Long-Term Sustainability Plan

City of Decatur Water Utility

May 2021

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CDM Smith

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Executive Summary

The City of Decatur Public Works Department Water Production Division and Water Services Division (Water Utility) jointly provide potable water to the residents of the City of Decatur and to the Village of Mt. Zion. The Water Utility has provided high-quality water from Lake Decatur for approximately 150 years.

The Water Utility is regulated by the United States Environmental Protection Agency (EPA) and Illinois Environmental Protection Agency (IEPA) under the authority of the Safe Drinking Water Act (SDWA). The South Water Treatment Plant (SWTP) provides high quality water meeting all requirements of the SWDA and other regulatory requirements. The SWTP treatment processes consist of coagulation, flocculation, sedimentation, dual media filtration, anion exchange for seasonal nitrate removal, and disinfection. In addition to the South Water Treatment Plant, the Water Utility operates and maintains the Raw Water Pump Station, South Booster Pump Station, William Street Pump Station, Dewitt County well field, three elevated water storage tanks, Lake Decatur (including the dam), lime residuals storage lagoons, 30,974 meters, over 10,000 isolation valves, over 4,000 fire hydrants as well as distribution and transmission mains.

The Water Utility serves a population of approximately 79,000 customers located inside and outside of the City limits.

ES.1 Water System Long Term Sustainability Plan Objectives

The Water Utility commissioned CDM Smith in December 2019 to complete a water system long term sustainability plan with the following objectives:

1. Evaluate the existing condition of existing Water Utility assets.
2. Perform a water quality review of current and proposed potential future regulations known at this time.
3. Using the City's calibrated hydraulic model, conduct hydraulic model simulations to evaluate pressure, fire flows, water age and chlorine residual concentrations under projected 2020 water demand scenarios
4. Evaluate the water distribution system and develop a water main replacement and rehabilitation program.
5. Evaluate different replacement and rehabilitation technologies that can be used by the Water Utility.
6. Evaluate water system storage capacity and identify system optimization opportunities.
7. Review operational cost, water usage and debt payments to develop a water system capital improvement plan over a 30-year planning horizon that projects the long-term revenue requirements.

ES.2 Water System Overview, Capacity and Demand

The Water Utility operates and maintains source water, a water treatment plant, pumping and storage facilities, and distribution assets. An overview of the Water Utility's system components and capacities are summarized in **Table ES-1** below.

Table ES-1. Overview of the Water Utility's Water Facilities

Component	Capacity
Dewitt County Well Field	630 MGY
Lake Decatur Storage Capacity	20,000 ac-ft
Intake Pipeline	42-inch diameter (Two Pipelines)
Raw Water Pump Station	40.5 MGD firm / 58.5 MGD total
Raw Water Transmission Main to SWPT	54-inch diameter (Single Pipeline)
South Water Treatment Plant (SWTP)	36 MGD nominal
High Service Pump Station	49.2 MGD firm / 52.4 MGD total
South Water Treatment Plant Reservoir	4.25 MG active / 7.5 MG total
South Zone Booster Pump Station	1.44 MGD firm / 2.88 MGD total
William Street Pump Station	12.1 MGD firm / 21.5 MGD total
William Street Reservoir	2.33 MG active / 5 MGD total
Garfield Avenue Elevated Storage Tank	1.5 MG
Division Street Elevated Storage Tank	1.0 MG
Franklin Elevated Storage Tank	1.0 MG

The Water Utility commissioned a separate study in 2018 for INTERA to evaluate water demands through the year 2050. These demands are summarized in **Table ES-2** and detailed in Section 3 of this report.

Table ES-2. Projected System Water Demand (in Million Gallons) by Scenario/Year

Year	Average Day	Maximum Day	Peak Hour	Minimum Day
2020	18.6	23.4	29.3	14.1
2030	18.5	23.3	29.1	14.1
2040	18.4	23.1	28.9	14.0
2050	18.3	23.0	28.8	13.9

As shown above, a slight decrease in average day consumption is anticipated over the planning horizon.

ES.3 Long Term Sustainability Plan Findings and Recommendations

The Water Utility has established performance criteria for the water system that is based on regulatory and non-regulatory requirements. The Water Utility currently meets current regulatory water quality criteria but is anticipating potential future changes to regulations. The Water Utility is also planning for replacement of aging infrastructure and necessary upgrades that were identified as part of the condition assessment performed by CDM Smith as well as other studies that developed various recommendations for source water (INTERA and

North Water Consulting), distribution system water age and water quality (Strand), and instrumentation and control (Concentric).

CDM Smith developed performance criteria to categorize and prioritize system improvements throughout all of the assets of the Water Utility. CDM Smith conducted the following assessments:

- Interviewed operations and maintenance staff to understand systems and equipment that need attention or require replacement.
- Conducted condition assessment of water facilities, including pump stations, treatment plant, water sources and storage facilities. The assessment is based on non-destructive visual examination, conversations with operation and maintenance staff, or review of existing maintenance data and/or reports. Recommended improvements were identified.
- Performed pump testing by evaluating flowrate, total dynamic head, and power (where feasible) of operating pumps to evaluate any differences or deviations from the original manufacturer's head-capacity curves and identify any opportunities for improvement.
- Reviewed operational and maintenance records.
- Completed a risk assessment of pipes in the distribution system to categorize pipes from high to low risk.
- Completed an analysis of the system's water storage capacity, and evaluated opportunities for system optimization

The results of the condition assessment summarized, and approximate pricing (in 2021 dollars) developed along with project identification.

Figure ES-1 provides a summary of the CIP projects that were identified which include improvements to source water, the water treatment plant, pumping and storage facilities, and distribution system.

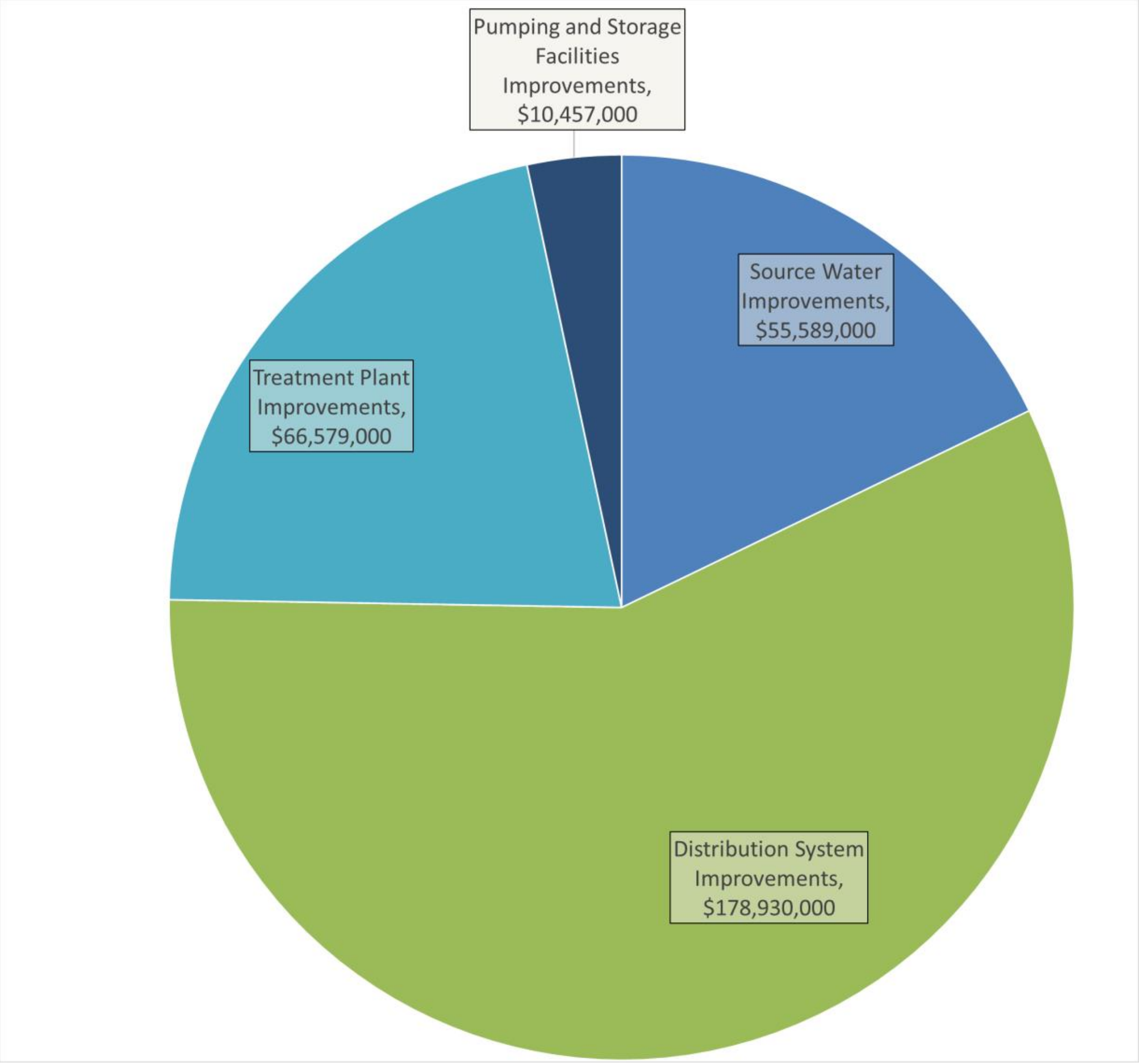

Figure ES-1. Capital Improvement Plan Project Summary Overview for Planning Horizon for Source Water, WTP, Pumping and Storage Facilities, and Water Distribution System Improvements

The results were then input into a 30-year capital improvements plan (CIP) with the goal of scheduling water system improvements projects from years 2021 to 2050. Projects were scheduled in order to try to equalize annual expenditures. The following highlight key points for the CIP (in 2021 dollars):

- The budget for projects to be completed in the immediate future, beginning as early as 2021 and completed in 2022, is approximately $21,000,000.
- The budget for short-term improvements, beginning 2023 and completed in 2027, is approximately $52,000,000.
- The budget for long-term improvements, beginning as early as 2028, is approximately $240,000,000.

The CIP projects at Water Utility facilities are identified individually in **Appendix N,** and compiled into project summaries in **Appendix A.** Project summaries for distribution pipeline identified based on medium to high risk, and fire flow improvements are included in **Appendix L** and **Appendix M.**

ES.4 Financial Impact of the CIP

The CIP projects were developed using January 2021 dollars and were compiled into a financial model to understand the financial impact of the CIP projects on the Water Utility's revenue requirements for a 30-year planning horizon starting in the year 2021. The projects were assumed to occur in specific years and to either be funded with General Obligation (GO) Bonds or cash. Using a mixture of GO Bonds and cash funded capital, the following total debt and expenditures were developed as shown in **Table ES-3.**

Table ES-3. Debt Service and Capital Expenditures

	2021	2025	2030	2035	2040	2045	2050
Existing Debt Service	$13,150,777	$11,684,756	$8,317,166	$4,681,313	$0	$0	$0
Anticipated Debt Service – GO Bonds	$174,400	$3,539,640	$10,038,985	$14,757,053	$20,450,846	$25,345,316	$29,869,127
Cash Funded Capital	$7,333,331	$1,725,776	$1,447,508	$1,785,358	$2,205,871	$2,719,666	$3,358,788
Total Debt Service and Capital Expenditures	**$20,658,508**	**$16,950,172**	**$19,803,659**	**$21,223,724**	**$22,656,717**	**$28,064,982**	**$33,227,915**

Table ES-3 shows that total debt service and capital expenditures are projected to be approximately $20.7M in 2021 and grow to approximately $33.2M by 2050. Meanwhile, existing debt service will be paid down during this timeframe until that debt is paid off.

