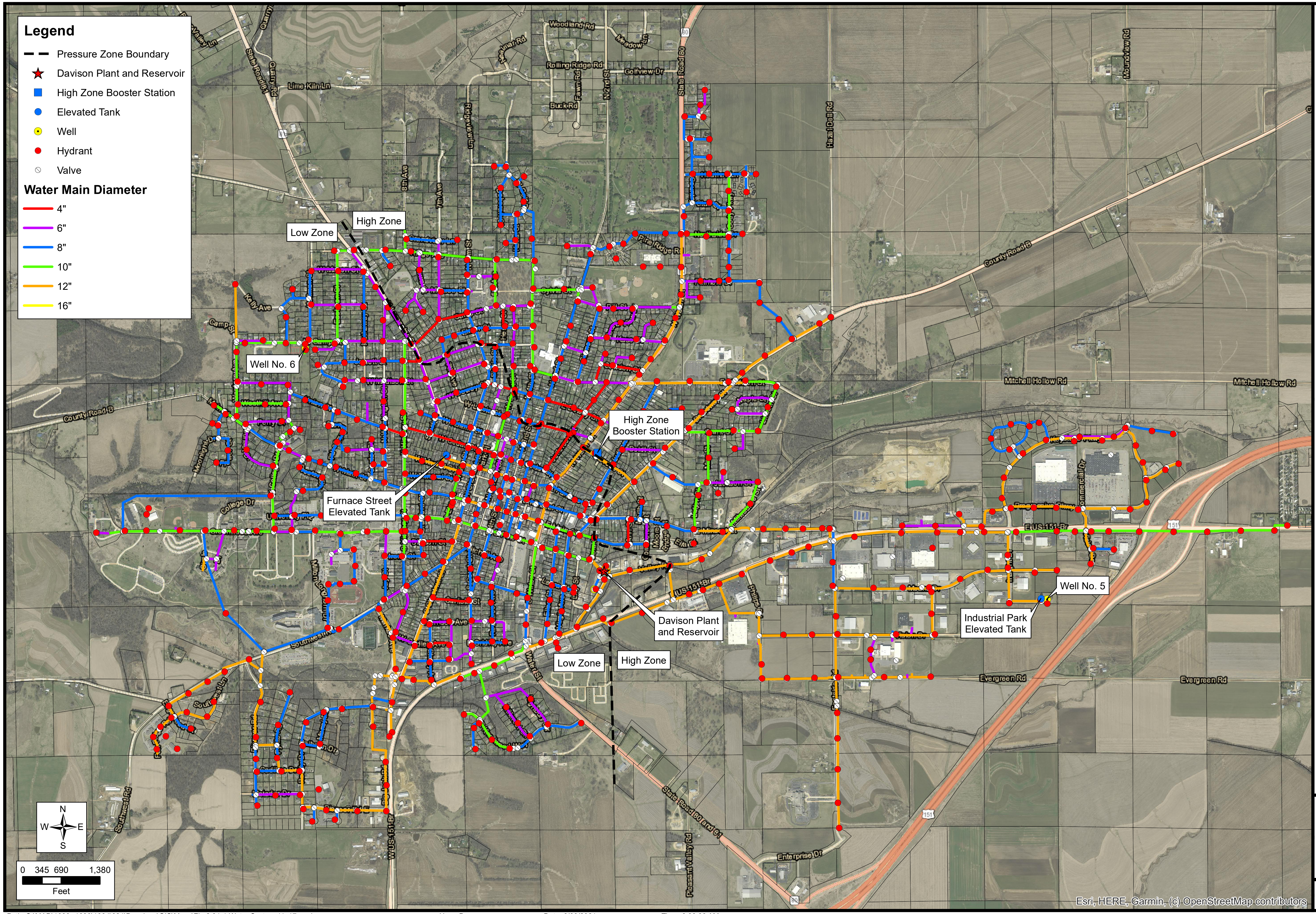
2.02 WELL SUPPLY SUMMARY

Table 2.02-1 presents the total and firm well capacities of the system. The firm well capacity of a system is defined as the total amount of capacity available when the largest well is out of service. The current capacities in this table are the pumping capacities reported by the City and are used for evaluating the ability of a system to meet present day and projected future demands. The current total well capacity for the system is 3,000 gallons per minute (gpm), or 4.32 million gallons per day (MGD). The current firm well capacity, assuming the largest well is out of service, is 1,900 gpm, or 2.74 MGD.

Well No.	Pumping Capacity (gpm)
3	900
5	1,100
6	1,000
Total Capacity	3,000
Firm Capacity*	1,900

*Assumes Well No. 5 is out of service

Table 2.02-1 Well Capacities



WATER DISTRIBUTION SYSTEM

CITY OF PLATTEVILLE
GRANT COUNTY, WISCONSIN

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FIGURE 2.01-1
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A. Well No. 3

Well No. 3, shown in Figure 2.02-1 is located at 750 Valley Road in the Low Zone of the distribution system. This well resides approximately 250 feet to the south of the Davison Plant, which is also located on the same site. Well No. 3 was constructed in 1932 as a sandstone and dolomite well and drilled to a depth of 927 feet and cased to a depth of 334 feet. At the time of construction, the well was test pumped at 1,050 gpm at a pumping level of 311 feet below ground. The well was reported to have a specific capacity of 9.5 gallons per minute per foot (gpm/ft) at the time of construction; however, it has since decreased significantly. According to televised well log records, the well was blasted down to a depth of 1,000 feet to increase the specific capacity. The well pump has a rated capacity of 900 gpm.

The Well No. 3 building, located adjacent to the Davison Plant, contains the well head and associated electrical controls. Water from the well is pumped into the Davison Plant where it is treated with sodium hypochlorite, sodium silicate, and caustic soda. These chemicals are stored in the south end of the facility near the point where the Well No. 3 water enters the building. Water quality sampling is taken at a point downstream of these chemical injection points. Well No. 3 cannot function in the event of a power outage as the generator can only provide water without chemical treatment as the well has its own dedicated generator plug and transfer switch. After chemical treatment, water flows to an on-site 500,000-gallon reservoir for storage until use.

The Davison Plant centrifugal booster pumps and motors are also located in the Davison Plant building and pump water from the on-site reservoir to the distribution system. One of the booster pump motors was installed in 1936 and is operating beyond its typical lifespan. The City noted that, if the motor were to experience a breakdown, there would be no replacement parts available to repair it. Additionally, there is a garage space, additional offices, and storage located in the Davison Plant.

Figure 2.02-1 Well No. 3

Well No. 3 was last rehabilitated in 2015, which included the installation of a new pump. In 2019, the well pump motor was replaced. This well is over 90 years old and has lost some of its capacity from initial installation. Due to the age of this well, Well No. 3 should be considered for rehabilitation on approximately a five-year cycle if the City desires to continue to use the well.

B. Well No. 5

Well No. 5, shown in Figure 2.02-2, is located on Insight Drive in the High Zone of the distribution system. This well is located on the same site as the City's High Zone elevated storage tank. Well No. 5 was constructed in 2011 as a sandstone and dolomite well with the intent to replace the decommissioned Well No. 2, which was beyond its recommended service life. Well No. 5 was drilled to a depth of 1,040 feet and cased to a depth of 655 feet. The well was test pumped at 1,541 gpm for 20 hours at a pumping level of 440 feet below ground. The well was reported to have a specific capacity of 12.7 gpm/ft at the time of construction. The well pump has a rated capacity of 1,100 gpm, and City records have shown that this well has been able to maintain that capacity.

Figure 2.02-2 Well No. 5

The Well No. 5 facility contains a pressurized filter for iron removal including oxidation and backwash equipment. Sodium hypochlorite is fed into the water pre- and post-filtration. Fluoride is also added to the water after filtration. This facility contains a future chemical room for corrosion control, if needed. The building also includes a water utility office, break room, locker rooms, and vehicle storage area. This filter system is shown in Figure 2.02-3.

Well No. 5 was last rehabilitated in 2019. Since its construction in 2011, the City has not noted any deficiencies or losses in capacity. The well and facility are in excellent condition. Because this well is relatively new, the well should be considered for rehabilitation on an 8-year cycle.

Figure 2.02-3 Well No. 5 Well Head and Filter System

C. Well No. 6

Well No. 6, shown in Figures 2.02-4 and 2.02-5, is located on Camp Street in Westview Park, on the same site as the City's recently abandoned Well No. 4. Well No. 4 was abandoned due to a collapse in formation and failed well casing causing cascading water, air entrainment, and reductions in specific capacity and pumping capacity. Well No. 6 was constructed in 2019 to a depth of 965 feet and cased to a depth of 500 feet. The well was test pumped at 1,483 gpm for 24 hours at a pumping level of 438 feet below ground. The well was reported to have a specific capacity of 8.6 gpm/ft at the time of construction. The well pump has a rated capacity of 1,000 gpm, and City records have shown that this well has been able to maintain that capacity.

Figure 2.02-4 Well No. 6 Building

Well No. 6 includes a submersible well pump and uses a pitless unit to discharge water through buried piping to the well facility. The piping room for Well No. 6 facility contains the injection points for the chemicals and the electrical gear. The chemical room contains the storage and feed systems for sodium hypochlorite, fluoride, sodium silicate, and caustic soda. The chemical feed lines are routed through the floor and into the piping room. This facility also contains a restroom.

Because Well No. 6 was constructed and the pump installed in 2019, and there have been no noticeable losses in capacity, the well and pump are considered to be in excellent condition. The well should be scheduled for rehabilitation on an approximately eight-year cycle or as performance data dictates the need. The well facility has been upgraded over time and is in very good condition.

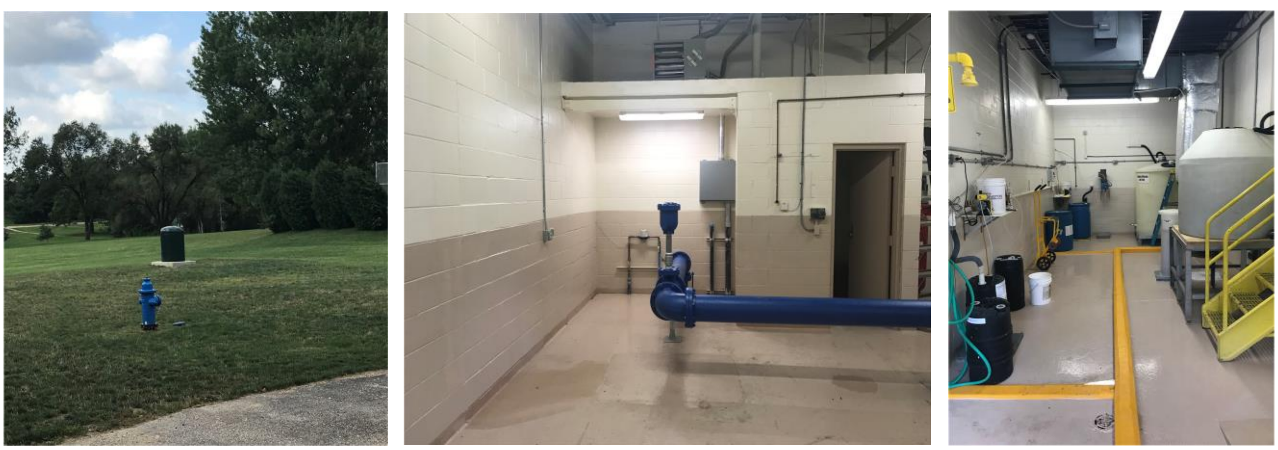


Figure 2.02-5 Well No. 6

2.03 WATER QUALITY SUMMARY

This section presents a summary of water quality in each of the raw water supply wells. Table 2.03-1 shows water quality results from the wells. Testing results are displayed as a range of minimum and maximum values that occurred during that time period or if there was a non-detect (ND). This analytical data is compared with the United States Environmental Protection Agency (USEPA) primary and secondary drinking water standards for each component. The primary drinking water standards, also known as maximum contaminant levels (MCL), are established to protect public health while secondary standards, also known as secondary maximum contaminant levels (SMCL), set maximum limits for aesthetic purposes.

Contaminant	MCL (mg/L)	SMCL (mg/L)	Well No. 3	Well No. 5	Well No. 6
Arsenic (As)	0.01	--	0 to 0.0003	0 to 0.0003	0.0012
Barium (Ba)	2	--	0.057 to 0.073	0.045 to 0.052	0.033
Chromium (Cr)	0.1	--	ND	ND	ND
Fluoride (F)	4	2	0.14 to 1.94	0.13 to 0.15	0.13
Hardness ($CaCO_3$)	--	--	315 to 331	309	330
Iron (Fe)	--	0.3	0.063 to 0.23	0.338	0.12
Manganese (Mn)	--	0.05	0 to 0.01	0.008	0.007
Mercury (Hg)	0.002	--	0 to 0.0002	ND	ND
Nickel (Ni)	0.1	--	0 to 0.0049	ND	0.0067
Nitrite-Nitrate (NO_3+NO_2)	10	--	0 to 0.26	ND	0.047
Sodium (Na)	--	--	1.43 to 12.6	1.41 to 1.51	5.2
Gross Alpha (pCi/L)	15	--	4.3 to 8.1	7.6 to 7.7	2.7 to 3.5
Combined Radium (pCi/L) (226+228)	5	--	3 to 4.5	1.1 to 1.7	0.6 to 2.6

mg/L=milligrams per liter

Table 2.03-1 Water Quality Summary

A. Well No. 3

Comparison of the inorganic and radionuclide data in Table 2.02-2 with the federal drinking water standards indicates that Well No. 3 samples do not exceed primary or secondary standards.

B. Well No. 5

Comparison of the inorganic and radionuclide data in Table 2.02-3 with the federal drinking water standards indicates that Well No. 5 complies with primary standards. The iron filter brings finished water quality below the secondary standards.

C. Well No. 6

Comparison of the inorganic and radionuclide data in Table 2.02-4 with the federal drinking water standards indicates that Well No. 6 samples did not exceed primary or secondary standards.

2.04 GROUND AND ELEVATED STORAGE SUMMARY

System storage includes two steel elevated tanks and a steel ground-level reservoir. The ground level reservoir is located at the Davison Plant in the Low Zone. Each zone includes one elevated storage tank. The High Zone elevated tank is located on Insight Drive in the eastern part of the distribution system. The Low Zone elevated tank is located on West Furnace Street near the center of the distribution system. A summary of the storage facilities is found in Table 2.04-1.