After analyzing the debt service, the financial model predicts the following revenue requirement over the 30-year planning horizon. After approximately 2041, the capital expenditures continue to exceed the revenue requirements which include the Water Utility's annual 2.5% rate increase as shown in **Figure ES-2**. The majority of the identified CIP projects for facility improvements occur within the first 10 years of this Long-Term Sustainability Plan, as improvements within this time period are more reliably predicted.

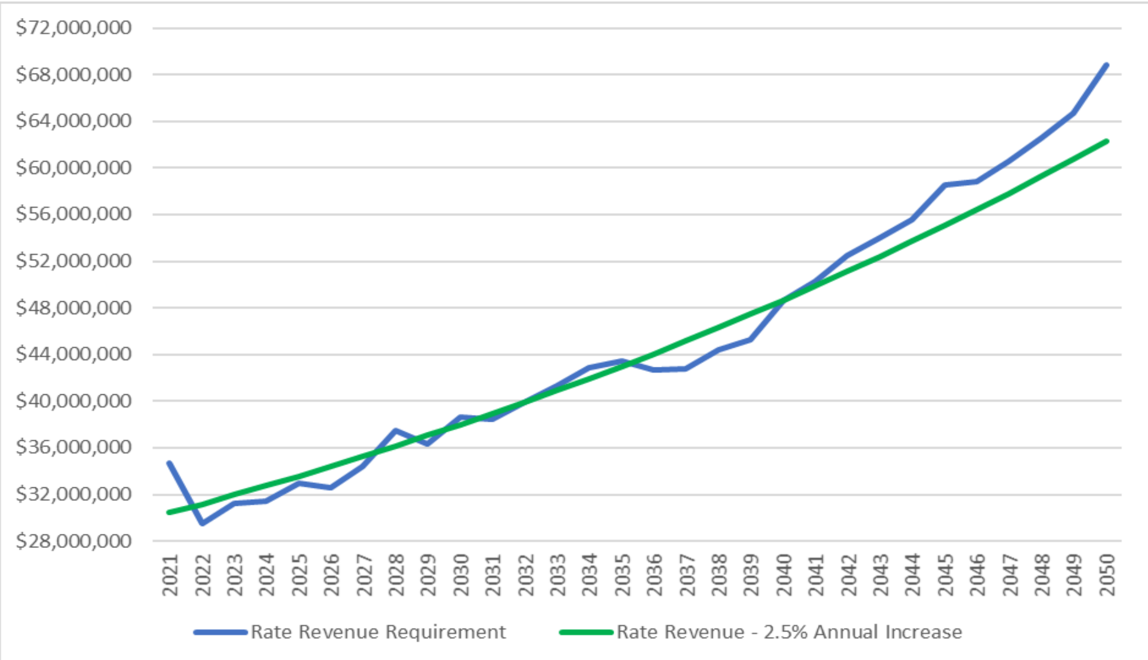

Figure ES-2. Revenue Requirement and Rate Revenue with 2.5% Annual Rate Increase

It is recommended that the Water Utility perform future planning and condition assessments in 5 to10 year cycles as the condition of equipment, facility envelopes, and other Water Utility assets will experience a level of deterioration over time, regulations may change, and other factors unknown at this time may materialize. By maintaining a cycle for evaluating and re-prioritizing projects, this will aid the Water Utility in maintaining a robust and planned capital improvement program.

Financial analysis should be updated at that time to determine if either re-prioritizing projects, or future rate increases are the best option. Either way, it is not anticipated that existing cash balances will be able to cover the anticipated revenue shortfalls. The City may also consider grants to close the gap between revenues and expenses.

The Water Utility operations and maintenance staff are highly engaged professionals, and with the appropriate resources and equipment, and continuous improvement, they will continue to provide high quality drinking water for many years to come.

Section 1

Introduction

This section provides an introduction to the Water Utility's Long-Term Sustainability Plan.

1.1 Overview

The Long-Term Sustainability Plan incorporates data gathered from numerous sources provided by the Water Utility, work performed by CDM Smith, and dialogue with Water Utility Staff to condense this information into a single document. The specific goals of the plan are outlined in this Section.

1.2 Water System Long-Term Sustainability Plan Objectives

The Water Utility commissioned CDM Smith in December 2019 to complete a water system long-term sustainability plan with the following study objectives:

- Conduct a condition assessment of water facilities, including intake, pump stations, treatment plant, dam, water sources and storage facilities.
- Interview staff at the water plant and identify needs and opportunities for improvement.
- Evaluate the treatment process and identify opportunities for optimization.
- Update and validate the distribution system hydraulic model
- Conduct hydraulic model simulations to evaluate pressure, fire flows, water age and chlorine residual concentrations under projected 2020 water demand scenarios
- Identify improvements based upon established level of service. Conduct pump testing at all pumping stations, including capacity and efficiency testing. Compare pumps to their original curves and identify opportunities for improvement.
- Evaluate the water distribution system and develop a water main replacement and rehabilitation program. Evaluate different replacement and rehabilitation technologies that can be used by the Water Utility.
- Evaluate water system storage capacity and identify system optimization opportunities.
- Review water system cyber security and provide preliminary recommendations.
- Develop a water system capital improvement plan over the 20-year planning horizon.

This report summarizes the results of these studies.

1.3 Report Organization

- Executive Summary
- Section 1 – Introduction

- Section 2 – Overview of Existing Infrastructure
- Section 3 – Water Supply and Demand
- Section 4 – Water Quality and Regulatory Review
- Section 5 – Water Treatment Plant, Storage and Pump Station Condition Assessment
- Section 6 – Distribution System Evaluation and Modeling and Storage and Pumping Optimization
- Section 7 – Distribution System Risk Assessment
- Section 8 – Water Infrastructure Capital Improvements Plan
- Section 9 – Financial Assessment
- Appendices

1.4 Abbreviations

A – Ampere

ADM – Archer Daniels Midland Company

CEC – Chemicals of Emerging Concern

CIP – Capital Improvement Plan

CoF – Consequence Upon Failure

D/DBPR – Disinfectants and Disinfection By-Products Rule

EDC – Endocrine Disrupting Compounds

EPA – United States Environmental Protection Agency

FBRR – Filter Backwash and Recycling Rule

GIS – Geographic Information System

GPD – Gallons per Day

GPM – Gallons per Minute

HAA – Haloacetic Acid

HCF – Hundred cubic feet

HGL – Hydraulic Grade Line

IESWTR – Interim Enhanced Surface Water Treatment Rule

IEPA – Illinois Environmental Protection Agency

IDNR – Illinois Department of Natural Resources

ISO – Insurance Services Office

LCR – Lead and Copper Rule

MCC – Motor Control Center

MCL – Maximum Contaminant Levels

MCLG - Maximum Contaminant Level Goals

MG – Million Gallons

MGD – Million gallons per Day

MV – Medium Voltage

N – Nitrogen

NBFU – National Board of Fire Underwriters

NPDWR – National Primary Drinking Water Regulations

NRF – Nitrate Removal Facility

NWTP – North Water Treatment Plant

PAC – Powdered Activated Carbon

PFAS – Perfluoroalkyl Substances

PFOA – Perfluorooctanoic Acid

PFOS – Perfluorooctanesulfonic Acid

ppb – Parts per Billion

ppt – Parts per Trillion

PPC – Public Protection Classification

psi – Pounds per Square Inch

PVC – Polyvinyl Chloride

RWPS – Raw Water Pump Station

SCADA – Supervisory Control and Data Acquisition

SDWA – Safe Drinking Water Act

SWTP – South Water Treatment Plant

SZBS – South Zone Booster Station

T&L – Tate & Lyle

TOC – Total Organic Carbon

TTHM – Total Trihalomethanes

UCMR – Unregulated Contaminants Rule

Water Utility – City of Decatur Water Utility

YR – Year

Section 2

Overview of Existing Infrastructure

This section provides an overview of the City of Decatur Water Utility's existing treatment and distribution infrastructure.

2.1 Overview

The City of Decatur Water Utility (Water Utility) treats source water that is drawn primarily from Lake Decatur through softening, filtration, and disinfection processes. Treated water is supplied to customers through a two-zone distribution system consisting of 536 miles of water mains, multiple storage tanks, and booster pump stations. Customers include the City of Decatur (population of approximately 72,000), two major industrial water users (Archer Daniels Midland Company [ADM] and Tate and Lyle), three adjacent wholesale customer communities: Village of Mt. Zion, Village of Harristown, and the Long Creek Township Water Department. Mt. Zion is the only wholesale customer community that relies fully on the Water Utility. Long Creek and Harristown only have emergency interconnections with the City of Decatur distribution system.

Located within the distribution system, the booster pump stations, and elevated storage tanks provide water with adequate pressure and quality. Infrastructure belonging to the Water Utility includes the following:

- Raw Water Supply and Intake Facilities
    1. Lake Decatur
    2. Lake Decatur Dam & Bascule Gates
    3. Lake Decatur Intake
    4. Dewitt County Well Field
    5. Raw Water Pump Station (RWPS)
    6. Raw Water Transmission Main
- Treatment Facilities and On-Site Storage
    1. South Water Treatment Plant (SWTP)
    2. Nitrate Reduction Building and Process
    3. SWTP Reservoir - 7.5MG
    4. High Service Pump Station
    5. Lime residual storage lagoons and return water pump station

- Distribution Facilities, System Storage, and Booster Pump Stations
  1. South Booster Pump Station
  2. William Street Reservoir and Pumping Station
  3. Garfield Elevated Tank
  4. Division Elevated Tank
  5. Franklin Elevated Tank
- Pipelines and Appurtenances
  1. Transmission Mains
  2. Distribution Mains
  3. Isolation valves, meters, and hydrants

2.2 Raw Water Supply and Intake Facilities

This section provides an overview of available raw water supplies and intake facilities.

2.2.1 Lake Decatur

Lake Decatur serves as the main water source for the SWTP and is fed by the Sangamon River. The Lake was constructed on the Sangamon River as a public water supply reservoir from 1920 to 1923 by installing the Lake Decatur Dam and developing six large basins as shown in **Figure 2-1** that are separated by the following five crossings: Reas Bridge Road, William Street (Illinois 105), US 36/Illinois 121, Lost Bridge Road, and US 51/Illinois 105.

Lake Decatur Dam consists of Bascule gates and two sluice gates. Operation of the Bascule gates is based off the elevation of the lake surface. Lake Decatur water surface readings are collected from two lake level stations located near the dam which measure the elevation of the Lake Decatur water surface elevation. In addition, operation staff have three tributary gauging stations at the following locations: Sangamon River in Monticello, IL; Long Creek in Mt. Zion, IL, and Friends Creek in Argenta, IL. Using radio telemetry, operations staff convert water surface elevations to flowrates for operators to have a general understanding of how much flow is entering and leaving the lake to maintain adequate storage upstream of the dam. Based on this information, operation staff can manually raise or lower the height of the Bascule gates to maintain adequate storage during normal operating conditions.

In 2019 the lake level was 614.3 feet, which translates to approximately 9.6 billion gallons of water storage. Lake Decatur has been be supplemented as needed by additional water sources (as identified below) in times of drought.

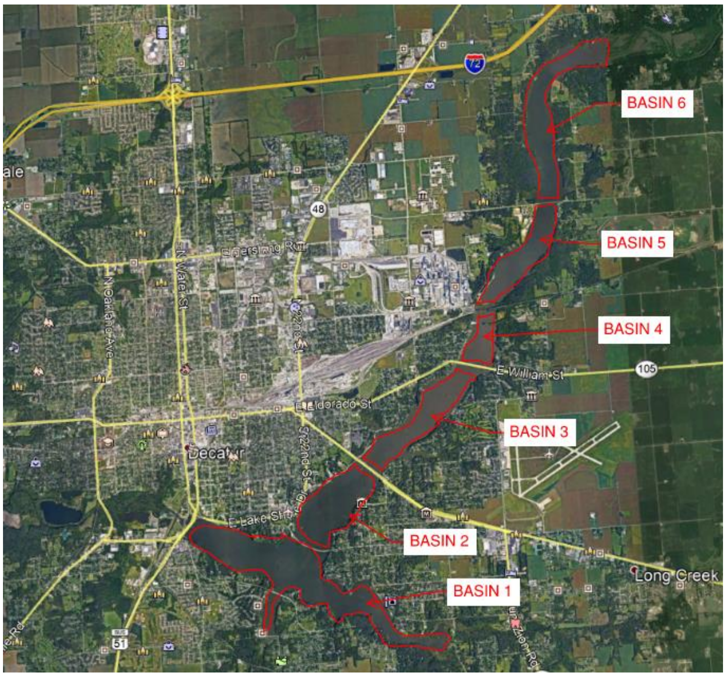

Figure 2-1. Lake Decatur

Since its original construction, natural and human caused erosion within the watershed of the Sangamon River has deposited sediment within the lake that has reduced the lake's available storage volume. As the Sangamon River continues to feed the lake, sediment continues to collect along the bottom of Lake Decatur claiming available water storage volume. The Water Utility required intermittent dredging to reclaim lost storage volume as follows:

- Basin 5 was dredged in 1993 to 1994
- Basin 6 was dredged in 2004 to 2011.
- Basins 1 to 4 were dredged in 2014 to 2019

Agricultural pollution and sediment transport (due to erosion) are two of the primary pathways of pollution loading on the Lake Decatur. In 2020, the Water Utility signed a contract with Northwater Consulting Inc. of Springfield, IL to develop a Phase I Watershed Management Plan. This program will develop a baseline assessment, and a five-year strategy to identify specific measures to reduce pollution and sediment transport to Lake Decatur within the watershed. Actions performed by stakeholders involved in the plan are anticipated to result in ultimately reducing the frequency the Water Utility is required to dredge Lake Decatur to reclaim lost storage volume by better managing the 925 square miles of watershed serving to recharge the lake.

ADM withdraws raw water from Lake Decatur for non-potable uses, which they treat at the North Water Treatment Plant (NWTP). ADM purchased the NWTP from the Water Utility in 2000 (INTERA, 2019). A 2019 report by INTERA on additional water supplies notes that the NWTP cannot pump water from the lake below an elevation of 607 feet. Therefore, dredging below this elevation does not provide benefit to this plant. ADM is allowed to withdraw up to 24 MGD of water from Lake Decatur during non-drought conditions. During droughts, withdraw rates are controlled by the Water Utility (City of Decatur, 2013).

2.2.2 Additional Water Sources

The Dewitt County wellfield is located in the Mahomet Aquifer, and is used as a supplemental water source in emergency situations. The wellfield was constructed in response to the drought of 1988 (INTERA, 2019). The wellfield consists of 8 diesel powered vertical turbine pumps drilled to depths up to 337-feet below ground level into the Mahomet Aquifer. A maximum withdrawal limit of 10 MGD exists for continuous withdrawal conditions, but it may be reduced based on the impact of using this source on nearby local private wells. This wellfield pumps water from the Mahomet Aquifer, through Friends Creek, into the Sangamon River, and eventually into Lake Decatur. Use of this wellfield during drought conditions has resulted in complaints from other well owners as to the hydraulic interference caused when these pumps are operated.

Cisco well is also located in the Mahomet Aquifer, but has not been used since 2012. The well is located along the Sangamon River south of Cisco, IL. It has been noted that this well potentially has hydraulic connections to the Sangamon River resulting in it not being a good alternate source of water during drought conditions.

The Vulcan Gravel Pit, also known as Vulcan Lake, was formerly operated by Vulcan Materials. This pit is located in the Sangamon River alluvial aquifer. It is owned by the Water Utility and has a maximum withdrawal limit of 4.3 MGD. The withdrawal limit is based on the operational ability of the floating pump station connecting the lake to the Water Utility's water treatment facilities. The use of the Vulcan Gravel Pit is further limited by a shared pump discharge pipe. The lime sludge lagoon's decant return flows through the same line and flow from both sources cannot take place simultaneously. When full, Intera estimates that the gravel pit can hold up to 250 MG of available storage (INTERA, 2019). The gravel pit is typically used either as a source for low nitrate water, or as a supplemental water source during drought.

Lake Tokorozawa and Lake Charles Rhodes (collectively called Lake Toko) have approximately 900 MG available. Intera indicates the owners of the lakes, the Rhodes family, typically refers to the larger lake as Lake Charles Rhodes, and the smaller lake as Lake Tokorozawa. Intera, as well as CDM Smith, refers to the two lakes collectively as Lake Toko as they are connected hydraulically by a culvert. Water is supplied to Lake Toko from the Sangamon River as well as by shallow groundwater. The lake has been temporarily permitted by the IEPA to be used by the Water Utility as a drought supply in the past. The lake is currently not equipped as a permanent supply and requires temporary piping to the SWTP.

The Vulcan gravel pit and Lake Tokorozawa are located downstream of the Lake Decatur dam, while the Dewitt County wellfield is located upstream of Lake Decatur to the north of Argenta, Illinois.

2.2.3 Raw Water Transmission Facilities

The raw water pump station is located along the shore of Lake Decatur. The pump station is fed from an intake located northwest of the Lake Decatur dam. The primary intake includes six 30-inch diameter pipes located in a headwall complete with sluice gates and trash racks. Water that enters the intakes flows through a 54-inch intake pipeline to the raw water pump station. At the raw water pump station, chlorine dioxide is added for oxidation and initial disinfection. The water is then pumped to the SWTP through a 54-inch raw water transmission main.

2.3 Treatment Facilities

This section provides an overview of water treatment facilities.

2.3.1 South Water Treatment Plant

The North Water Treatment Plant (NWTP) was originally used to provide potable water to the City. However, following the construction of the South Water Treatment Plant (SWTP) in the late 1980s, the NWTP was purchased from the City by ADM in 2000 which now withdraws and treats water for non-potable process use. Both the SWTP and NWTP continue to draw surface water from Lake Decatur.

The SWTP has a 36 MGD nominal capacity. Inflow to the SWTP is pumped directly from the raw water pump station. At the plant, powdered activated carbon (PAC) is added for control of taste and odor compounds and other organics before the water is separated into two parallel trains of lime softening and clarification treatment in order to remove hardness, total organic carbon (TOC), and turbidity. Each softening train consists of a primary and secondary basin. Alum and lime are added in the primary basins for coagulation and softening. Settled sludge is removed from each basin and pumped to the lime sludge storage lagoons. Polymer and carbon can be added in the secondary basins to improve settling and for additional organics control, respectively.

Recarbonation with carbon dioxide is performed after softening in order to lower pH and prevent precipitation of calcium carbonate in downstream processes. Carbon dioxide, sodium hexametaphosphate, fluoride, and chlorine are also added to the effluent of the softening trains. Fluoride is added per IEPA requirements to reduce tooth decay. Sodium hexametaphosphate is added to sequester carbon and further reduce precipitation of calcium carbonate.

The SWTP has six-dual media filters used to further treat softened water. The filters contain 22 inches of anthracite over 14 inches of sand.

2.3.2 Nitrate Reduction Facility

The Nitrate Removal Facility (NRF) was constructed and began operation in June 2002. The facility is located on the property of the SWTP. The NRF is typically operated during periods of elevated nitrate-nitrogen (nitrate-N) concentrations in Lake Decatur which only occur a few months out of the year. During these periods, the SWTP treatment process cannot remove the elevated nitrate-N levels, so the NRF is activated to reduce nitrate-N below required water quality standards. Up to 16 MGD of water can be treated through the NRF, with the NRF flow rate selected as needed to decrease the finished water nitrate levels down to acceptable levels. Outside of the window of elevated nitrate, the NRF is seldom used. Additional positive impacts for the use of this facility are reduced total organic carbon and reduced disinfectant byproduct formation in the finished water.

Treated water from the NRF is blended with the rest of the filtered water from the SWTP and flows to the SWTP reservoir.

2.3.3 South Water Treatment Plant Reservoir

The SWTP reservoir is a 7.5 million gallon (MG) prestressed concrete reservoir installed in approximately 1989 with the South Water Treatment Plant. This reservoir provides chlorine contact time to meet disinfection requirements. The reservoir also serves as a supply for the high service pump station. Approximately 3.25 MG, or a depth of 13.8 feet, must be maintained in the reservoir in times of maximum demand to maintain sufficient chlorine contact time and filter backwash flow (Strand, 2008). To add a safety factor, plant staff have an in-house rule to not let the water level in the reservoir go below 20.0 feet to allow for additional filter backwash capacity. To monitor the water levels, the SCADA system has low and high target elevation range of 20 feet to 30.5 feet, respectively. If the water level is outside of this range an alarm will activate, and the reservoir elevation light will remain flashing until it is brought back into the target elevation range. Due to this requirement, the reservoir has a usable storage capacity of approximately 4.25 MG. Additional chlorine is added at the influent to the reservoir to aid in disinfection.

2.3.4 High Service Pump Station

The SWTP high service pump station (HSPS) has a total design capacity of 76.5 MGD. In 2008, Strand Associates evaluated and performed field tests on each of the five high service pumps; three pumps have a capacity of 18 MGD, one pump has a capacity of 13.5 MGD, and one pump has a capacity of 9 MGD. The firm capacity of the plant for average day conditions, assuming Pump No. 5 (18 MGD pump) was out of service, was forecasted to be 49.2 MGD in YR 2020 (Strand, 2008). Strand Associates noted that the total high service capacity exceeds SWTP treatment capacity, and that the HSPS can use the SWTP reservoir for short periods of time to meet high hourly demand conditions (Strand, 2015). The high service pump station pumps directly from the SWTP Reservoir.

2.4 Distribution Facilities

The distribution system is separated into two pressure zones, the North Zone and the South Zone. Both zones are served by the high service pump station located at the SWTP.