Storage Facility	Year Constructed	Capacity (gallons)	Overflow Elevation (feet MSL)
Davison Plant Reservoir	1988	500,000	937
High Zone Elevated Tank	1993	400,000	1,155
Low Zone Elevated Tank	1958	500,000	1,108
Total Storage		1,400,000	

MSL=mean sea level

Table 2.04-1 Existing Storage Capacity

A. Davison Plant Reservoir

The Davison Plant Reservoir, shown in Figure 2.04-1, is located to the north of the Davison Plant facility on the same site. This steel ground-level reservoir was constructed in 1988 and has a capacity of 500,000 gallons. The reservoir is 32 feet from the base to the overflow.

Figure 2.04-1 Davison Plant Reservoir

The tank was last cleaned in 2019. The tank's drain pipe and manway are both located on the south side of the tank. In general, the tank's exterior coatings are in good condition. This tank has also experienced issues with leakage around the base of the tank between the steel and concrete, which is presumed to be from the reservoir supply pipe.

B. High Zone Elevated Storage Tank

The High Zone Elevated Storage Tank, shown in Figure 2.04-2, is located on the same site as the Well No. 5 facility on Insight Drive. This elevated tank is a steel spheroid-style tank and was constructed in 1993. The elevated tank has a capacity of 400,000 gallons. The tank is 104 feet tall from the base to the overflow, which corresponds to an overflow elevation of 1,155 feet MSL.

The tank was last cleaned in 2019. The tank has several antennas mounted at the top of the tank. The tank's drain pipe and concrete splash pad are located on the northeast side of the tank, and the access door is located on the southeast side of the tank. The exterior coatings appear to be in good condition with a few areas near the top that are showing signs of wear.

Based on the age of the tank and original coating, the tank should be evaluated in more detail for overcoating or abrasive blasting and repainting.

Figure 2.04-2 High Zone Elevated Tank

Figure 2.04-3 Low Zone Elevated Tank

C. <u>Low Zone Elevated Storage Tank</u>

The Low Zone Elevated Tank, shown in Figure 2.04-3, is located near the center of the distribution system on Furnace Street. This elevated tank is a steel multi-legged tank with a capacity of 500,000 gallons. This tank was constructed in 1958 and is the oldest of the City's storage facilities. The height from the base to the overflow is 105 feet, which corresponds to an overflow elevation of 1,108 feet MSL.

The tank was last painted in 2019. The tank has several antennas mounted near the top of the tank. There are a few ancillary structures around the base of the tank owned by cellular and cable providers. The tank's drain pipe is located on the northwest side of the tank, and the ladder is located on the west side of the tank. Because the tank was last painted in 2019, new coatings are not needed in the near future. The City should plan on a 25-year life cycle for tank coatings.

2.05 CONDITION ASSESSMENTS

This section presents condition assessments of each facility based on a field visit conducted during this evaluation.

A. <u>Well No. 3 Well House and Davison Plant</u>

The Well No. 3 well house is in good condition overall. The City has proactively maintained the facility and the well continues to be a reliable source of water. However, the well has been in service for 90 years and is approaching the end of the typical service life. The City should plan to replace the well in the next 10 years.

The Davison Plant building, shown in Figure 2.05-1, is in fair to poor condition overall. Many parts of the building have been repurposed over time and the City has maintained the building to the extent possible. The masonry on the exterior and interior has been repaired in spots but continues to deteriorate. While the chemical feed equipment is well maintained and is in good condition, the chemicals are housed in a common room and the systems do not adhere to current Wisconsin Department of Natural Resources (WDNR) code.

Figure 2.05-1 Davison Plant

Table 2.05-1 summarizes the condition of major components of Well No. 3 and Davison Plant. The City should begin planning for a new well facility to replace Well No. 3. Further recommendations are discussed in Section 6.

Table 2.05-1 Condition of Major Components

Major Component	General Condition	Typical Remaining Life	Recommendation
Well No. 3 pump and motor	Excellent; replaced in 2015.	10 years	Continue operation and monitor performance.
Booster pumps and motors	Excellent (pumps). Poor (motors). Original motor from 1936. No replacement parts available	Pump heads: 10 to 15 years Motors: <5 years (at end of useful life)	Continue operation and monitor performance. Decommission when facility removed from service.
Electrical panels and building wiring	Fair to Poor, original equipment.	<5 years (at end of useful life)	Continue operation and monitor performance. Repair as needed.
Supervisory control center (SCC) and SCADA components	Fair.	Monitor for rehabilitation or replacement every 10 years	Continue operation and monitor performance.
Floor and wall coatings	Poor; floor coating is either missing or cracked away and spalling.	<5 years (at end of useful life)	Repair as needed.
Chemical feed equipment	Pumps and scales are in fair condition. Chemical containment curbing is needed for storage tanks.	5 to 10 years	Continue operation and monitor performance. Replace components when needed.
Water piping	Poor; paint is cracked away or peeling in many places.	10 to 15 years	Continue operation and monitor condition. Repair as needed.
Insulation	Poor; missing in some places.	<5 years (at end of useful life)	Continue operation and monitor condition. Repair as needed.
Heating, ventilation, and air conditioning (HVAC)	Poor; original equipment.	<5 years (at end of useful life)	Continue operation and monitor condition. Repair as needed.
Roof	Poor; original roof in the southern one-half of the facility. Roof is also missing insulation.	<5 years (at end of useful life)	Repair as needed.
Interior drywall	Poor; cracked or missing in many places leaving exposed brick.	<5 years (at end of useful life)	Leave as is until facility removed from service.
Brick veneer and fascia	Poor; original materials.	<5 years (at end of useful life)	Repair as needed.
Doors and hardware	Fair.	5 to 10 years	Leave as is until facility removed from service.
Driveway	Fair.	5 to 10 years	Leave as is until facility removed from service.

B. Well No. 5 Facility

The Well No. 5 facility is in excellent condition. Because the Well No. 5 facility was constructed in 2011 and has been maintained in excellent condition since its construction, all major facility components at this facility are in good working condition.

The City should continue to monitor well performance including capacity and water levels. Well rehabilitation should be performed when well performance starts to decline, or every 10 years. The well pump should be pulled every eight to 10 years to evaluate the need for repairs.

Filter performance should be monitored to look for signs of deterioration. Filter media has a typical life of 20 to 30 years. Filter media should be checked annually by visual observation to look for signs of media loss and uniform thickness. Uneven bed depths and clumping of media may be a sign of inefficient backwashing. Media should be added to top off the cells if signs of media loss are apparent.

C. Well No. 6 Facility

Well No. 6 and the well facility have undergone considerable improvements since 2018, with the drilling of the new well and installation of the new pitless unit site piping. The facility is in overall good condition with few improvements needed.

A recommended improvement at the Well No. 6 facility is the replacement of the chemically resistant floor coatings. While a portion of the coatings were replaced as a part of the 2018 project, approximately one-half of the coatings in the piping room are older and flaking in places. It is recommended that, at minimum, the floor in the piping room be recoated where it was not coated as a part of the 2018 well project.

Another potential improvement at the Well No. 6 facility is increasing the currently limited amount of additional space for input/output (I/O). If additional electronically driven processes were needed at this facility, there may not be enough space in the control cabinet to accommodate them. It is recommended to install additional space for I/O as needed.

D. Pressure Zone Transfer Station

The Pressure Zone Transfer Station, shown in Figure 2.05-2, located on Stevens Street has three pumps (with capacities of 500, 500, and 2,000 gpm, respectively) to pump water from the Low Zone to the High Zone. This facility is in excellent condition with no noted deficiencies or improvements needed. The City continues to maintain the facility regularly including rebuilding of pumps and control valves. It is recommended to continue to monitor the facility, including periodic operation of valves and pumps, to ensure the facility continues to operate in good condition.

Figure 2.05-2 Pressure Zone Transfer Station

SECTION 3
HISTORICAL AND PROJECTED DEMANDS

3.01 GENERAL

This section presents historic water demand trends observed by the City and develops a projection of future demands. Water use trends are applied to population projections and anticipated growth areas to estimate future water demands to the year 2040.

Water demand rate terminology used in this report is defined as follows:

A. Average Day

The total volume of pumped water in a year divided by the number of days in the year.

B. Maximum Day

The day of the year on which the maximum amount of water is pumped. The maximum day typically occurs during dry summer months when lawn watering is at a maximum. Maximum day use can also be attributed to main breaks and system maintenance.

C. Maximum Hour

The hour on the maximum day during which the maximum amount of water is pumped.

D. Fire Demand

The estimated amount of water required in a community to fight a fire. This demand is generally specified as a rate of flow, in gpm, for a given period of time, in hours. The calculated fire demand is added to the domestic demand during the maximum day to obtain the demand on a day that a major fire occurs. Fire demand generally increases the volume of storage that must be available on a maximum day.

Due to the number of variables impacting water consumption, it is difficult to make precise estimates of future water sales and demands. The Manual of Water Supply Practices M50 *Water Resources Planning* from the American Water Works Association (AWWA) outlines forecasting methods in Chapter 3. The Population Method was used along with reasonable usage factors and patterns experienced by the City to forecast future demands.

Prudent operation of a water utility requires that the system capacity always be in excess of system demands. Hence, recommended future improvements may be deferred until they become necessary, or be implemented sooner if demands increase at a rate faster than projected.

3.02 POPULATION

Figure 3.02-1 presents United States Census Bureau population data for the City from 1980 to 2020. This figure also shows projections from the Wisconsin Department of Administration (WDOA) Demographic Services Center and a modified projection that uses the WDOA growth rate applied to the 2020 Census value. Census data shows the City's population grew at a rate of approximately 5 percent between 2010 and 2020. The WDOA and modified growth projections reflect a growth rate of approximately 6 percent each decade. The modified growth projection is expected to provide a reasonable basis for future water supply needs.

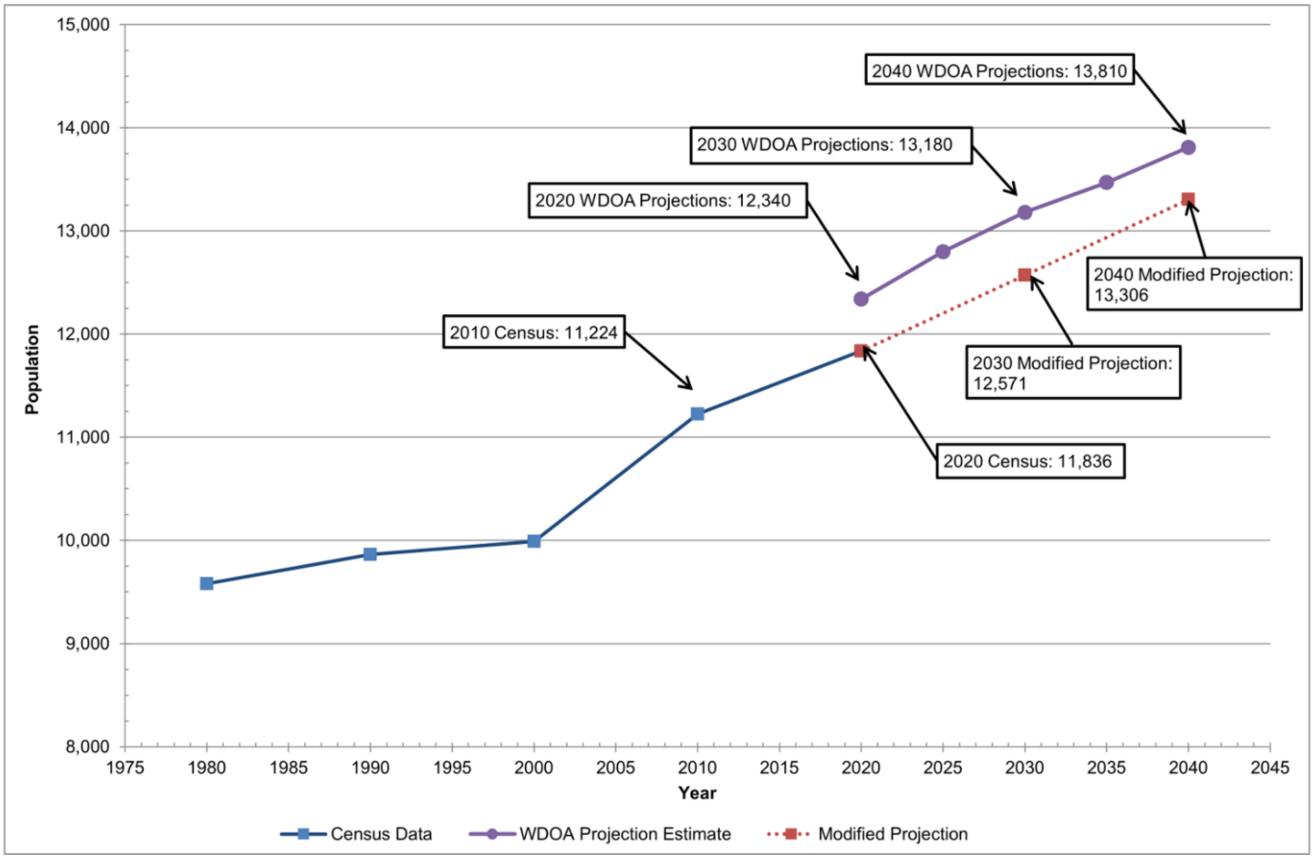


Figure 3.02-1 Population Projections

3.03 WATER SALES AND PUMPAGE

Historical water use records were obtained from the PSC *Water, Electric, Gas, and Sewer Annual Report* (WEGS) for the years 2006 through 2019. A summary of the historical water pumpage and sales data is shown in the Appendix.

A. Sales-to Pumpage Efficiency

Figure 3.03-1 presents the percentage of water pumped that was accounted for by metered sales since 2006. Sales will be less than pumpage because of meter losses, leakage, water main breaks, and hydrant flushing. This efficiency has ranged from 97 percent in 2013 and 2017 to 81 percent in 2008. The trend in efficiency has typically ranged from 90 to 95 percent in recent years, so an intermediate value of 92 percent will be used for the present day and 2040 design years.