The distribution system is made up of 536 miles of water mains. Water main diameters range from 4-inches to 48-inches, and pipe materials include cast iron, ductile iron, and polyvinyl chloride (PVC). Pressure is maintained throughout the system through the use of booster pump stations and elevated storage tanks

2.4.1 William Street Reservoir and Pumping Station

The William Street reservoir and pumping station serves the North Zone of the distribution system. The reservoir holds 5 MG of finished water. Three pumps, with capacities of 4.9 MGD, 4.9 MGD, and 2.3 MGD, deliver water from the William Street ground level reservoir to the distribution system.

2.4.2 Garfield Elevated Tank

The Garfield elevated tank serves the North Zone of the distribution system. It helps to maintain pressure in the system and provide supplemental water to meet demands. The tank was designed to store 1.5 MG and was built in 1950.

2.4.3 Division Street Elevated Tank

The Division Street elevated tank also serves the North Zone of the distribution system and helps to maintain pressure in the system and provide stored water to meet demands. The Division Street elevated tank was designed to store 1.0 MG and was built in 1957.

2.4.4 South Zone Booster Pump Station

The South Zone of the distribution system is served by the South Zone booster pump station from the North Zone. Two pumps, both rated at 1,000 gpm, supply the South Zone based on the water level in the Franklin elevated storage tank.

2.4.5 Franklin Elevated Tank

The Franklin elevated storage tank serves the South Zone of the distribution system. It helps to maintain pressure and meet water demands. The tank was designed to store 1.0 MG of finished water and was built in 2009.

Figure 2-2 provides an overview of the major assets of the Water Utility and how they are arranged throughout the system to provide high quality drinking water to all customers.

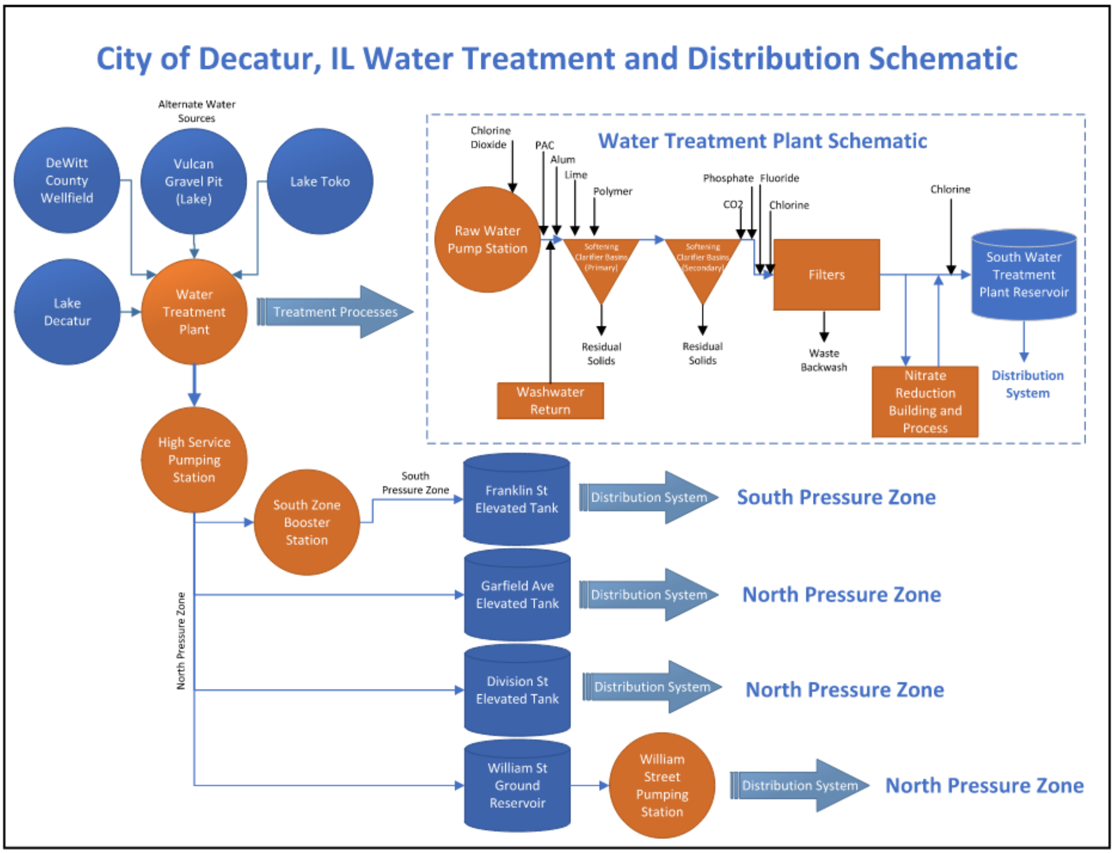

Figure 2-2. Schematic of Decatur Water Treatment and Distribution Assets

2.5 References

City of Decatur (Revised July 30, 2013). *Low Lake Level/Drought Action Plan*.

INTERA (November 2019). *Additional Water Supplies – Local Solutions to Manage Drought Risk*. Final Draft.

Strand Associates, Inc (2008). *High Service Pump Station and Water Treatment Plant Storage*.

Strand Associates, Inc (2015). *Water System Master Plan Update*.

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Section 3

Water Supply and Demand

This section provides an overview of the Water Utility's current and future anticipated water supply and demand.

3.1 Water Supply

In 2019, the Water Utility commissioned INTERA to perform a study to analyze current and future water demands and evaluate their associated impact on the Water Utility's source water. A 10-month design drought combining record drought conditions from 2012 and 1930 was used to calculate a supply gap. The supply gap is the difference between water available and water demand. Based on the estimated water demand for the year 2050, the data indicates that there is a 3.1-billion-gallon supply gap resulting from the 10-month design drought (INTERA, 2019). The study analyzed alternatives for the Water Utility to make up that supply gap. The recommended course of action to obtain supplemental water was to lower the minimum lake operating level in Lake Decatur and then to increase withdrawal rates from the DeWitt Well Fields and former Vulcan Gravel Pit. The secondary alternative the 2019 report suggested was to consider if Lake Tokorozawa would be a feasible water source to develop.

Diversifying water sources would provide more benefits than just minimizing the supply gap. Blending multiple water sources can help the Water Utility to minimize turbidity and nitrates when surface runoff affects surface water sources. Ground water can also be blended with a surface water source when temperatures fluctuate outside of optimal conditions. Overall, regular use of additional water supply options increases Decatur's resiliency and ability to manage natural and man-made hazards.

3.2 Water Demand

The City of Decatur's current and projected water demands are discussed below.

3.2.1 Future Average Daily Projection

The INTERA report includes an updated water demand forecast for the Water Utility (INTERA, 2019). The analysis evaluated historical water use at SWTP, Archer Daniels Midland (ADM), and Tate & Lyle (T&L). A statistical model was then developed to project future water demand to the year 2050 under four scenarios of weather and growth assumptions. **Table 3-1** is an excerpt from the INTERA report that summarizes the 2020 to 2050 water demand projections. Projected daily average SWTP demand ranged from 18.61 to 19.29 MGD for 2020 and 18.27 to 21.92 MGD for 2050. The analysis also projected raw water demand for ADM, but that demand is fulfilled by the NWTP that ADM purchased from the Water Utility in 2000.

The projected 2020 demands under normal weather and expected growth were used for storage analysis and system evaluation. The expected (baseline) growth scenario is based on a gradual reduced population projection to approximately 68,194 by the year 2050 while the higher growth scenario is based on a gradual increased population project to approximately 78,740 for that same timeframe as approximated from Figure 3.1 of the INTERA report (INTERA, 2019).

Table 3-1. Decatur Average Daily Water Demand Forecasts by Scenario in MGD

Condition	2020	2030	2040	2050
Normal Weather & Expected Growth				
SWTP (excluding ADM and T&L Potable)	8.20	8.08	7.96	7.86
ADM Potable	5.05	5.05	5.05	5.05
T&L Potable	5.36	5.36	5.36	5.36
SWTP Total	18.61	18.49	18.37	18.27
Normal Weather & Higher Growth				
SWTP (excluding ADM and T&L Potable)	8.25	8.53	8.82	9.12
ADM Potable	5.14	5.41	5.68	5.97
T&L Potable	5.45	5.73	6.03	6.34
SWTP Total	18.84	19.67	20.53	21.43
Drought & Expected Growth				
SWTP (excluding ADM and T&L Potable)	8.54	8.41	8.29	8.19
ADM Potable	5.15	5.15	5.15	5.15
T&L Potable	5.37	5.37	5.37	5.37
SWTP Total	19.06	18.93	18.81	18.71
Drought & Higher Growth				
SWTP (excluding ADM and T&L Potable)	8.59	8.88	9.18	9.49
ADM Potable	5.24	5.51	5.79	6.09
T&L Potable	5.46	5.74	6.03	6.34
SWTP Total	19.29	20.13	21.00	21.92

3.2.2 Maximum Day, Peak Hour and Minimum Day Projection

This section revisits the projected maximum day, peak hour of maximum day, and minimum day demands. **Figure 3-1** presents the minimum, average and maximum daily pumpage of high service pumping station by year from 2007 to 2019. The actual data were more than 20% lower than the estimates used in the 2015 Water Master Plan update (Strand, 2015). The average demand from the last 5 years (19.8 MGD) is also slightly higher than our 2020 projections ranging from 18.6 to 19.3 MGD.

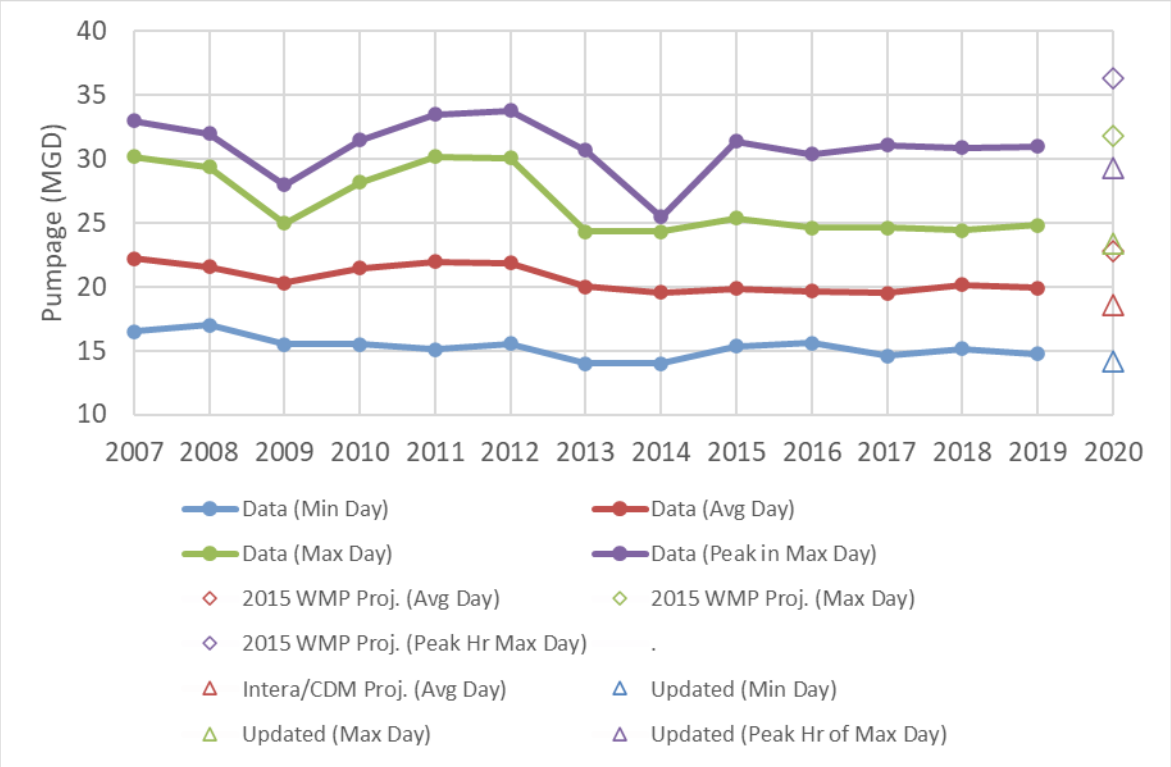

Figure 3-1. High Service Pumpage from 2007 to 2019

Using the last 5 years of high service pumpage data (2015 – 2019), peaking/dipping factors were developed to calculate minimum day, maximum day and peak hour of maximum day demands, as follows:

- Maximum day = average day x 1.26
- Peak hour = maximum day x 1.25 (or average day x 1.58)
- Minimum day = average day x 0.76

Table 3-2 summarizes the updated system demand applied to normal weather and expected growth scenario. These numbers were used for storage analysis and hydraulic model simulations.

Table 3-2. System Water Demand (in MGD) by Scenario/Year

Year	Average Day	Maximum Day	Peak Hour	Minimum Day
2020	18.6	23.4	29.3	14.1
2030	18.5	23.3	29.1	14.1
2040	18.4	23.1	28.9	14.0
2050	18.3	23.0	28.8	13.9

3.3 Summary

Depending on hydrologic conditions and growth, the 2050 system demand for the SWTP is projected to be between -2% and +10% of 2020 system demands. For the worst-case scenario, there would be a 3.1-billion-gallon supply gap resulting from the 10-month design drought over 107 days in 2050 (combined gap for both the SWTP and the NWTP). INTERA recommends that an additional 3.1 billion gallon water supply be obtained to fill this supply gap during a drought. This gap can be addressed by increasing withdrawal rates from the DeWitt Well Fields and former Vulcan Gravel Pit, and modifying Lake Tokorozawa to become a permanent water source (INTERA, 2019).

3.4 References

INTERA (November 2019). *Additional Water-Supplies: Local Solutions to Manage Drought Risk.*

Strand & Associates (March 2015). *Water System Master Plan Update.*

Section 4

Water Quality and Regulatory Review

This section provides a summary of the analyses of raw source water quality, treated water quality, and discusses current, pending, and future anticipated regulations pertaining to water supply and water quality and how they relate to the City of Decatur. This section also summarizes recommendations for the water system pertaining to trihalomethane levels, per- and polyfluoroalkyl substances (PFAS) sampling, and clarifier improvements.

4.1 Summary of Drinking Water Quality Regulations

Drinking water is federally regulated by the United States Environmental Protection Agency (EPA) under the authority of the Safe Drinking Water Act (SDWA). The SDWA was extensively amended in 1986 and in 1996 and its regulations have been adopted by the Illinois Environmental Protection Agency (IEPA) which has been given primacy by the EPA for enforcing these regulations in Illinois. The SDWA has set primary and secondary standards for contaminants in potable water systems. The National Primary Drinking Water Regulations (NPDWR) are legally enforceable standards that apply to all public water systems and consist of maximum contaminant level goals (MCLGs), which are non-enforceable goals, as well as maximum contaminant levels (MCLs). MCLs are enforceable limits set as close to the MCLGs as practical, considering cost and feasibility of attainment. National Secondary Drinking Water Regulations (NSDWR), also referred to as secondary standards, are federally non-enforceable guidelines regulating contaminants that may cause human cosmetic effects (such as skin or tooth discoloration) or aesthetic effects in drinking water (such as taste, odor, or color). Compliance with secondary standards is recommended but not enforced.

The Clean Water Act (CWA) establishes the basic structure for regulating water quality standards for surface waters and discharges of pollutants into waters of the United States. As with the SDWS, the CWA is also enforced by the IEPA.

4.1.1 Existing Regulations

The EPA has enacted several regulations since the 1996 SDWA amendments. The regulations that are particularly relevant to the Water Utility include the NPDWR Phase II Rule, Interim Enhanced Surface Water Treatment Rule (IESWTR), Stage 1 and Stage 2 Disinfectants and Disinfection By-Products Rules (D/ DBPR), revisions to the Lead and Copper Rule (LCR), Fluoride Rule, and Filter Backwash Recycling Rule (FBRR). These regulations require that water systems meet MCLs and/or use certain treatment techniques to protect against adverse health effects in regard to turbidity, primary and secondary disinfection, disinfection by-products (DBPs), corrosion by-products, fluoride, and nitrate. The regulated contaminants relevant to the Water Utility and the controlling regulations are further detailed in the Water Quality and Regulatory Review Technical Memorandum from September 2020, included as **Appendix B**.

4.1.2 Potential Future Regulations

Potential near-term changes to existing regulations and possible new regulations that are relevant to the Water Utility include tightening of filter effluent requirements, treatment of spent filter backwash water before it is returned to the head of the plant, establishing a minimum disinfectant residual throughout the distribution system, expanding the haloacetic acid (HAA) regulation to include all nine brominated and chlorinated HAAs, lowering the bromate MCL, setting MCLs for nitrosamines, and chlorate and perchlorate. Proposed revisions to the LCR have been released. If these revisions are accepted as they are currently proposed, they would impose a trigger level of 10 ppb of lead that would require action by the Water Utility and adjust other aspects of the current LCR.

Cyanotoxin formation has also been a concern over the past decade, especially in lakes and reservoirs. Ten cyanotoxins were listed in the EPA's fourth Unregulated Contaminants Monitoring Rule (UCMR 4). It is possible that the EPA will issue health advisories for several of them. A number of other chemicals of emerging concern (CECs), such as endocrine disrupting compounds (EDCs) and pharmaceutical and personal care products (PPCPs) are also the subject of much research and were also listed in UCMR4, but it is unlikely that a regulation will be proposed in the near future.

The IEPA is currently conducting a statewide investigation of per- and polyfluoroalkyl substance (PFAS) compounds in community water supplies. Once completed, the IEPA is expected to develop a future MCL for PFAS compounds. In addition, the EPA's fifth UCMR (UCMR5) includes 29 PFAS compounds that may lead to a future EPA PFAS regulation.

4.2 Raw Water Quality

The Water Utility's SWTP draws raw water directly from Lake Decatur which is fed directly by the Sangamon River. Key raw water quality data for Lake Decatur is detailed in a tabular format in **Appendix B.** Certain water quality parameters are detailed below.

4.2.1 Nitrate

Nitrate is currently the largest concern in the raw water source used by the Water Utility. Lake Decatur is fed by the Sangamon River which carries seasonally high levels of nitrate from agricultural runoff in the area. Snow melt, rain, and nitrogen fertilizer application in the watershed supplying the Sangamon River cause the nitrogen levels in Lake Decatur to peak in winter and early spring.

4.2.2 Hardness, Alkalinity, and pH

Hardness of the raw water entering the SWTP ranges from approximately 200 to above 291 milligrams per liter of calcium carbonate (mg/L as $CaCO_3$) and can be classified as very hard. The total alkalinity of the raw water is considered moderate, typically ranging from 164-231 mg/L as $CaCO_3$. Because total alkalinity is generally lower than hardness in the source water, some of the hardness is non-carbonate hardness and will not be removed in a standard lime softening process. The SWTP has the capability to feed soda ash to the softening process to add alkalinity to remove additional hardness. The hardness is approximately two-thirds calcium hardness and one-third magnesium hardness.

Typical raw water pH ranges from 7.6 to 8.3. In this range, the alkalinity is mostly in the bicarbonate and carbonate form. The pH is raised during the lime softening process, then lowered by recarbonation.

4.2.3 Cryptosporidium

The Water Utility completed their second round of LT2 ESWTR sampling for cryptosporidium from 2015 thru 2017. The average cryptosporidium was found to be 0.013 oocysts/L, corresponding to a classification of Bin 1. Bin 1 classification means that no additional cryptosporidium treatment is required of the filtered water from the SWTP.

The LT2 ESWTR required only two rounds of cryptosporidium sampling. While there currently is no requirement for the Water Utility to conduct further cryptosporidium sampling, it is conceivable that future regulations may call for additional rounds of sampling.

4.2.4 Total Organic Carbon

Total organic carbon (TOC) is naturally occurring organic material that can react with disinfectants to form disinfection byproducts (DBPs). The reported TOC levels in the raw water ranged from 3.0 to 5.8 mg/L. Under the Stage 1 DBP Rule, softening water treatment plants need to achieve a 15 percent reduction in TOC through the treatment process when the raw water TOC is between 2.0 and 4.0 mg/L. A 25 percent reduction must be achieved when the raw water TOC is between 4.0 and 8.0 mg/L.

4.2.5 Source Water Protection

In 2020, the Water Utility contracted with Northwater Consulting Inc. (NWC) to develop a Lake Decatur Watershed Management Plan. Over the course of the next few years, NWC will work with stakeholders to develop a plan to more actively plan, manage, and implement watershed management strategies as it relates to the watershed supplying the Sangamon River.

4.2.6 Additional Raw Water Sources

The Water Utility operates and maintains additional raw water sources that can be used in times of drought to supplement the raw water supply. These additional sources include the DeWitt County wellfield and the Vulcan gravel pit. In addition, the Water Utility is investigating the potential to acquire rights to use the water within Lake Charlie Rhodes and Lake Tokorozawa. Detailed information about each of these sources and the potential impact of their use on the performance of the SWTP is included in **Appendix B.**

4.3 Finished Water Quality

Finished water is delivered from the SWTP to customers in the City of Decatur and the Village of Mount Zion. The Water Utility provides safe drinking water to its customers as consumer confidence reports dating back to 2014 show no violation of EPA or IEPA drinking water regulations. A summary of the key water quality parameters shown in a tabular format is included in **Appendix B.** A discussion of select aspects of the finished water quality is included herein.

4.3.1 Hardness, Alkalinity, and pH

The SWTP uses lime softening to reduce the finished water hardness to 74 – 175 mg/L as $CaCO_3$. The alkalinity in the finished water ranges from 20-100 mg/L as $CaCO_3$. During treatment, alkalinity is removed in the softening and recarbonation processes. Hardness and alkalinity do not have any primary or secondary drinking water standards, however for corrosion control the Water Utility maintains total hardness levels at 80 mg/L or greater and alkalinity levels at 20 mg/L or greater. The Water Utility has an internal goal to maintain finished water hardness levels between 80 to 110 mg/L as $CaCO_3$ and finished water alkalinity levels greater than 20 mg/L as $CaCO_3$.

Finished water pH is consistently between 9.0 and 9.4. The pH is raised during the lime softening process and lowered by recarbonation – lowering pH is necessary to prevent excessive calcium carbonate precipitation in the downstream treatment processes and in the distribution system. For corrosion control and water stability, the Water Utility has a goal to maintain finished water pH levels between 9.0 and 9.5.