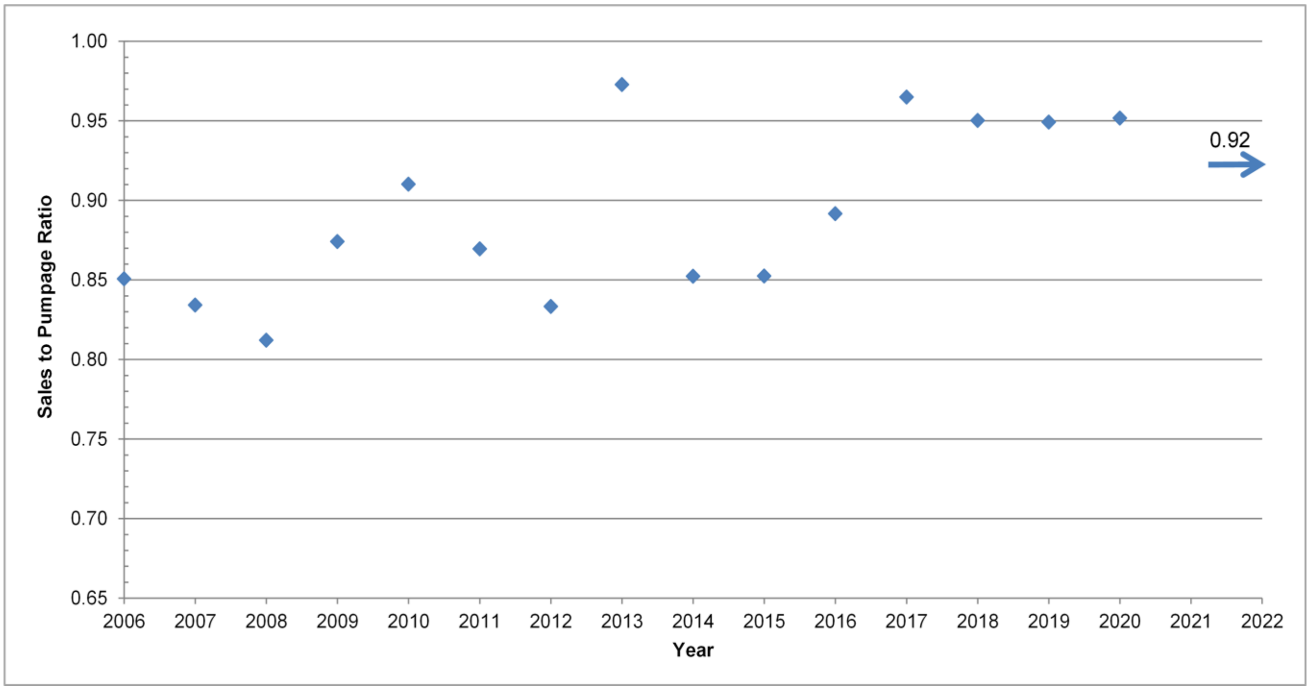


Figure 3.03-1 Sales to Pumpage Ratio

B. Maximum to Average Day Demand Ratio

Figure 3.03-2 presents maximum day to average day demand ratios since 2006.The values range from 1.43 in 2013 to 3.07 in 2019. Apart from the 2019 outlier, the maximum to average day demand ratio has typically ranged from 1.5 to 2.0 since 2006. A maximum to average day ratio of 2.5 will be used to forecast 2019 and 2040 maximum day demands. This value is higher than the 10-year average of 1.90. However, as ratios of over 3.0 have been reached in recent years, a higher ratio is being considered to provide a conservative value for planning purposes.

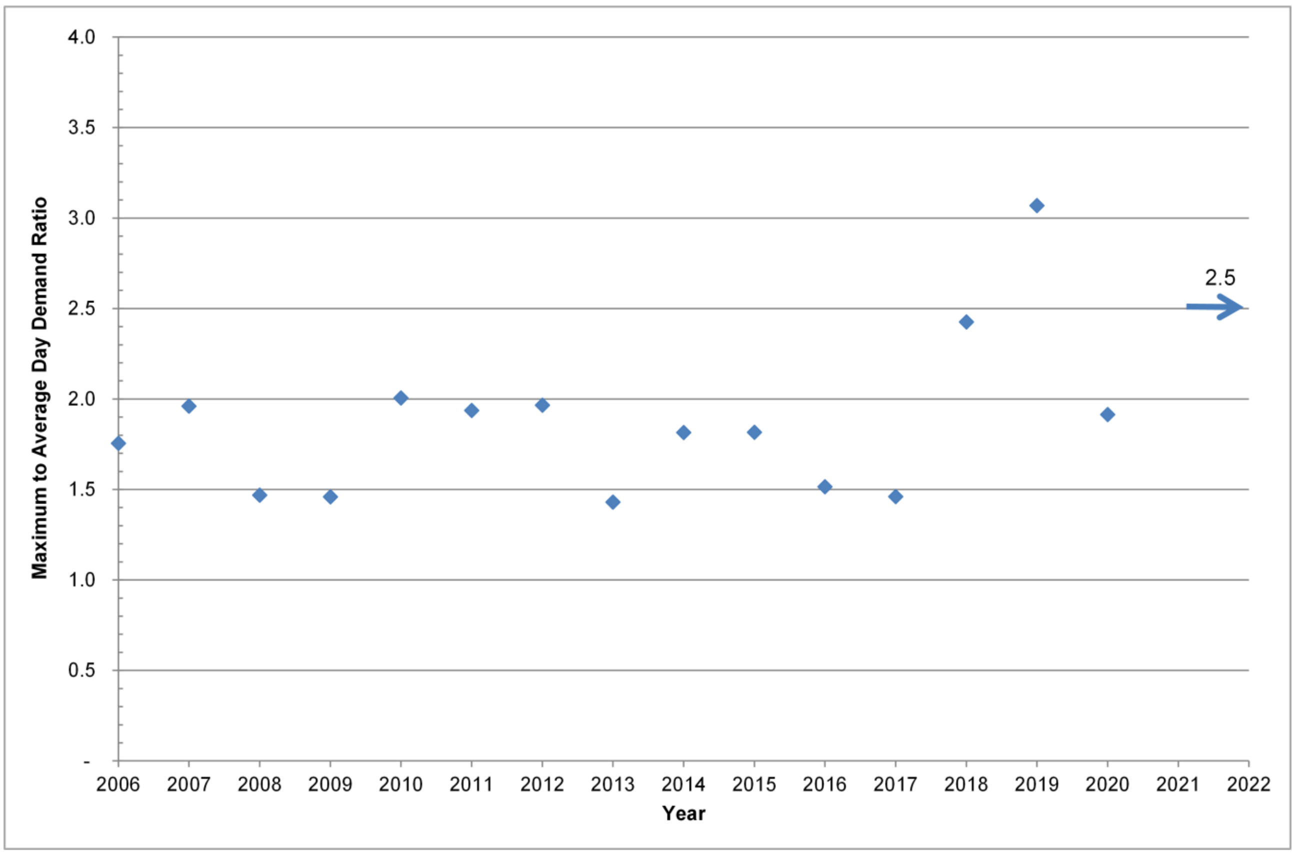


Figure 3.03-2 Maximum to Average Day Demand Ratios

C. Per Capita Sales By Category

Figure 3.03-3 presents sales per capita per day values since 2006 for residential, commercial, industrial, and public sales categories. Sales per capita is calculated by taking the total sales for each category and dividing by the estimated population that year. Industrial, public, and multifamily per capita sales have remained relatively steady since 2006 with little fluctuation. Residential sales declined in 2014 but have remained steady since the initial decline. Commercial per capita sales also showed the same trends as the residential sales, but their decline began in 2013. It is anticipated that future water use will remain consistent with past usage. To reflect recent trends in the per capita sales categories, a residential per capita sales of 25 gallons per capita per day (gpcd), a commercial per capita sales of 10 gpcd, a public per capita sales value of 12 gpcd, an industrial per capita sales value of 7 gpcd, and a multifamily per capita value of 6 gpcd will be used.

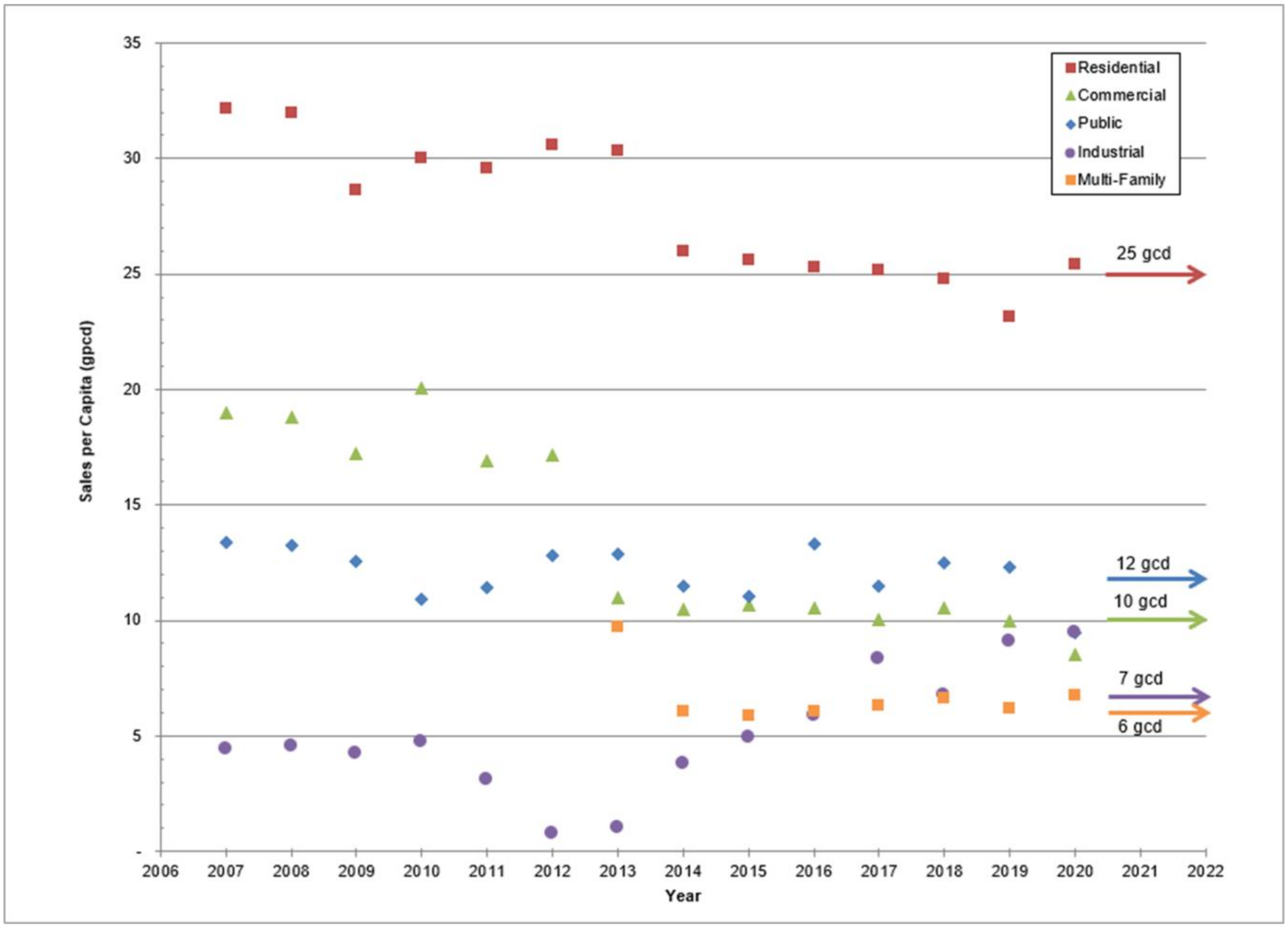


Figure 3.03-3 Per Capita Sales by Category

D. Total Per Capita Sales

Figure 3.03-4 presents total sales per capita per day values since 2006 and presents the summation of residential, commercial, industrial, public, and multifamily sales categories. Historic data shows an overall decreasing trend in the total per capita sales between 2006 and 2014, and an increasing trend since 2015. The minimum value of 53 gpcd occurred in 2015, and the maximum value of 69 gpcd occurring in 2007. Adding together the residential, commercial, industrial, public, and multifamily per capita sales forecasts results in a total per capita sales forecast of 60 gpcd. This value will be used for the present day and 2040 design year demand projections.

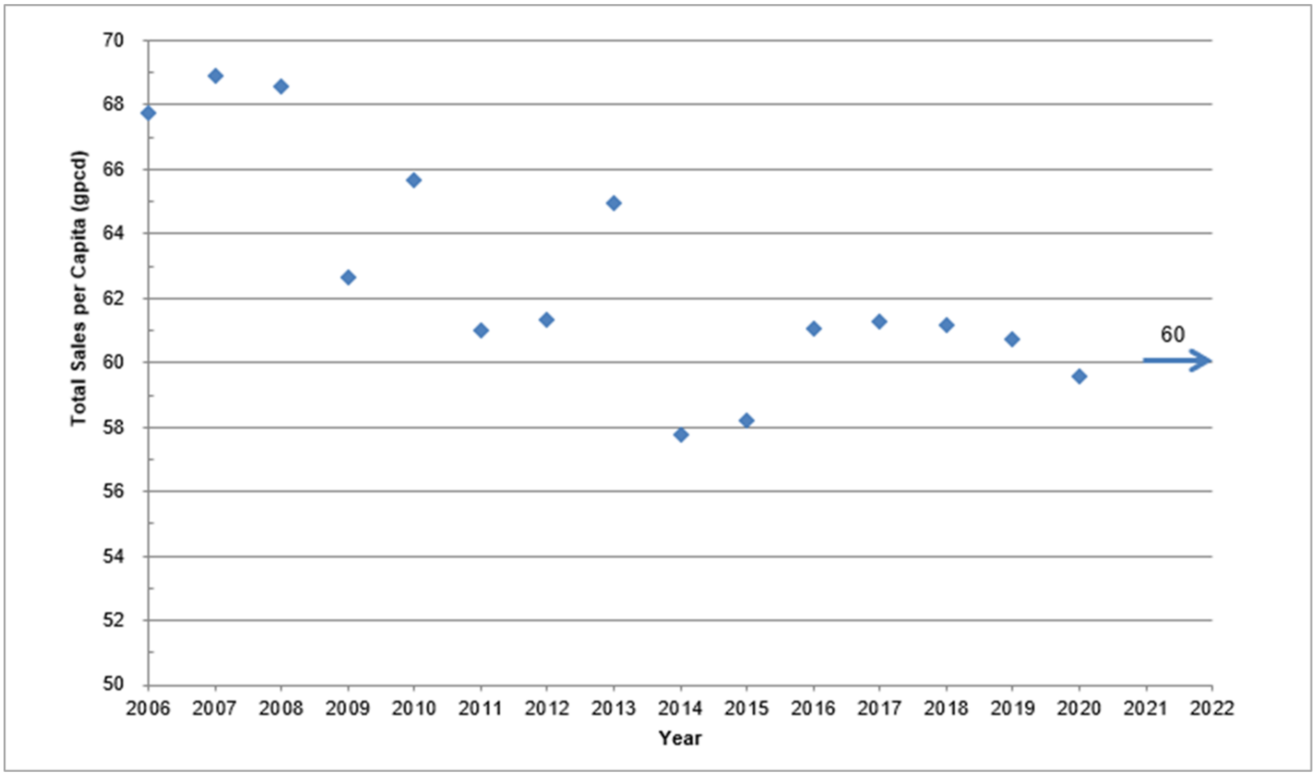


Figure 3.03-4 Total Per Capita Sales

3.04 2022 PROJECTED DEMANDS

Demand projections were calculated using the water use trends from the previous sections. The projected present day and 2040 demands will be used in the following section where demands will be compared to available supply.