4.3.2 Nitrate

The EPA has set both a maximum contaminant level limit and a maximum contaminant level goal for nitrate at 10 mg/L as N. The average nitrate concentration of 4.5 mg/L analyzed by SWTP operations staff is below these EPA limits. Currently, the Water Utility uses ion exchange to remove nitrate from the water when levels approach the 10 mg/L limit, with a goal of keeping nitrate below 8.5 mg/L. When raw water nitrate levels are sufficiently low to keep the finished water nitrate level below the target level without treatment, the ion exchange treatment system is not used.

Some groups have called for a lowering of the nitrate standard. During its most-recent Six-Year Review of existing drinking water standards, completed in 2016, the EPA indicated that the current nitrate standard is "not appropriate for revision at this time", citing an ongoing health effects assessment. Until EPA completes this assessment and publishes its recommendations, it is unknown whether or not the current nitrate standard will be changed, and if changed, what the new proposed limit would be.

4.3.3 Cyanotoxins

Cyanotoxins pose a potential risk for any surface water susceptible to algal blooms. For the 2019 finished water sampling data provided, microcystins (detection limit 0.3 µg/L), cylindrospermopsin (detection limit 0.09 µg/L), and anatoxin-a (detection limit 0.03 µg/L) were all below method detection limits. Use of nitrogen and phosphorus fertilizers in the watershed creates a nutrient-rich agricultural runoff that enters source waters. Increasing use of these fertilizers could lead to eutrophication of Lake Decatur and increase the frequency of algal blooms and therefore cyanotoxins.

4.3.4 Lead and Copper

The EPA mandated Lead and Copper Rule (LCR) sets a lead action level of 15 µg/L (ppb) and a copper action level of 1.3 mg/L. Compliance sampling is conducted on "high risk" homes every three years. To comply with the LCR, the 90th percentile lead and copper concentrations must be below their action limit. Consumer confidence reports dating back to 2014 show no violations of the LCR within the Water Utility's water distribution system. Lead and copper are absent from water leaving the treatment plant but can enter the water through corrosion of service lines and plumbing fixtures. Water Utility distribution staff have been removing lead service lines from the distribution system for over 30 years and have removed approximately 85% of the system's ¾-inch to 1-inch services.

The proposed LCR revisions include a 10-ppb trigger level (for 90th percentile lead) and a "find-and-fix" requirement for individual lead samples above 15 ppb. The 90th percentile for lead has remained below 10 ppb in Decatur since 2008, so it is not expected that the Water Utility will have difficulty complying with the revisions as proposed. There was an individual sample collected that contained 15 ppb of lead when analyzed; in the future, the Water Utility could be required to investigate the source of any lead readings above 15 ppb and assist in remediating the cause.

4.3.5 Turbidity

Finished water turbidity is consistently kept under 0.3 NTU. High turbidities are usually associated with pathogenic microbes, so the EPA limits the turbidity to 1 NTU in all samples, and 0.3 NTU as the 95th percentile for the samples in any month for treatment plants that use conventional filtration. The Water Utility has an internal goal of maintaining filtered water turbidity lower than 0.2 NTU and have programed the SCADA system to alarm whenever the filtered water turbidity rises above 0.15 NTU. Finished water turbidity is usually below 0.10 NTU. Calcium carbonate carryover in the filtered water from lime softening is likely the cause of the higher turbidity measurements. Turbidity in the form of calcium carbonate is not a risk to public health as opposed to turbidity caused by organic materials or sediment that passes through the water treatment process. Key to achieving low filtered water turbidity is achieving low settled water turbidity. The Water Utility has an internal goal of achieving settled water turbidity levels at or below 1.5 NTU for the water leaving the second stage softeners.

4.3.6 Disinfection Byproducts and TOC

EPA drinking water standards limit the sum of the five most common haloacetic acids (HAA5) to 60 µg/L and total trihalomethanes (TTHMs) to 80 µg/L. These compounds are the byproduct of chlorine disinfectant reacting with organic carbon. Decatur is in compliance with these standards. Running annual averages were provided for eight locations from 2012 through 2019. The highest of these running averages was 23.3 µg/L for HAA's and 63.6 µg/L for TTHM's, both under their respective limits. While the annual averages have been below the limit for TTHMs, individual quarters have measured THM levels greater than 80 ug/L. Therefore, the safety factor for compliance is low. CDM Smith prepared a technical memorandum discussing strategies to reduce THMs. The findings are summarized in Section 5.4 below.

The Stage 1 Disinfection Byproduct Rule (DBPR) sets TOC removal requirements based on source water TOC and alkalinity. Lake Decatur has TOC between 3.0 and 5.8 mg/L. Alkalinity is typically above 120 mg/L as $CaCO_3$. At these levels, the Stage 1 DBPR requires 15% TOC removal during treatment. Occasionally, the source water alkalinity dips into the 60 to 120 mg/L as $CaCO_3$ range. In these rare instances, the Stage 1 DBPR requires 25% TOC removal. The water treatment plant achieved greater than 35% TOC removal (averaging 55%) from 2017 through 2019, so meeting the Stage 1 DBPR requirements is typically not a concern throughout the normal range of source water TOC and alkalinity levels.

Total organic carbon (TOC) concentration, chlorine dose, water age, and water temperature are all factors that influence the formation of disinfection byproducts (DBP). The Water Utility has commissioned Strand Associates to complete a study to consider chlorine residuals, water age and DBP formation. Initial recommendations from this study include a recommendation to install mixers within some of the finished water storage tanks at the SWTP and within the distribution system to strip some of the volatile DBPs that are formed. This may be an effective strategy to reduce TTHMs and maintain compliance with the current regulatory standard. However, there are many DBPs that are not volatile and would not be reduced by these tank mixing systems. Thus, it is still important for the SWTP to achieve good TOC removal prior to chlorine addition and to keep water age within acceptable levels within the distribution system.

4.3.7 PFAS

PFAS is a group of thousands of manmade fluorinated compounds with unique chemical and physical characteristics which make them repel oil, water and stain, act as a surfactant, tolerate high temperatures and promote friction reduction when used in the production of a wide range of industrial and consumer products such as non-stick cookware, water-repellent clothing, stain resistant fabrics and carpets, and some cosmetics. They are typically found in facilities with processes in relation to metal plating, wire coating and insulation, photolithography, textile and paper products, cookware and in numerous other commercial and household applications. They have also been used in fire-fighting foams that extinguish petroleum-based fires. Exposure to PFAS has been linked to developmental issues, increased risk of cancer, and other health concerns. So far, the US EPA has not yet established a PFAS standard, but has published a health advisory level of 70 parts per trillion (ppt) for the combined concentrations of Perfluorooctanoic Acid (PFOA) and Perfluorooctane Sulfonate (PFOS). The IEPA is currently conducting a statewide investigation of PFAS compounds in community water supplies. This investigation includes sampling the finished water at 1,456 entry points to the distribution system representing 1,749 community water supplies across Illinois. The purpose of this investigation is to support the potential development and promulgation of MCL standards in Illinois for certain PFAS compounds. In January 2021 the IEPA issued statewide Health Advisories for four PFAS compounds, as shown in **Table 4-1**.

Table 4-1. Illinois Statewide PFAS Health Advisories (January 2021)

PFAS Analyte (Acronym)	Health Advisory Level
Perfluorooctanoic acid (PFOA)	2 ppt
Perfluorohexanesulfonic acid (PFHxS)	140 ppt
Perfluorobutanesulfonic acid (PFBS)	140,000 ppt
Perfluorohexanoic acid (PFHxA)	560,000 ppt

Under the Third Unregulated Contaminant Monitoring Rule (UCMR3) sampling, the effluent from the SWTP was evaluated for six PFAS compounds as shown in **Table 4-2**. While no PFAS compounds were detected in the UCMR3 sampling, the detection limits at the time were not as sensitive as is currently available. Current analytical methods can detect a greater number of PFAS compounds that was possible at the time of the UCMR3 testing.

Table 4-2. UCMR3 PFAS Sampling Results for the SWTP

PFAS Analyte (Acronym)	Health Advisory Level
Perfluorobutanesulfonic acid (PFBS)	Less than 90 ppt
Perfluoroheptanoic acid (PFHpA)	Less than 10 ppt
Perfluorohexanesulfonic acid (PFHxS)	Less than 30 ppt
Perfluorononanoic acid (PFNA)	Less than 20 ppt
Perfluorooctanesulfonic acid (PFOS)	Less than 40 ppt
Perfluorooctanoic acid (PFOA)	Less than 20 ppt

On December 17, 2020 and January 19, 2021, the IEPA collected finished water samples from the SWTP and analyzed them for 18 PFAS compounds. For some of the compounds, the IEPA has set Screening Levels that correspond to the current or estimated future health advisory levels.

In each sample from the SWTP only one PFAS compound was detected, PFHxA, as shown in **Table 4-3**.

Table 4-3. Results from PFAS Sampling of SWTP (IEPA 2020-2021)

PFAS Analyte (Acronym)	IEPA Sampling		IEPA Screening Level
	12/17/20	1/19/21	
Perfluorohexanoic acid (PFHxA)	2.4 ppt	2.6 ppt	560,000 ppt
Perfluorobutanesulfonic acid (PFBS)	*	*	140,000 ppt
Hexafluoropropylene oxide dimer acid (HFPO-DA)	*	*	560 ppt
Perfluorohexanesulfonic acid (PFHxS)	*	*	140 ppt
Perfluorononanoic acid (PFNA)	*	*	21 ppt
Perfluorooctanesulfonic acid (PFOS)	*	*	14 ppt
Perfluorooctanoic acid (PFOA)	*	*	2 ppt
Perfluoroheptanoic acid (PFHpA)	*	*	---
Perfluorodecanoic acid (PFDA)	*	*	---
Perfluorododecanoic acid (PFDoA)	*	*	---
Perfluorotetradecanoic acid (PFTA)	*	*	---
Perfluorotridecanoic acid (PFTrDA)	*	*	---
Perfluoroundecanoic acid (PFUnA)	*	*	---
11-chloroeicosafluoro-3-oxaundecane-1-sulfonic acid (11Cl-PF3OUdS)	*	*	---
9-chlorohexadecafluoro-3-oxanone-1-sulfonic acid (9Cl-PF3ONS)	*	*	---
4,8-dioxa-3h-perfluorononanoic acid (ADONA)	*	*	---
N-methyl perfluorooctanesulfonamidoacetic acid (NMeFOSAA)	*	*	---
N-ethyl perfluorooctanesulfonamidoacetic acid (NEtFOSAA)	*	*	---

Note: "*" indicates that measured value was below the minimum reporting level (MRL) of 2 ppt.

The levels of PFHxA found in the finished water from the SWTP are well below the current IEPA Health Advisory level for this compound. However, for any level of PFAS detection the IEPA requests that the following steps be taken:

- Inform consumers of sample results: direct mailing/notices or posting to the CWS website or other means,
- Initiate quarterly monitoring of all raw water sources and finished water for PFAS analytes,
- Begin evaluation of options and develop plans/timeline to reduce public exposure to PFAS in potable water provided by the CWS.

4.4 Finished Water Quality Improvement and Investigation Strategies

The Water Utility provides safe drinking water to its customers as consumer confidence reports dating back to 2014 show no violation of EPA or IEPA drinking water regulations. A summary of the key water quality parameters shown in a tabular format is included in **Appendix B**. A discussion of select aspects of the finished water quality is included herein

4.4.1 THM Reduction Method Investigation

Annual averages of THMs have been recorded below the 60 ug/L EPA drinking water standards limit for years, but some individual quarters have recorded THM levels greater than 80 ug/L. Additionally, IEPA recently raised the minimum chlorine residual limit which may increase the amount of chlorine required for dosing. Therefore, levels of THMs may increase. Common methods used to reduce THM levels include delaying chlorine addition, reducing chlorine dosage, and reducing chlorine contact time. Additional strategies are detailed in CDM Smith's Trihalomethane Evaluation Planning Technical Memorandum included as **Appendix C**.

In order to evaluate which THM reduction strategy would be best for the Water Utility, an investigation into the current treatment processes and THM formation is recommended. The first proposed step in investigation would be to collect and analyze samples from different locations in the SWTP. Additional investigation steps include analyzing the impact of delaying chlorine application until after filtration, additional anion exchange treatment, biofiltration treatment, increased PAC dosing, and use of GAC filter media. Details of each investigation step are also further detailed in **Appendix C**.

4.4.2 PFAS Substances Sampling

While neither the EPA nor the IEPA have yet established a regulatory MCL for PFAS compounds, both have indicated that they intend to do so. In the meantime, it is recommended that the Water Utility initiate quarterly sampling of the raw water sources and the finished water as recommended in the CDM Smith Per- and Polyfluoroalkyl Substances Sampling technical memorandum (**Appendix D**) and as requested by the IEPA.

It is recommended that the Water Utility conduct testing in multiple locations including the SWTP finished water, Lake Decatur, Dewitt wells, Former Vulcan pit, and Lake Toko. The presence of PFAS in source water could impact the Utility's future treatment processes. If the sample results show appreciable removal of PFAS compounds through the water treatment processes, the Water Utility may wish to conduct intermediate PFAS sampling (e.g., before and after softening, before and after anion exchange) to better understand the effectiveness of individual treatment processes used at the SWTP on PFAS removal.

4.4.3 Chlorine Gas Alternatives Evaluation

The Water Utility has utilized elemental chlorine gas safely and effectively without reported issues for many years to provide disinfection. The SWTP maintains a chlorine gas scrubber should a leak occur. However, in light of growing concerns of the use of this deadly gas, regulators and government agencies have implemented more complex and costly requirements to regulate the transport, storage and handling of toxic gases. The CDM Smith Chlorine Gas Alternatives Review technical memorandum (**Appendix Q**) provides a background, historical chlorine use, and evaluation of alternatives. In a comparison of alternatives, the technical memorandum concludes that maintaining the use of a chlorine gas system is currently the most cost-effective approach. However, the Water Utility should monitor non-cost factors, and continue to monitor regulatory changes that may require the Water Utility to reconsider this analysis.

4.4.4 Clarifier Improvements Evaluation

The Water Utility uses two parallel trains of softening clarifiers to pretreat and soften raw water. These clarifiers are nearing the end of their anticipated useful life and therefore the Utility commissioned a study into replacement, rehabilitation, or repair options and probable costs. CDM Smith prepared a technical memorandum, Clarifier Improvements Evaluation, which evaluated the alternative strategies and provided comments and recommendations. This technical memorandum is included as **Appendix G**.

The recommended short-term improvement is to replace the east clarifier train with ClariCone technology within 0-5 years. The updated technology allows for simpler operation, lower maintenance requirements, and lower annual operation and chemical costs. The long-term recommendation, to complete in 6-10 years, is to assess the experience constructing and operating the new ClariCone technology and evaluate whether to replace the west clarifier train with ClariCone technology or to rehabilitate.

4.5 Summary

The Water Utility is in compliance with current drinking water regulations. To keep the system in compliance with and in an even stronger position to meet new regulations, CDM Smith recommends the following:

- Continue removing lead and galvanized steel service lines in their entirety, and identify remaining lines made of lead or galvanized steel;
- Continue to monitor for regulatory changes that could impact the use of chlorine gas as a disinfectant and continues to assess non-cost factors associated with chlorine gas use to determine if change to an alternative form of disinfectant is warranted.
- Monitor total trihalomethanes closely and consider the final results of the Strand study to further lower DBP levels to help ensure continued DBP regulatory compliance. Consider the draft recommendation of implementing new tank mixers and investigation into other reduction methods. Continue to achieve good TOC removal through the lime softening process and maintain the chlorine application point until after softening.
- Monitor for PFAS in the raw and finished water, as requested by the IEPA and in anticipation of potential new regulations related to PFAS.
- Collect and review water quality information on potential groundwater sources of supplemental water supply to evaluate any water quality impacts of blending.
- Continue to maintain a highly engaged and knowledgeable staff by retaining institutional knowledge, cross training staff, and perform long term workforce planning to avoid potential reductions in drinking water quality by properly training staff.

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Section 5

Water Treatment Plant, Storage and Pump Station Condition Assessment

This section provides an overview of the condition assessments that were performed on each water system facility, current and future storage, existing pumping capacity, and summarizes future recommended improvement projects.

5.1 Overview

CDM Smith conducted field visits in July 2020 to perform assessments of the Water Utility's water system facilities. CDM Smith produced a summary of findings and recommended means of addressing issues. Five technical memorandum summarized assessments of the Water Utility's clarifiers, dam, pump, filter, and clearwell performance. Each of these memoranda can be found in **Appendices G, H, I, J,** and **K** but are summarized in this section. CDM Smith also conducted a series of staff interviews between March 31, 2020 and April 3, 2020. The interviews informed the master plan by providing information on the day-to-day procedures and issues encountered that CDM Smith would overwise have no opportunity to observe. A detailed table of all issues is included as **Appendix F,** but a summary of the most critical improvements required is included in this section.

5.2 Condition Assessments

The methodology and the results of the field visits and assessments of the Water Utility's facilities are detailed in this section.

5.2.1 Facilities Condition Assessment Methodology

CDM Smith staff conducted site visits at each of Decatur's water system facilities to evaluate equipment performance and complete visual inspections. Data collected through this process was used to evaluate the condition of the Water Utility's raw water intake, South Water Treatment Plant (SWTP), storage and pumping facilities and water sources. Specialists from each of the following disciplines attended the site visits in order to evaluate the facilities for all types of issues and possible improvements:

- Site Civil
- Architecture
- Structural
- Process Mechanical
- HVAC
- Plumbing
- Electrical
- Instrumentation

Certain sites could not be observed during the site visit, so they were analyzed using previous studies conducted. The Lake Decatur Dam and Oakley Sediment Basin were reviewed based on previous reports and studies performed. Additionally, the clarifiers were previously assessed and CDM Smith reviewed the assessment and provided a recommendation based on the reported information.

Assessment findings were compiled into one list and sorted by facility designation, further by area designation within the water treatment plant, facility condition rating, issue criticality and class, and facility functional category. Each of the issues found were translated into a recommended project with estimated construction, engineering, and total project costs. A list of all projects can be found as **Appendix N** and a summary table of project costs, separated by facility can be found in **Table 5-5.**

A risk-based assessment of each projects' condition and function was performed to determine the most critical projects that should be included in Decatur's Capital Improvement Plan (CIP). Each project received a condition score on a scale of 1 to 5, with 1 being the most urgent condition and 5 being the least urgent condition, as detailed in **Table 5-1**. Each project also received a criticality ranking on a scale of 1 to 5, with 1 being the most critical condition and 5 being the least critical condition. There were five considerations made in each criticality ranking and they are detailed in **Table 5-2**. Each project was then assigned a Problem Class based on the facility's condition and criticality, as shown in **Table 5-3**.

Table 5-1. Condition Rating

Condition Rating	Description
1	A condition in which failure of a primary item of equipment or structure is imminent, and its failure would directly result in loss of a significant portion of plant capacity, jeopardize water quality, jeopardize the safety of personnel, or cause further damage to equipment or other structures.
2	A condition in which failure of a backup unit of equipment or structure is imminent, and failure to attend to the problem would result in loss of backup capacity, jeopardize the safety of personnel, or cause further damage to equipment or other structures (e.g., a device providing the first level of backup, such as an engine generator or the third pump in a bank of three pumps in which two pumps will be required to meet peak requirements).
3	A condition of failure or imminent failure in some ancillary equipment or structure (e.g., leaking window frames), the failure of which will not impair the process or safety, but may lead to deterioration which could result in increasing repair costs if not attended to in a timely manner.
4	An improvement which has not been made but which would result in protecting the status quo with regards to water quality, water quantity or safety (e.g., updating lighting fixtures).
5	Anything which should be corrected or improved, which is not listed above and the failure of which does not imperil water quality, water quantity, or safety (e.g., removal of equipment which is not in use).

Table 5-2. Criticality Rating

Criticality Ranking	Plant Capacity Impacted	Water Quality/ Regulatory	Safety Hazard	Replacement Lead Time (Years)	Severity Level
1	Plant Shutdown	Regulatory Violation	Regulatory Violation	> 6	Catastrophic
2	75%	Major Quality Impact	Major Hazard	3 to 6	Critical
3	50%	Moderate Quality Impact	Moderate Hazard	1 to 3	Moderate
4	25%	Minor Quality Impact	Minor Hazard	0.5 to 1	Low
5	No Impact	No Impact	No Hazard	< 0.5	None

Table 5-3. Problem Class

		Condition Rating				
		1	2	3	4	5
Criticality Rating	1	Class 1	Class 1	Class 1	Class 2	Class 3
	2	Class 1	Class 1	Class 2	Class 3	Class 3
	3	Class 2	Class 2	Class 3	Class 3	Class 4
	4	Class 3	Class 3	Class 4	Class 5	Class 5
	5	Class 4	Class 4	Class 4	Class 5	Class 5

Projects were further organized by functional category designation, with Operational Items being the most critical, followed by Non-Operational Items and the least critical designation being Maintenance Items. These categories are further detailed in **Table 5-4**.

Table 5-4. Functional Categories

Functional Category	Description
Operational Items	Items that directly affect the production or quality of water and the expense of remediation would be covered under a capital improvement project.
Non-Operational Items	Items that do not directly affect the production or quality of water and the expense for remediation would be covered under a capital improvement project.
Maintenance Items	Items that the expense for remediation would be covered under a maintenance budget, as opposed to being treated as an individual capital project under the capital budget.

Assessment of the Water Utility's facilities also came from Water Utility staff interviews. CDM Smith conducted interviews in April 2020 with the Operations and Maintenance staff. Operations staff all perform various operations tasks at the SWTP as well as a restricted level of maintenance activities. The operators interviewed ranged in operations experience from less than 1 year to over 30 years. The maintenance staff interviewed ranged in experience from less than 1 year to over 35 years. The focus of the interviews was in the following areas:

- Category I – Systems or equipment needing attention, repair, replacement, etc., including equipment requiring continuous maintenance.
- Category II – How they operate, specifically any specific issues with regard to plant operation, distribution system pumping, etc.

- Category III – What they would like to see being done differently or additional tools or resources needed to do their job more efficiency. Overall, all staff felt very positive about their jobs, the proactive nature of staying up on equipment maintenance and replacement, and the overall condition of the facility. They felt everything works well, they are well organized, but they indicated that there may be some opportunities for improvement.

5.2.2 Facilities Assessment Findings

The results of the condition assessments are detailed in this section.