A. <u>2022 Average Day Demand</u>

Based on the demand factors above, the average day demand in 2022 is estimated to be 781,500 gpcd (543 gpm). This was calculated by multiplying the design population of 11,983 by the projected per capita sales (60 gpcd) and dividing by the corresponding sales to pumpage ratio (0.92).

B. <u>2022 Maximum Day Demand</u>

1. Domestic

The 2022 maximum day pumpage is estimated to be approximately 1.95 MGD, which was calculated by applying the maximum to average day ratio of 2.5 to the 2022 average day pumpage. This is equal to a demand rate of 1,357 gpm.

2. Domestic Plus Fire

The Insurance Services Office (ISO) typically recommends basic fire flow requirements that are based on the amount of water a municipality should be able to supply. The required fire flow for individual buildings can range from a minimum of 500 gpm for 2 hours to a maximum of 12,000 gpm for 4 hours for large industrial complexes. The maximum basic fire flow requirement the ISO will credit a community that contains industrial-type facilities is 3,500 gpm for a duration of 3 hours. A fire flow of 3,500 gpm for 3 hours will be assumed for this study.

The total volume of water required to fight a fire on the 2020 maximum day is estimated as follows:

Domestic Maximum Day	1,953,750 gallons
Fire (3,500 gpm for 3 hours)	630,000 gallons
Total	2,583,750 gallons

The average rate at which this water would be used during the fire would be:

Domestic Maximum Day	1.95 MGD	=	1,357 gpm
Fire (3,500 gpm for 3 hours)	5.04 MGD	=	3,500 gpm
Total	5.99 MGD	=	4,857 gpm

3.05 AREAS OF FUTURE GROWTH

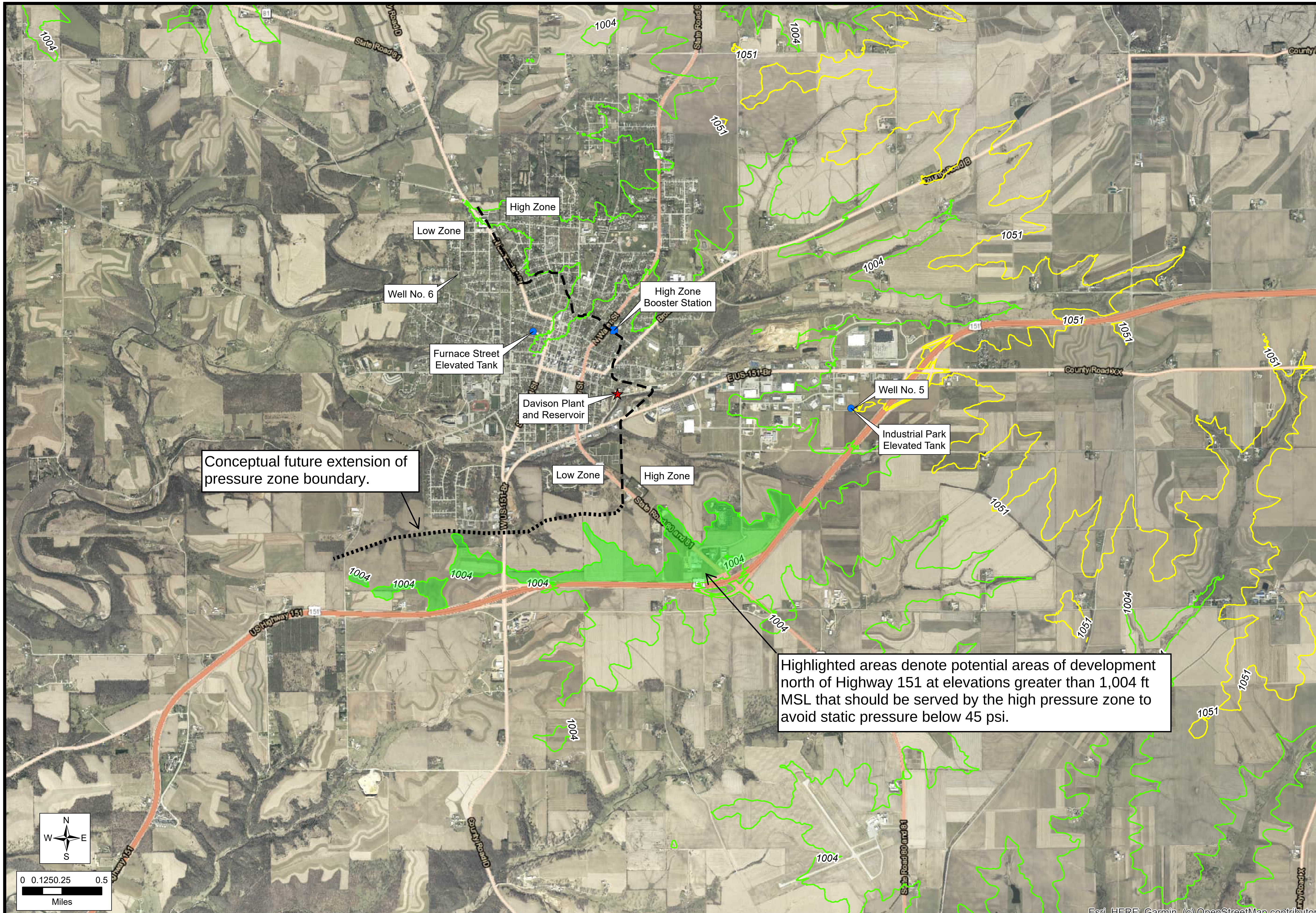
This section reviews areas of potential growth and the associated water system expansion needed to serve those areas. Assumed growth areas are based on previous planning documents provided by the City and discussion with City staff about recent development activity.

In general, the south and southeast areas bound by Highway (Hwy) 151 and the existing water service area provide the highest potential for growth. In particular, the City receives periodic inquiries regarding development in the corridor along Hwy 80/81. Development in this area is assumed to be a mixture of residential, commercial, and light industrial. The north part of the City is a potential area of moderate residential growth.

Areas of growth were evaluated to determine which pressure zone is best suited to serve new areas of development. Water service pressure is dictated by ground elevation and the water level in the controlling elevated storage tank in each zone. For the purpose of evaluating future areas of expansion, this study assumes a minimum acceptable service pressure of 45 psi and maximum pressure of 100 psi. Figure 3.05-1 shows ground elevations and the conceptual extension of the pressure zone boundary.

The overflow elevation of the Furnace Street water tower (low pressure zone) is 1,108 feet MSL. Assuming a minimum static system pressure of 45 psi, the corresponding maximum service elevation is 1,004 feet MSL.

The overflow elevation of the Industrial Park water tower (high pressure zone) is 1,155 feet MSL. Assuming a minimum static system pressure of 45 psi, the corresponding maximum service elevation is 1,051 feet MSL.



AREAS OF FUTURE GROWTH

CITY OF PLATTEVILLE
GRANT COUNTY, WISCONSIN

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FIGURE 3.05-1
1064.021

Path: S:\MAD\1000--1099\1064\021\Drawings\GIS\Maps\Fig 3.05-1 Areas of Future Growth 11x17.mxd User: danc Date: 6/22/2022 Time: 9:12:26 AM

As shown on Figure 3.05-1, there are areas of land north of Hwy 151 and on both sides of Hwy 80/81 at elevations that exceed 1,004 feet MSL. These areas would best be served by the high pressure zone by extending and building upon the existing water main along Eastside Road.

There are no areas that present a concern as it relates to pressure exceeding 100 psi.

As development occurs in the south and southeast, the City should obtain a site for a future well and storage facility to replace the Davison Plant as discussed in Section 6. A pressure relief valve (PRV) should also be considered near Hwy 80/81 as the pressure zone boundary is extended. A PRV in this area will provide a redundant connection allowing the high pressure zone to feed the low pressure zone when needed.

Future development in the north part of the City can be served by either zone based on where development occurs.

3.06 2040 PROJECTED DEMANDS

A. <u>2040 Average Day Demand</u>

The average day demand in 2040 was estimated to be 867,800 gpd (603 gpm). This was calculated by multiplying the design population of 13,306 by the projected per capita sales (60 gpcd) and dividing by the corresponding sales to pumpage ratio (0.92).

B. <u>2040 Maximum Day Demand</u>

1. Domestic

The 2040 maximum day pumpage is estimated to be approximately 2.17 MGD, which was calculated by applying the maximum to average day ratio of 2.5 to the 2040 average day pumpage. This is equal to a demand rate of 1,507 gpm.

2. Domestic Plus Fire

The total volume of water required to fight a fire on the 2040 maximum day is estimated as follows:

Domestic Maximum Day	2,169,500 gallons
Fire (3,500 gpm for 3 hours)	630,000 gallons
Total	2,799,500 gallons

The average rate at which this water would be used during the fire would be:

Domestic Maximum Day	2.17 MGD	=	1,507 gpm
Fire (3,500 gpm for 3 hours)	5.04 MGD	=	3,500 gpm
Total	7.21 MGD	=	5,007 gpm

Figure 3.06-1 displays the historical and projected average and maximum day demands from 2006 through the 2040 design year. Note, the outlier maximum day demand in 2019 was caused by a water main break on Camp Street.

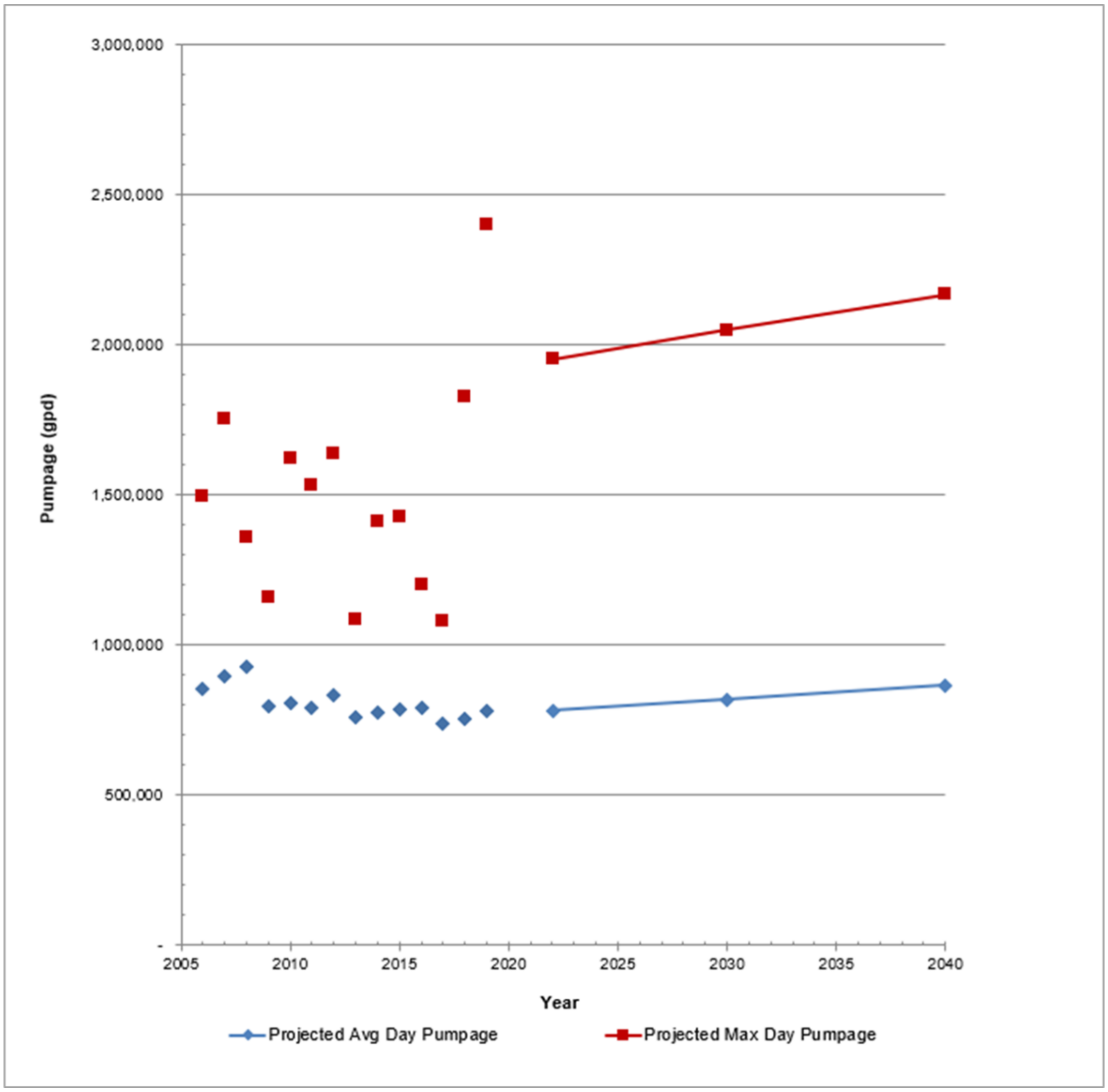


Figure 3.06-1 Historical and Projected Average and Maximum Day Pumpage

SECTION 4
ADDITIONAL REQUIRED CAPACITY

4.01 GENERAL

Days of maximum demand can occur consecutively, especially during the warm summer months. As a result, water withdrawn from storage during any one maximum day must be replaced before the following day to ensure an adequate supply of water for the next day. Therefore, total demand on the maximum day determines the minimum amount of water that must be available for the next day. It is recommended the system be designed to meet maximum day domestic demands with the most critical well out of service. The total amount of water that can be pumped from the wells with the largest well out of service is referred to as the firm capacity. If the firm capacity is less than the maximum day demand, storage will be depleted, and an inadequate amount of water may exist for the following day. Alternatively, if the firm capacity meets or exceeds total demands, all storage facilities may be refilled during any 24-hour period and water will be available to meet the following potential maximum day demand. Refer to Section 2 of this study for the current rated pump capacities.

If the firm pumping capacity just equals the maximum day domestic demand, the amount of storage required would be equal to fire requirements plus peak daily demands. Water withdrawn from storage facilities to meet fire demand does not need to be replaced the same day or the day following the fire. However, it is recommended to replenish the storage as soon as possible.

4.02 2022 CAPACITY ANALYSIS

A. 2022 Maximum Day Demand

The estimated 2022 maximum day demand is 1,357 gpm (1.95 MGD). The combined current capacity of the existing wells is 3,000 gpm (4.32 MGD). The firm capacity with Well No. 5 out of service is 1,900 gpm (2.74 MGD). Firm well capacity exceeds the 2022 maximum day demand and the system has a well supply surplus of 543 gpm. No additional well supply is needed to meet the present day maximum day demands.

B. 2022 Maximum Day Demand–Fire Flow

The total amount of water available to satisfy maximum day demand plus fire demand is equal to the firm well capacity plus the water available from usable storage. Section 3 of this study discusses the fire demand scenarios for the City. A demand rate of 4,857 gpm (1,357 gpm domestic demand plus 3,500 gpm fire demand) for three hours must be satisfied to provide the necessary fire protection for the system. Because a fire can start at any time during the day, the expected domestic demand must be taken into account when calculating available supply. It is assumed that the storage is not depleted at the start of the three-hour fire demand, and that 25 percent of elevated storage is reserved for daily operational needs.

Although there are 500,000 gallons of available storage at the ground-level reservoir at the Davison Plant, storage is only available at the capacity of its booster pumps that is in excess of the Well No. 3 pump capacity. The three booster pumps have a cumulative capacity of 4,800 gpm; however, it is assumed that only two of the booster pumps (totaling 2,800 gpm) will be operating during a fire event. Therefore, the rate of water that can be withdrawn from the Davison Plant reservoir is equal to 1,900 gpm.

Maximum Day Demand	-	1,357	gpm
Fire Demand	-	3,500	gpm
Firm Well Capacity	+	1,900	gpm
Elevated Storage Capacity[1]	+	3,750	gpm
Ground-Level Storage Capacity	+	1,900	gpm
Total	+	2,693	gpm

[1]Elevated Storage Capacity=675,000 gallons per 180 minutes

During a three-hour fire event, the system is projected to have a capacity surplus of 2,653 gpm, or approximately 485,000 gallons. Storage in the system is able to meet the 2022 maximum day demand with fire flow, and no additional storage is required.

4.03 2040 CAPACITY ANALYSIS

For the purpose of this analysis, it is assumed that no new supply or storage was constructed or decommissioned before the 2040 design year.

A. 2040 Maximum Day Demand

The total pumpage on the maximum day in 2040 is estimated to be 1,507 gpm (2.17 MGD). The existing firm well capacity is 1,900 gpm (2.74 MGD). The existing firm well capacity exceeds the 2040 projected maximum day demands. For the 2040 design year, the City has a surplus in well supply of 393 gpm and no additional well capacity is required. If an existing well were to be abandoned, additional supply would be needed to satisfy maximum day demands.

B. 2040 Maximum Day Demand–Fire Flow

The total amount of water available to satisfy maximum day demand plus fire demand is equal to the firm well capacity plus the water available from usable storage.

Maximum Day Demand	-	1,507	gpm
Fire Demand	-	3,500	gpm
Firm Well Capacity	+	1,900	gpm
Elevated Storage Capacity[1]	+	3,750	gpm
Ground-Level Storage Capacity	+	1,900	gpm
Total	+	2,543	gpm

[1]Elevated Storage Capacity=675,000 gallons per 180 minutes

During a three-hour fire event, the system is projected to have a capacity surplus of 2,543 gpm, or approximately 458,000 gallons. No additional storage is needed to meet the 2040 maximum day demand; however, the storage analysis should be reevaluated if existing storage, such as the Davison Plant reservoir, is removed from service.

4.04 CAPACITY ANALYSIS WITHOUT WELL NO. 3, DAVISON PLANT, AND RESERVOIR

Because of the age and condition of the infrastructure at the Davison Plant, it is recommended that this facility and the reservoir be taken out of service before 2040. For the purpose of this analysis, it is assumed that Well No. 3, the Davison Plant Reservoir, and the Davison Plant booster pumps are no longer in service. It is also assumed that no additional supply or storage has been constructed to replace the existing infrastructure. This will provide an estimate of supply and storage needs if these facilities are removed from service.

A. Present Day (2022) Maximum Day Demand

The total pumpage on the maximum day in 2022 is estimated to be 1,357 gpm (1.95 MGD). If Well No. 3 is abandoned, the firm well capacity of the system becomes 1,000 gpm (1.44 MGD). The firm well capacity without Well No. 3 is below the 2040 projected maximum day demands. Under this scenario, the system would have a well supply deficit of 357 gpm. Additional well capacity is needed before decommissioning Well No. 3 and the Davison Plant.

B. Present Day (2022) Maximum Day Demand–Fire Flow

The total amount of water available to satisfy maximum day demand plus fire demand is equal to the firm well capacity plus the water available from usable storage. Note: because the firm well capacity in this scenario is unable to meet the maximum day demands, a new well will be needed regardless of this storage and fire flow analysis. Because of the need for this new well, it was assumed that the firm well capacity would be sufficient enough to supply maximum day demands.

Maximum Day Demand	-	1,357	gpm
Fire Demand	-	3,500	gpm
Firm Well Capacity	+	1,357	gpm
Elevated Storage Capacity[1]	+	3,750	gpm
Ground-Level Storage Capacity[2]	+	0	gpm
Total	+	250	gpm

[1]Elevated Storage Capacity=675,000 gallons per 180 minutes

[2]Assumes Davison Plant Reservoir and booster pumps are taken out of commission

During a three-hour fire event, the system is projected to have a capacity surplus of 250 gpm, or approximately 45,000 gallons. While the projections show a system surplus of 250 gpm, additional storage is highly recommended to provide additional supply in the event of a large fire if Well No. 3 and the Davison Plant are removed from service.

C. 2040 Maximum Day Demand

The total pumpage on the maximum day in 2040 is estimated to be 1,507 gpm (2.17 MGD). If Well No. 3 is abandoned, the firm well capacity of the system becomes 1,000 gpm (1.44 MGD). The firm well capacity without Well No. 3 is below the 2040 projected maximum day demands. Under this scenario, the system would have a well supply deficit of 507 gpm. Additional well capacity is needed if Well No. 3 and the Davison Plant were to be decommissioned.

D. 2040 Maximum Day Demand–Fire Flow

The total amount of water available to satisfy maximum day demand plus fire demand is equal to the firm well capacity plus the water available from usable storage.

Maximum Day Demand	-	1,507	gpm
Fire Demand	-	3,500	gpm
Firm Well Capacity	+	1,507	gpm
Elevated Storage Capacity[1]	+	3,750	gpm
Ground-Level Storage Capacity[2]	+	0	gpm
Total	+	250	gpm

[1]Elevated Storage Capacity=675,000 gallons per 180 minutes
[2]Assumes Davison Plant reservoir and booster pumps are taken out of commission

During a three-hour fire event, the system is projected to have a capacity surplus of 250 gpm, or approximately 45,000 gallons. Again, while this analysis projects a system storage surplus of 250 gpm, additional storage is recommended for both the 2020 and 2040 scenarios if Well No. 3 and the Davison Plant are decommissioned.

4.05 ADDITIONAL REQUIRED CAPACITY

Timing the construction of the new supply and storage facilities is critical to ensure that the Davison Plant infrastructure is not relied upon too far beyond its useful lifespan while also ensuring that system demands are met. Figure 4.05-1 shows the firm well supply with and without Well No. 3 along with historic and projected maximum day demands. The figure illustrates the need for additional well supply before removal of Well No. 3 from service. A new well with a capacity of 1,000 gpm is recommended to keep pace with population growth and allow for replacement of Well No. 3. The timing and costs of a new well are discussed in Section 6.

Similarly, demolition of the 500,000-gallon reservoir at the Davison Plant will create a near-deficit of available storage. If the reservoir is removed from service, a new water storage facility with a capacity of 400,000 gallons is recommended for construction before decommissioning the Davison Plant and reservoir. Storage can be added in the form of either elevated storage or ground-level storage as part of the new well facility.

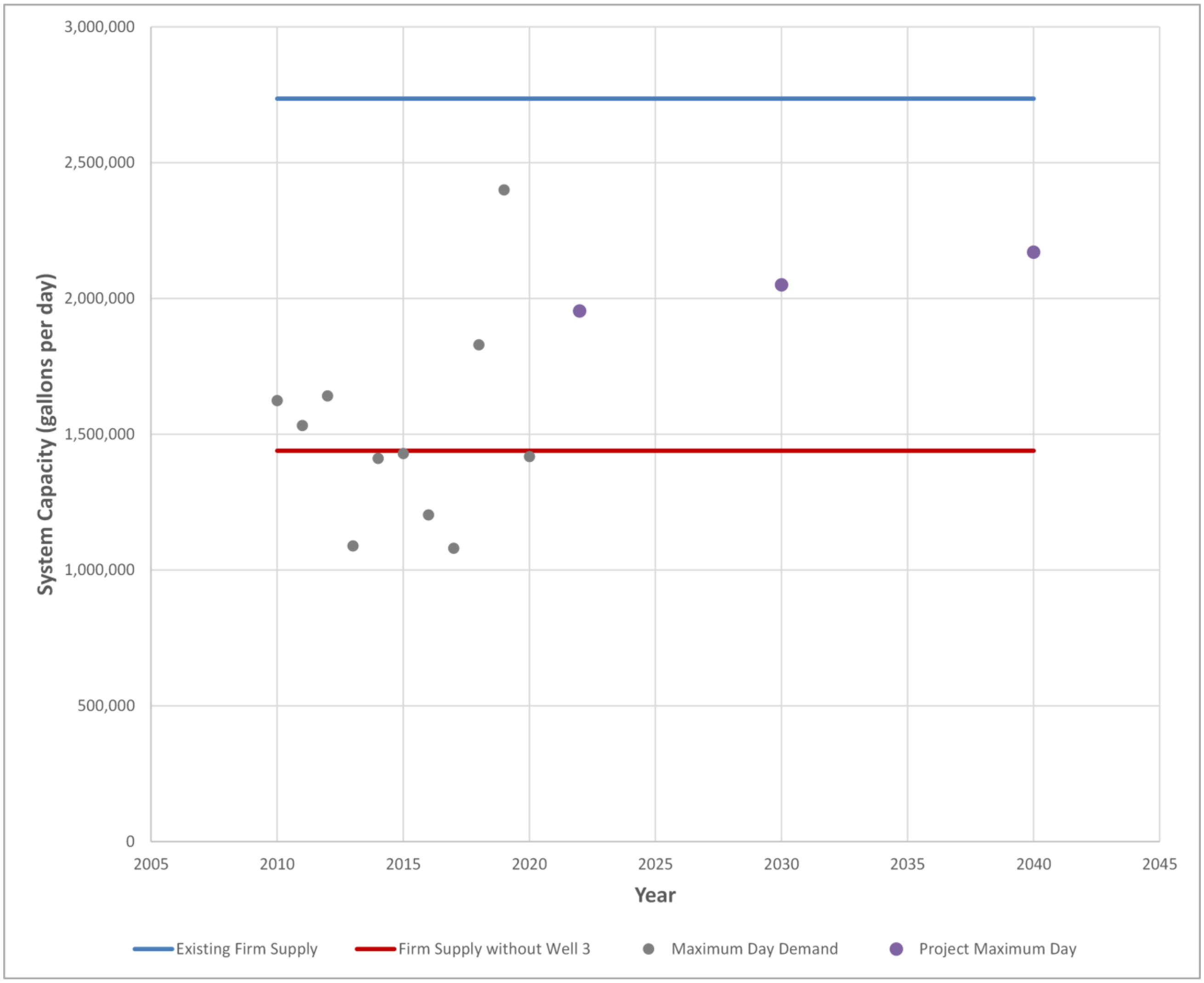


Figure 4.05-1 Demands Versus Firm Supply

SECTION 5
COMPUTER MODELING

5.01 GENERAL

This section summarizes the services completed in creating and calibrating the City's water system model, including the results of the current and future system modeling.

5.02 MODEL CREATION AND CALIBRATION

A. Model Creation with Geographic Information System (GIS) Data

A computer model of the City's water distribution system was created using WaterGEMS CONNECT software. The existing distribution system was imported into the model through the software's Model Builder tool with GIS shapefiles provided by the City. Attributes incorporated in these shapefiles include water main diameter, hydrant locations, and isolation valve locations. Well pump and storage facility information was manually entered into the model from information provided by the City. Each model junction between two (or more) pipes was assigned an elevation based on a 2-foot topographic Grant County contour map that was imported to the model using the software's Terrain Extractor tool.

The average and maximum day demands were distributed evenly through the system using a blanket demand approach.

B. Model Calibration with Field Flow Data

To properly calibrate the hydraulic model and simulate the existing pipe network, the model's results were checked against observed conditions in the system. These conditions were obtained through field testing of hydrants throughout the distribution system. On October 28, 2020, ten field fire flow tests were conducted. The locations of these field flow tests were chosen to provide a representative sample of the conditions throughout the distribution system.

Two hydrants were used for each field flow test: one monitoring hydrant and one flowing hydrant. Before the flowing hydrant was opened, a pressure gauge was attached to the monitoring hydrant to record the static pressure at the hydrant. This pressure gauge and the hydrant were both air purged before a static pressure reading was taken.

After recording the static pressure, the flowing hydrant was opened using one 2.5-inch outlet. A residual pressure reading was taken at the monitoring hydrant. If the pressure at the monitoring hydrant dropped more than 10 pounds per square inch (psi), gauge readings were recorded and the test was considered complete. If the pressure drop was less than 10 psi, an additional 2.5-inch outlet was opened. A pressure reading was taken at the flowing hydrant using a pitot tube and gauge.

After completing the field flow tests, the flows from the hydrants were calculated. This calculation was done using the pitot tube and gauge reading observed from the flowing hydrant and the diameter of the open outlet. Hydrant flow was calculated using the following equation:

$Q=(29.83)(C)(D^2)(P^{0.5})$=flow in gpm
C=outlet discharge coefficient (typically 0.9 for 2.5-inch-diameter outlets)
D=diameter of the outlet in inches
P=pitot pressure in psi

After the completion of the field flow tests, operating data including elevated storage tank levels and well pump flow was obtained from the City's SCADA system. This data was used to set the boundary conditions of the model. After inserting this data into the model, scenarios were run in the model that correspond to the flow tests under these observed conditions. Static and residual pressure results from the model simulations are presented with the field observed pressures for comparison in Table 5.02-1. The testing locations for each field flow test are shown in Figure 5.02-1.

Test Number	Flowing Hydrant Location	Field Static Pressure (psi)	Modeled Static Pressure (psi)	Field Residual Pressure (psi)	Modeled Residual Pressure (psi)	Field Measured Flow (gpm)
1	Heather Lane	45	48.2	35	33.9	1,061
2	North Elm Street	91	95.5	71	62.8	1,455
3	Jefferson Street	66	67.4	47	44.1	650
4	Cornerstone Circle	64	65.3	55	57.6	1,300
5	East Knollwood Way	70	69.8	59	54.0	1,405
6	Pyrite Road	92	90.4	78	79.6	1,500
7	Flower Court	74	74.8	60	56.6	1,405
8	Madison Circle	63	65.1	55	55.8	1,300
9	Mitchell Avenue	64	63.5	57	61.5	1,350
10	Eastside Road	65	64.3	55	55.9	1,190

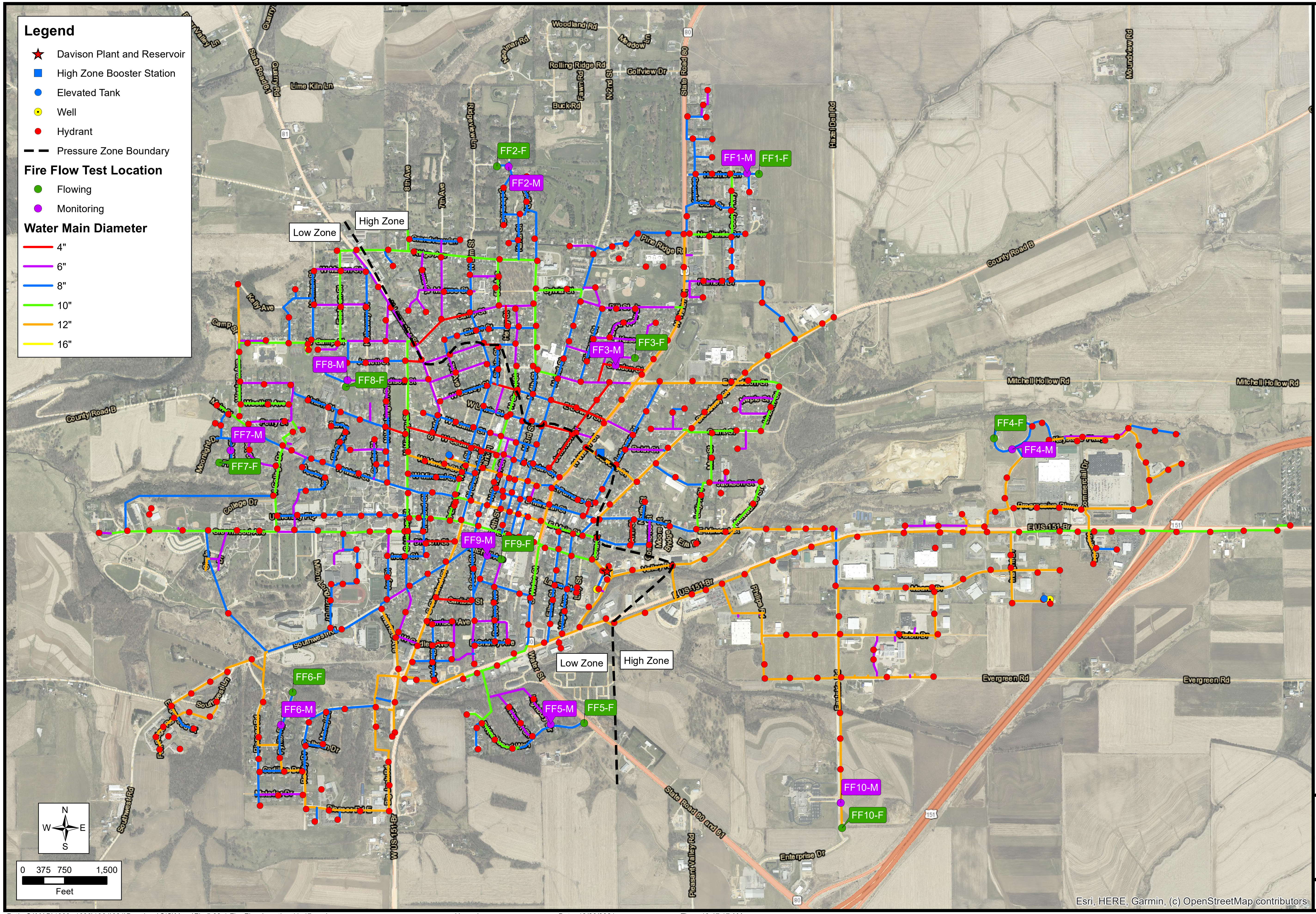
Table 5.02-1 Model Calibration Results

A computerized model is considered calibrated when static and residual pressures predicted by the model are within 5 psi of the observed field tests results. To bring the differences between the modeled and field pressures into acceptable calibration levels, C-factors were adjusted within the model based on pipe size, material, and age. For this calibration, pipe material was the primary consideration in determining C-factors using data available in the City's GIS system. The City's pipe materials ranged from polyvinyl chloride (PVC) to ductile and cast iron, all of which exhibit different roughening characteristics as they age. Table 5.02-2 displays the C-factors assigned to water main in the model based on water main material.

Pipe Material	C-Factor
HDPE	120
DI	110
CI	110
Unknown	125
PVC	125

HDPE=high density polyethylene
DI=ductile iron
CI=cast iron

Table 5.02-2 Water Model C-Factors



FIRE FLOW TEST LOCATIONS

WATER SYSTEM COMPREHENSIVE PLAN
CITY OF PLATTEVILLE
GRANT COUNTY, WISCONSIN

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FIGURE 5.02-1
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Path: S:\MAD\1000--1099\1064\021\Drawings\GIS\Maps\Fig 5.02-1 Fire Flow Location 11x17.mxd User: danc Date: 12/20/2021 Time: 10:47:17 AM

In addition, based on the initial round of static modeling results, a transducer correction was applied to the Low Zone Tank. This was necessary as transducer readings recorded placed the tank readings outside of the actual operating range of the tank. Shifting all tank readings by this transducer correction moved the static tests into calibration.

Following the C-factor and transducer corrections to the model, Test No. 2 was the only test not within calibration. However, it was observed during flow testing that field pressure recordings swung approximately 5 psi in either direction of the recorded value while the test was taking place. The instability of the observed pressures at this location, reduces its usefulness as a calibration location and as a result it was no longer considered as part of the calibration set. The modeled results in this area were lower than the observed results, and as a result should result in conservative estimates of available flow and pressure. Care should be taken when evaluating model results in this general area.

Based on the field testing and model simulations mentioned previously, the hydraulic model is considered calibrated for steady-state simulations.

5.03 PRESENT DAY MODEL ANALYSIS

The model was used to analyze present day conditions under several demand and flow scenarios. Three types of steady-state simulations were performed with this model: an average day domestic demand (non-fire) simulation, a maximum day domestic demand simulation, and a fire flow simulation.

A steady-state simulation evaluates the behavior of the system at a specific point in time under static conditions. In this type of simulation, the behavior of the well pumps, elevated tank, and the overall supply and storage relationship can be analyzed. This type of simulation is useful for determining pressures within the distribution system and flow rates under different demand conditions.

A fire flow simulation provides an instantaneous snapshot of the amount of water available at hydrants in the system while maintaining a minimum of 20 psi residual pressure. The model simulates a separate fire event at each junction in the system and increases the flow until either the hydrant itself or any point in the system reaches the 20-psi residual pressure threshold. Very high available fire flows (greater than 5,000 gpm) are not considered realistic, but rather indicate areas of very strong hydraulic connectivity.

A. Steady-State Average Day Demand

The average day domestic demand condition, equaling 543 gpm or 0.78 MGD, was modeled using a steady-state analysis with Well Nos. 5 and 6 operating, no booster pumps operating, the Davison Plant Reservoir out of service, and the elevated tanks set to 10 feet below its overflow elevation. The model projected pressures in the system range from 33 to 109 psi. The low pressures occur in the high zone on the far east end of the system on County Road XX. The low pressures are a result of older water main at relatively higher elevations. The areas of high pressure occur in low lying areas near Southwest Road and the City’s wastewater treatment plant. The high pressures are a result of low elevations relative to the rest of the system.

While these modeled simulations show system pressures below 35 psi, water levels in the elevated tanks are typical maintained higher than 10 feet below overflow so normal operating pressures across the system are above 35 psi. Because of elevations and modeled system pressures in the area east of Highway 151 on County Road XX, additional modeling analyses should be conducted if potential developments were to occur in this area.

B. Steady-State Maximum Day Demand

The maximum day domestic demand condition, equaling 1,357 gpm or 1.95 MGD, was modeled using a steady-state analysis with Well Nos. 5 and 6 operating, no booster pumps operating, the Davison Plant Reservoir out of service, and the elevated tanks set to 10 feet below its overflow elevation. The model projected pressures in the system range from 32 to 109 psi. The high- and low-pressure locations are the same as the average day demand scenario. Figure 5.03-1 displays the resulting pressure contours from this analysis.

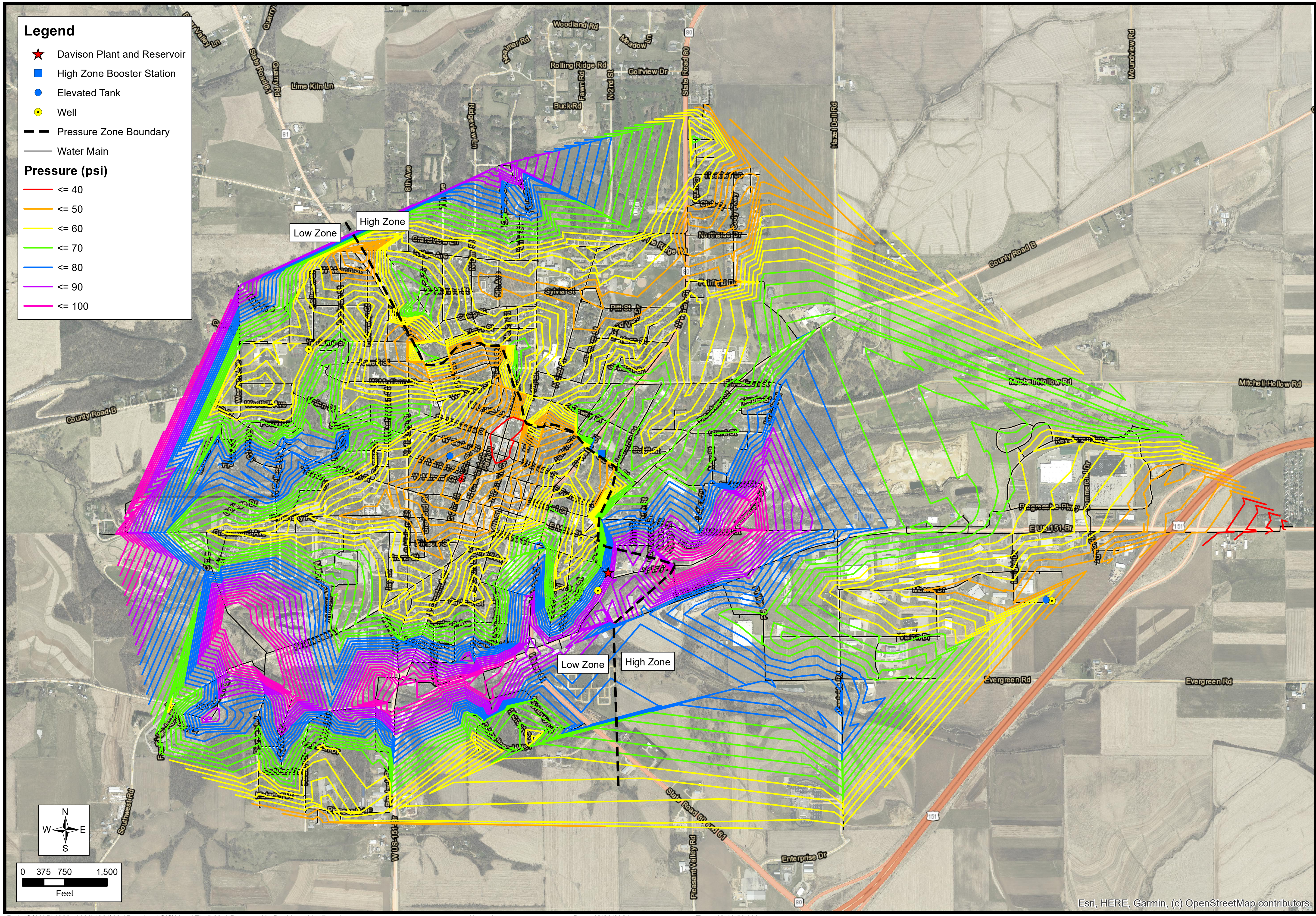
C. Steady-State Fire Flow Analysis

A steady-state fire flow analysis was completed in the model using the maximum day domestic demand condition. This modeled simulation had Well Nos. 5 and 6 operating, no booster pumps operating, the Davison Plant Reservoir out of service, and the elevated tanks set to 10 feet below its overflow elevation. The model-projected available fire flow, which was based on a 20-psi residual pressure threshold, ranged from 674 gpm to a model-controlled maximum of 5,000 gpm. Available fire flow can be anticipated to increase when additional pumps are brought into service. These fire flow values represent the amount of fire flow available at the end of the hydrant lead and do not take hydrant losses into consideration. Typically, the available fire flow will be highest near elevated storage, wells, and large diameter transmission main in the City. Figure 5.03-2 shows the model-generated available fire flow throughout the system.

The lowest available fire flow of 674 gpm is located at the end of the Jefferson Street cul-de-sac. This low fire flow is due to the hydrant being located on a long dead end with 4-inch-diameter cast iron water main in the area. The highest available fire flows of 5,000 gpm are all located in the low pressure zone, with the exception of the area surrounding the high pressure zone water tower. As depicted in Figure 5.03-2, fire flows are generally higher in the low pressure zone, which is because of a strong network of water main loops that are well-connected to the Furnace Street elevated tank. The high pressure zone fire flows are lower in comparison to the low pressure zone because the network of mains is more dispersed (less looping) and the north part of the system is further from the high pressure zone tank.

5.04 DAVISON PLANT ANALYSIS

Due to the age and condition of the infrastructure currently at the Davison Plant, the facility is recommended to be removed from service in the coming years. See Sections 2 and 6 of this report for additional details. This section evaluates how the system pressures and available fire flows compare with and without the Davison Plant in operation.



2020 MAXIMUM DAY DEMAND PRESSURE CONTOURS (NO DAVISON PLANT)

WATER SYSTEM COMPREHENSIVE PLAN
CITY OF PLATTEVILLE
GRANT COUNTY, WISCONSIN

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FIGURE 5.03-1
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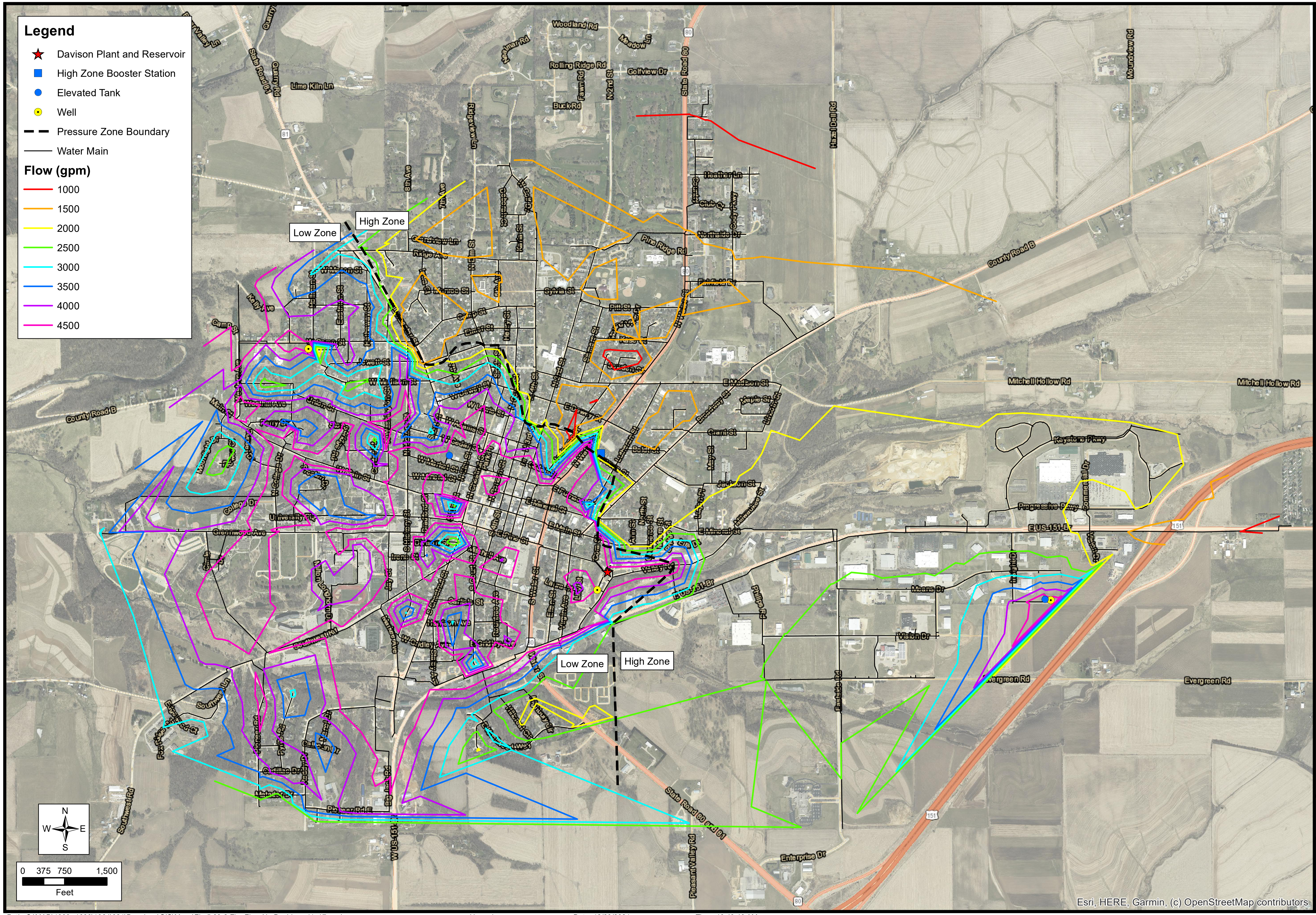
Path: S:\MAD\1000--1099\1064\021\Drawings\GIS\Maps\Fig 5.03-1 Pressure No Davidson 11x17.mxd User: danc Date: 12/20/2021 Time: 10:48:50 AM



2020 MAXIMUM DAY DEMAND FIRE FLOW CONTOURS (NO DAVISON PLANT)

WATER SYSTEM COMPREHENSIVE PLAN
CITY OF PLATTEVILLE
GRANT COUNTY, WISCONSIN

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FIGURE 5.03-2
1064.021

Section 5.03 evaluated the system with the Davison Plant Reservoir and booster pumps not in operation. For the steady-state scenarios evaluated in this section; it is assumed that the Davison Plant reservoir is set to 5 feet below overflow, Davison Booster Pump No. 1 is operating, Well No. 5 is operating, and the elevated tanks are set to 10 feet below their overflow elevations.

A. Steady-State Maximum Day Demand

The maximum day domestic demand condition, equaling 1,357 gpm or 1.95 MGD, was modeled using a steady-state analysis. The model projected pressures in the system range from 32 to 109 psi. The high- and low-pressure locations are the same as the steady state scenarios simulated in Section 5.03. Figure 5.04-1 displays the resulting pressure contours from this analysis.

B. Steady-State Fire Flow Analysis

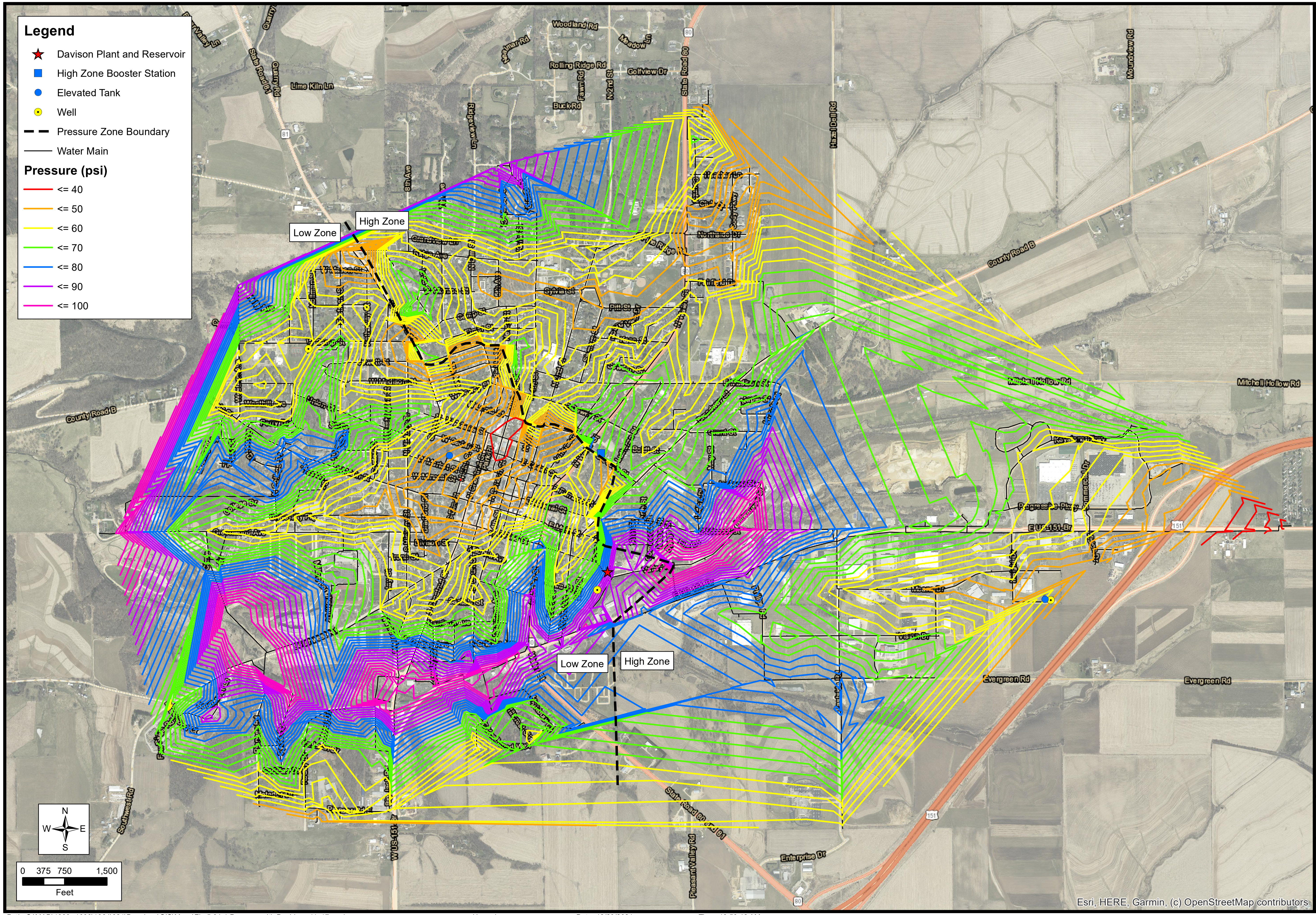
A steady-state fire flow analysis was completed in the model using the maximum day domestic demand condition. The model-projected available fire flow, which was based on a 20-psi residual pressure threshold, ranged from 674 gpm to 5,000 gpm. Areas of low fire flow in the system include hydrants located further from sources of supply and storage, on long dead ends, or on small diameter water main. Figure 5.04-2 shows the model-generated available fire flow throughout the system.

C. Comparison of Results

As shown by the pressure and fire flow contour figures in this section and Section 5.03, the system is able to sustain sufficient pressures and fire flows both with and without the Davison Plant in service. There is very little difference between supplying the low zone using Well No. 6 compared to the Davison Plant, as pressures and available fire flows are nearly identical.

The largest impact of eliminating the Davison Plant is related to the supply capacities in the low zone and the high zone. If the Davison Plant were to be decommissioned and Well No. 5 were out of service for maintenance, the high zone could still be supplied through the Stevens Street Booster Station. The same is true for the low zone. If Well No. 6 is temporarily offline, the high zone can supply the low zone through the control valve in the transfer station.

As described in the previous sections, eliminating Well No. 3 and the Davison Plant will require a new well and storage facility to satisfy long term supply and storage needs.



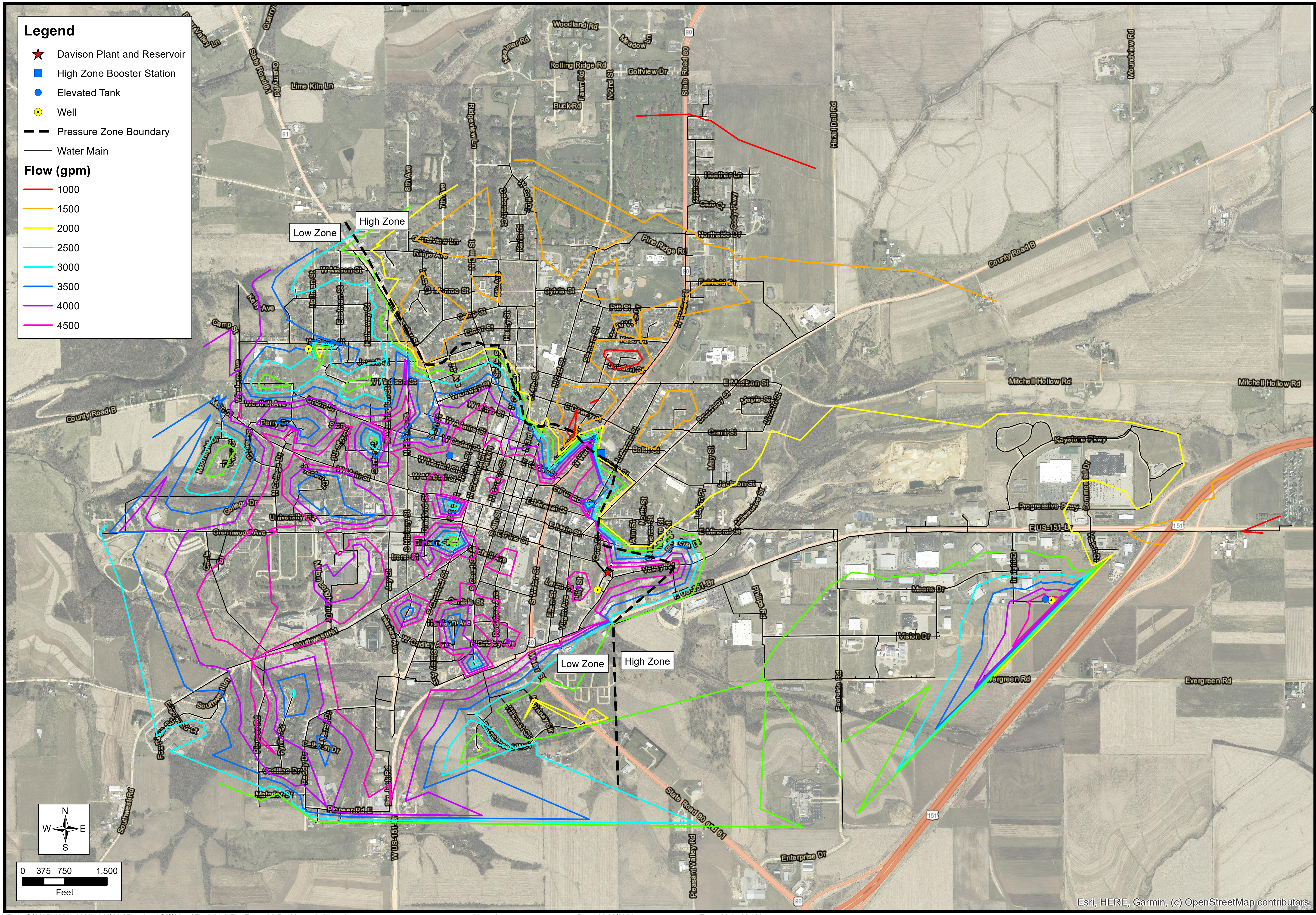
2020 MAXIMUM DAY DEMAND PRESSURE CONTOURS (WITH DAVISON PLANT)

WATER SYSTEM COMPREHENSIVE PLAN
CITY OF PLATTEVILLE
GRANT COUNTY, WISCONSIN

STRAND ASSOCIATES®

FIGURE 5.04-1
1064.021

Path: S:\MAD\1000--1099\1064\021\Drawings\GIS\Maps\Fig 5.04-1 Pressure with Davidson 11x17.mxd User: danc Date: 12/20/2021 Time: 10:50:42 AM



2020 MAXIMUM DAY DEMAND FIRE FLOW CONTOURS (WITH DAVISON PLANT)

WATER SYSTEM COMPREHENSIVE PLAN
CITY OF PLATTEVILLE
GRANT COUNTY, WISCONSIN

STRAND ASSOCIATES®

FIGURE 5.04-2
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SECTION 6
RECOMMENDATIONS AND CAPITAL IMPROVEMENT PLAN

6.01 CONCLUSIONS AND RECOMMENDATIONS

This section summarizes the conclusions and recommendations of the Water System Comprehensive Plan. A list of system improvements and anticipated costs is provided along with a discussion of implementation timing. The following OPCCs are based on 2022 dollars. Contingency amounts equivalent to 30 percent are included to cover general uncertainties, market fluctuations, professional services, and administrative costs.

Despite continued growth of the City's water service area, average day water use since 2006 has remained essentially unchanged. This trend is consistent with most other communities in the state who have seen level or declining water use trends, even as populations increase. Maximum day water use is trending upward, likely because of system maintenance, extreme weather, and ongoing expansion of the water system.

The City continues to develop its water distribution system infrastructure with future growth in mind. Annual water main replacements improve aging areas of the system, and areas of development provide opportunities to construct new water main from existing parts of the system. No areas of critical deficiency were found within the existing system that give one area of water main priority over another for replacement.

A. Water Supply and System Demands

The estimated 2022 maximum day demand is 1,357 gpm. The system's firm capacity with Well No. 5 out of service is 1,900 gpm. Under the 2022 maximum day demand scenario with Well No. 5 out of service, the system has supply surplus of 543 gpm. The estimated 2040 maximum day demand is 1,564 gpm. Assuming no additional supply has been added, the City would have a supply surplus of 393 gpm.

With Well No. 3 and the Davison Plant and reservoir removed from service, the system's firm capacity becomes 1,000 gpm. Assuming no additional supply has been added after taking the plant out of service, the City would have a present day supply deficit of 357 gpm. The deficit grows to 507 gpm by 2040 without Well No. 3. Additional well capacity is required to replace Well No. 3 and to meet the 2040 maximum day demands. A new deep aquifer well with a capacity of 1,000 gpm should be constructed before removal of Well No. 3. The timing of a new well facility depends on the desired schedule for retiring Well No. 3 and the Davison Plant.

If the City wishes to pursue drilling a new deep aquifer well, further exploration of sites can proceed at any time. If future developments proceed in areas thought to be promising for a new deep well, the City may wish to secure potential well sites before buildout of the development.

B. Storage Capacity

The City operates 900,000 gallons of elevated storage. The ground-level reservoir at the Davison Plant is 500,000 gallons. As described in Section 4, the system has an existing storage surplus of approximately 478,000 gallons and a surplus of approximately 448,000 gallons based on the 2040 design year, assuming no changes to supply or storage are made.

With the eventual decommissioning of the Davison Plant and reservoir, the system is expected to have a storage surplus of only 45,000 gallons based on the 2040 design year. Additional storage is recommended for the system, and could be added in the form of a new elevated tank or ground-level storage constructed as part of a new well facility. Because the existing system has elevated storage in each zone, ground level storage constructed as part of a new well facility is recommended.

C. Water System Modeling

As a part of this comprehensive plan, a hydraulic model of the City's water system was created, calibrated, and used to simulate system operation. No significant deficiencies were noted in the system as it pertains to service pressure and available fire flow.

The model should be maintained and updated on an annual basis to reflect water main improvements installed each year. The model can be used to simulate operation of proposed well and storage facilities as those projects proceed.

D. Well and Pumping Equipment Maintenance Plan

The historic well rehabilitation activities described earlier in the report and typical rehabilitation cycles were used to estimate ongoing well rehabilitation needs. Table 6.01-1 presents the projected time frames for rehabilitation of each well. Well rehabilitation activities are anticipated to include rehabilitation of the borehole, wire brushing, chemical treatment, and possible air impulse blasting. Well pump, column, and motor replacement may also be required during this rehabilitation process. A budgetary cost of $100,000 is assumed if well and well pump work is needed.

The wells and pumping equipment may not deteriorate at the same rate. The pumping rate, static water levels, pumping water levels, and specific capacity of each well should be monitored on a monthly basis. Declines in these key indicators of well performance are anticipated to occur over time. If the rates of decline change, the timing of the rehabilitation may need to be changed accordingly. The plan assumes that Well No. 3 will be abandoned within the next 10 years.

Well No.	Rehabilitation Years
3	2024
5	2027 and 2035
6	2028 and 2036

Table 6.01-1 Well Rehabilitation Plan

E. Elevated Tank Painting

Periodic painting of the steel elevated water storage tanks is needed to protect the structures from corrosion and prolong their useful lives. Each storage tank should be inspected before painting to assess the condition of the existing coatings and determine what scope of work will provide the most benefit to the City. In general, elevated tanks will require painting on 15- to 20-year cycles. Overcoating existing paint may be possible if the existing coatings show good adhesion. Overcoats have lifetimes in the

10- to 15-year range. Full-abrasive blasting and recoating is typically needed after overcoat systems begin to fail or if the existing coatings are not suitable to support overcoating. The capital improvement plan includes a tentative schedule and costs for tank painting.

6.02 WELL AND STORAGE ALTERNATIVES

A combination of new water supply and storage facilities will be necessary to satisfy demands and fire flow needs under present and future maximum day demand scenarios if the City were to eliminate Well No. 3 and the Davison Plant. It is recommended that the City construct a new 1,000-gpm-deep aquifer well and 400,000 gallons of storage to satisfy future demands and removal of Well No. 3 and the Davison Plant. This section discusses various alternatives available for adding well capacity and storage to the system.

A. Well Drilling

Selection of a well site is the first step in developing a new well facility. Because geology of the deep aquifer is not expected to vary significantly across the City, well sites can be selected based on existing City properties, areas of growth, and system hydraulics. Table 6.02-1 presents tasks, timelines, and estimated costs associated with each phase of the well drilling process. Well facility alternatives are discussed in the following sections.

The table includes costs related to drilling deep aquifer test wells. The cost and benefits of a test well should be considered at the start of the project. The primary advantage of a deep test well is to determine water quality. In lieu of a test well, water quality can be determined based on the production well and the well facility designed with appropriate treatment systems.

Task	Timeline for Completion (months)	OPCC
Well Site Investigation (multiple sites)	3	$20,000
Land Acquisition or Easements	1 to 3	$50,000
Test Well Drilling (if desired by the City)	8	$200,000
Test Well Drilling Design	2	---
WDNR Well Drilling Permit Review	3	---
Well Drilling	3	---
Production Well Drilling	11	$580,000
Well Drilling Design and Bidding	4	---
WDNR Well Drilling Permit Review	3	---
Well Drilling	4	---
Subtotal		$850,000
Contingency and Engineering (30 Percent)		$255,000
Total	**14 to 22**	**$1,105,000**

Table 6.02-1 Well Siting and Drilling Timelines and Costs

B. Well Facility

Table 6.02-2 outlines the tasks and estimated costs associated with a new well facility. The costs for a well facility range from $1.8 million to $3.2 million and vary based on the need for treatment. A treatment system similar to the filter installed at Well No. 5 may be needed to address iron and/or radium levels if found in significant concentrations in the new well. Implementation time is approximately two years from the beginning design to start up of the facility.

Task	Timeline for Completion	OPCC
Well Facility (No Treatment)		
Facility Design	7 months	---
WDNR Well Facility Permit Review	3 months	---
Facility Construction	12 months	---
Facility Structure		$600,000
Well Pump		$100,000
Mechanical and Piping		$250,000
Electrical and Controls		$300,000
Site Work and Restoration		$150,000
Subtotal		$1,400,000
Contingency and Engineering (30 Percent)		$420,000
Total	22 months	**$1,820,000**
Well Facility (Iron and Radium Treatment)		
Facility Design	8 months	---
WDNR Well Facility Permit Review	3 months	---
Facility Construction	14 months	
Facility Structure		$750,000
Well Pump		$100,000
Treatment Equipment		$750,000
Mechanical and Piping		$350,000
Electrical and Controls		$350,000
Site Work and Restoration		$150,000
Subtotal		$2,450,000
Contingency and Engineering (30 Percent)		$735,000
Total	**25 months**	**$3,185,000**

Table 6.02-2 Well Facility Timelines and Costs

C. Storage Alternatives

Water storage can be added in the form of an elevated tank or ground-level reservoir and pumping station.

1. Elevated Storage

Table 6.02-3 summarizes the OPCC for a new 400,000-gallon elevated tank. The OPCC includes the price of the tank, cathodic protection, piping and valves, HVAC and electrical work, site work, site acquisition, and contingency and professional services. The OPCC assumes the tank will be constructed using a standard shallow foundation. Upon preliminary design of an elevated tank, it is recommended the City hire a geotechnical engineer to determine the compatibility of the soils under the tank. If a deep foundation is required, additional project costs would be incurred. It is also recommended to verify the cost of site acquisition to determine the actual price before moving forward with tank design for budgeting purposes.

Advantages of elevated storage include reliability in the event of power loss and ease of operations. Disadvantages include the cost of periodic repainting which makes the life cycle cost higher than the cost of operating a concrete ground-level storage tank.

Task	Timeline for Completion (months)	OPCC
Preliminary Design and Siting	2	---
Tank Design	3	---
WDNR Permit Review	3	---
Construction	16	---
Tank Construction		$1,400,000
Cathodic Protection		$15,000
Electrical and HVAC		$60,000
Site Work		$150,000
Land Acquisition		$50,000
Subtotal		$1,675,000
Contingency and Engineering (30 Percent)		$502,500
Total	**24**	**$2,177,500**

Table 6.02-3 OPCC–400,000-Gallon Elevated Tank

2. Ground-Level Storage and Pumping

Additional storage could be constructed along with a new well facility to include a cast-in-place concrete reservoir and booster pumps. With this arrangement water would flow from the well, through treatment (if needed), and into the reservoir. Booster pumps would draw water from the reservoir and pump to the distribution system. This arrangement would be similar to the existing Well No. 3 and Davison Plant process. Table 6.02-4 provides costs that are intended to be additive to the well facility costs in Table 6.02-2.

Advantages of this option include a shorter timeline for implementation and lower cost of ownership.

Task	Timeline for Completion (months)	OPCC
Reservoir and Pump Design	3	---
WDNR Permit Review	3	---
Construction	9	---
Reservoir Construction		$1,200,000
Pumps and Pumping Station		$250,000
Electrical and HVAC		$100,000
Site Work		$50,000
Subtotal		$1,600,000
Contingency and Engineering (30 Percent)		$480,000
Total	**24**	**$2,080,000**

Table 6.02-4 OPCC–Ground-Level Storage and Pumping

6.03 CAPITAL IMPROVEMENT PLAN

This section provides a tentative schedule and OPCCs for improvements discussed in the report. Table 6.03-1 presents a list of projects including capital projects and major maintenance projects including tank repainting and well rehabilitation.

All costs are based on 2022 dollars and include 30 percent contingency as detailed above. The OPCC should be evaluated before implementing each project as needed to refine the project budgets.

Estimated Date	Recommendation	OPCC
2023	High Zone Elevated Tank Painting	$500,000
2023	New Well (Well No. 7) Well Site Investigation	$20,000
2024	Well No. 3 Rehabilitation	$100,000
2024	Well No. 7 Well Drilling	$1,085,000
2026	Well No. 7 Well Facility Construction	$1.8 million to $3.2 million
2026	Construct Storage Reservoir and Booster Pumps as part of Well No. 7 Facility (if desired)	$2.1 million
2027	Decommission Well No. 3 and Davison Plant	$100,000
2027	Well No. 5 Well Rehabilitation	$100,000
2028	Construction 400,000-Gallon Elevated Tank (if storage not constructed with Well No. 7 facility)	$2.2 million
2028	Well No. 6 Well Rehabilitation	$100,000

Table 6.03-1 10-Year Capital Improvement Plan

APPENDIX
HISTORIC WATER PUMPAGE AND SALES DATA

HISTORIC WATER PUMPAGE AND SALES DATA

Year	Annual Pumpage (gal)	Average Day Pumpage (gpd)	Maximum Day Pumpage (gpd)	Average Day Sales (gpd)	Sales to Pumpage Ratio	Maximum to Average Day Ratio	Revenue Water (gal)	Non-Revenue Water (gal)	Non-Revenue Water (gpd)
2006	311,942,000	854,051	1,498,000	726,667	0.85	1.75	265,415,000	46,527,000	127,471
2007	327,294,000	896,082	1,756,000	747,567	0.83	1.96	273,049,000	54,245,000	148,616
2008	338,379,000	926,431	1,360,000	752,474	0.81	1.47	274,841,000	63,538,000	174,077
2009	290,600,000	795,619	1,161,000	695,600	0.87	1.46	254,068,000	36,532,000	100,088
2010	295,862,000	810,026	1,624,000	737,320	0.91	2.00	269,306,000	26,556,000	72,756
2011	289,025,000	791,307	1,532,000	688,183	0.87	1.94	251,359,000	37,666,000	103,195
2012	304,905,000	834,784	1,641,000	695,691	0.83	1.97	254,101,000	50,804,000	139,189
2013	278,173,000	761,596	1,089,000	740,912	0.97	1.43	270,618,000	7,555,000	20,699
2014	283,978,000	777,489	1,411,000	662,694	0.85	1.81	242,049,000	41,929,000	114,874
2015	287,385,000	786,817	1,429,000	670,820	0.85	1.82	245,017,000	42,368,000	116,077
2016	289,988,000	793,944	1,203,000	708,003	0.89	1.52	258,598,000	31,390,000	86,000
2017	270,127,000	739,567	1,080,000	713,711	0.97	1.46	260,683,000	9,444,000	25,874
2018	275,387,000	753,969	1,829,000	716,586	0.95	2.43	261,733,000	13,654,000	37,408
2019	285,717,000	782,251	2,400,000	742,628	0.95	3.07	271,245,000	14,472,000	39,649

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