5.2.2.1 Raw Water System Facilities

The raw water system, including the Dewitt Well Field, Cisco Wells, Vulcan Pit, Lake Toko, Raw Water Pump Station and the intake structure, had multiple issues in each discipline ranging from Class 1-5. There were only two operational, Class 1 projects. Both of these projects were noted at the Raw Water Pump Station.

- The outside pad mounted transformers for the service entrance and low voltage power are rusting and approaching the end of their useful life, these transformers need to be replaced within 5-10 years. Failure would result in shutdown of the pump station.

- The main Medium Voltage Motor Control Center (MV-MCC) at the Raw Water Pump Station is at the end of its useful life. The MV-MCC should be replaced in-kind within the next 5-10 years. Additionally, the application of variable frequency drives should be considered.

The raw water facilities have four more operational projects that range from Class 2-5 and multiple other issues that are nonoperational or maintenance projects lower than Class 1. The details of all projects can be found in **Appendix N**.

5.2.2.2 Water Treatment Facilities

The SWTP has multiple operational, nonoperational, and maintenance projects. The nine Class 1 operational projects are detailed below. Additionally, there are Class 2-5 operational issues and many non-operational and maintenance projects. The SWTP had the greatest number of projects detailed. A summary of the costs associated with the SWTP and all other facilities can be found in **Table 5-5.**

1. The most expensive Class 1 operational issue is the rehabilitation or replacement of the east clarifier train with new Claricones. This issue should be addressed immediately.

2. The most critical issue in the worst condition is the leaking in the aging caustic system. The caustic system needs a full replacement and was given a Condition and Criticality Rating of 1.

3. The chlorinator with remote control has a control valve that is not large enough to meet all the systems' needs. As a result, operations staff need to supplement chlorine feed with the manual chlorinators, which require additional operator oversight to keep adjusted at the desired chlorine feed rate. The 1000 ppd control valve should be replaced with 2000 ppd control valve.

4. The chlorine feed piping is aging and becoming brittle. The piping should be inspected and replaced as needed.

5. The chlorine vaporizers are aging. The aging vaporizers should be replaced.

6. The Main 4160V Westinghouse Switchgear and the two 4160V-480V step-down transformers in the main electrical room are in good working condition but approaching the end of their useful life. These are original to the plant and maintenance parts may become increasingly difficult to procure. Main switchgear and two transformers should be replaced within 5-10 years.

7. Westinghouse MCC-1 and MCC-2 are in good working condition but are original to the plant and approaching the end of their useful life. Additionally, MCC-1 is located in a main hallway and not a dedicated electrical area. MCC-1 and MCC-2 should be replaced with a new MCC within 5-10 years. MCC-1 should also be relocated to a dedicated electrical room.

8. The 4160V High Service MCC appears to be in good working condition however it is original to the plant and approaching the end of its useful life. The High Service MCC should be replaced with a new MV-MCC within 5-10 years.

9. The service 34.5kV - 4160Y/2400V, 3750 kVA transformers and 34.5kV 600A primary switches are in good working condition but are very rusted and nearing the end of their useful life. These should be replaced with new transformers and primary switches within 5-10 years.

5.2.2.3 Water Distribution Facilities

The water distribution facilities include the South Booster Station, William Street Reservoir and Pump Station, the Division Street Tank, Garfield Street Tank, Meters, and Bulk Water Purchase Stations. There were no Class 1, operational issues noted at any of these facilities. Many of the issues are maintenance or non-operational issues and therefore may not be of immediate concern for the Water Utility. A complete list including details and recommended projects can be found in **Appendix N**.

5.2.2.4 All Facilities Assessment Findings Overview

Table 5-5 summarizes the raw water, water treatment, and water distribution facilities recommended improvements' costs by facility and discipline in year 2020 dollars. Costs are rounded up to include additional conservatism and clarity.

Table 5-5. Summary of Condition Assessment Costs by Facility and Discipline

Facility/Discipline	Cost	% of Total Facilities Recommended Improvements Costs
South Water Treatment Plant		
Site Civil	$10,600,000	--
Architectural	$340,000	--
Structural	$140,000	--
Process Mechanical	$37,390,000	--
HVAC	$290,000	--
Plumbing	$70,000	--
Electrical	$16,080,000	--
Instrumentation	$7,920,000	--
Total	**$72,830,000**	**64.24%**

Facility/Discipline	Cost	% of Total Facilities Recommended Improvements Costs
Raw Water Pump Station		
Site Civil	$1,920,000	--
Architectural	$80,000	--
Structural	$6,000	--
Process Mechanical	$3,640,000	--
HVAC	$5,000	--
Plumbing	$1,000	--
Electrical	$2,410,000	--
Instrumentation	$150,000	--
Total	**$8,220,000**	**7.25%**
South Booster Pump Station		
Architectural	$4,000	--
Process Mechanical	$20,000	--
HVAC	$1,000	--
Electrical	$50,000	--
Total	**$80,000**	**0.07%**
William Street Reservoir and Pump Station		
Site Civil	$43,000	--
Architectural	$160,000	--
Structural	$1,000	--
Process Mechanical	$720,000	--
HVAC	$40,000	--
Plumbing	$6,000	--
Electrical	$250,000	
Instrumentation	$40,000	--
Total	**$1,260,000**	**1.11%**
Lime Sludge Lagoons		
Site Civil	$90,000	--
Process Mechanical	$330,000	--
HVAC	$5,000	--
Electrical	$110,000	--
Total	**$540,000**	**0.48%**
Bulk Water Purchase Stations		
Instrumentation	$180,000	--
Total	**$180,000**	**0.16%**
Cisco Wells		
Site Civil	$60,000	--
Total	**$60,000**	**0.05%**
Dewitt Well Field		
Site Civil	$5,390,000	--
Total	**$5,390,000**	**4.75%**
Division Street Tank		
Site Civil	$30,000	--
Structural	$30,000	--
Total	**$60,000**	**0.05%**

Facility/Discipline	Cost	% of Total Facilities Recommended Improvements Costs
Garfield Tank		
Site Civil	$40,000	--
Structural	$40,000	--
Total	**$80,000**	**0.07%**
Source Water		
Site Civil	$10,400,000	--
Total	**$10,400,000**	**9.17%**
Vulcan Pit		
Site Civil	$1,320,000	--
Total	**$1,320,000**	**1.16%**
Lake Toko		
Site Civil	$12,960,000	--
Total	**$12,960,000**	**11.43%**
Facilities Recommended Improvements, TOTAL	**$113,380,000**	

Each project recommended under this Section has been included in a Capital Improvement's Project. Project forms detailing cost, schedule and recommendations are included as **Appendix A**. **Tables 8-1, 8-2,** and **8-3** outline the implementation schedule for each capital project.

5.2.2.5 Cyber Security System

Remote access to the SCADA network is restricted to a select list of required personnel, both from the Water Utility's operations staff and within the Water Utility's SCADA integrator's team. Water Utility operations staff access the SCADA system remotely only from the municipal network within the water treatment plant, via a remote desktop protocol (RDP) connection on their Water Utility-issued computers. Concentric staff, when requested by Water Utility operations staff, access the SCADA system remotely using the ConnectWise Automate application.

Based on the criticality of the SCADA system to water operations and the age of the computers and network hardware in the SCADA system (some systems were still running Windows 7 at the time of the investigation though a project was planned to address this), the system may still be at risk. A cybersecurity risk assessment should be conducted on the water SCADA system, to identify areas of concern and propose countermeasures to remediate the concerns. This assessment, utilizing tools and techniques developed to be used in a control system environment and not adversely impact the operation of sensitive legacy controllers and hardware, will identify, and prioritize existing vulnerabilities that should be rectified to best protect the operation of the system. The risk assessment should also identify threats that could take advantage of these vulnerabilities and compromise the operation of the system and reduce the Water Utility's ability to deliver water at the proper quality and/or quantity to its customers, and rank the relative probability of the vulnerabilities being exploited by threats and the impact to the system and Water Utility if it does occur. Topics or areas the assessment should touch on include:

1. System asset inventory

2. Network segregation/segmentation
3. AAA services (Authentication, authorization, and accounting)
4. Firewalls/security appliances (selection, configuration, maintenance)
5. Demilitarized zone (DMZ) configuration/services
6. Intrusion detection/prevention systems (IDS/IPS) - configuration, long term maintenance
7. Antivirus/antimalware
8. Device configuration/hardening
9. OS configuration/hardening
10. Network configuration/hardening
11. Patch management
12. Change management
13. Disaster recovery
14. Cybersecurity policy for the SCADA system (could be a subset of the overall IT policy or standalone)
15. Procedures to achieve the goals outlined in the policy

The assessment should be conducted by a provider regularly engaged in these services, and that also is familiar with industrial control systems (also known as operational technology, or OT) and the differences they pose from a traditional information system (IT) infrastructure. Some common practices utilized to manage and maintain IT infrastructure can have detrimental effects when applied within OT environments.

By conducting the security assessment and understanding the risks present in the water SCADA system, the Water Utility can properly prioritize resources and projects to reduce the exposure to threats that are becoming increasingly prevalent to control systems.

In addition, the Water Utility and its SCADA system provider (Concentric) should develop a formal understanding for the division of security responsibilities between the municipal and SCADA networks. This will help ensure that no aspect of network security, especially the overlapping barrier between the municipal network and the SCADA network, is missed or one party assumes responsibility of the other. By closer coordination and greater understanding of responsibilities, the overall security of both systems will be enhanced.

Appendix E contains a presentation provided to Water Utility personnel describing these concepts in more detail.

5.2.2.6 Staff Interview Results Summary

The results of the staff interviews were divided into the three categories of focus. Category I included systems or equipment needing attention, repair, replacement, etc., including equipment requiring continuous maintenance. Category II included any specific issues with regard to plant operation, distribution system pumping, etc. Category III included what staff would like to see being done differently or additional tools or resources needed to do their job more efficiency. A detailed summary of the results of the interviews can be found as **Appendix F.**

5.2.3 Clarifier Assessment

The SWTP has two parallel trains of two-stage softening clarifiers that have been in service more than 30 years and are approaching the end of their anticipated reliable life. The clarifiers are used to soften the raw water prior to filtration. The Water Utility is evaluating six alternative strategies including repair of the existing clarifiers, rehabilitation of the existing clarifiers, and replacement with new ClariCone technology. CDM Smith performed an evaluation of the proposed strategies and provided a technical memorandum that can be found as **Appendix G.**

ClariCone technology has many advantages over the existing clarifiers. ClariCones allow for a simpler operation, less maintenance and a lower operational annual cost. Due to these advantages, it is recommended that the Water Utility replace the east clarifier train with two ClariCones within the next 5 years. In the next 6-10 years, it is recommended that the Water Utility assess the first few years of operating the South Water Treatment Plant with two ClariCones and the existing west clarifiers. Based upon the successes or challenges of operating with two softening treatment technologies, the Water Utility should evaluate whether it is preferable to proceed with rehabilitation of the west clarifier train or to proceed with replacement of the west clarifiers with ClariCone technology.

5.2.4 Lake Decatur Dam and Oakley Sediment Basin Assessment

The Water Utility shared condition assessments of the Oakley Sediment Basin and the Lake Decatur Dam from the years 2016, 2017, 2018 and 2019. CDM Smith reviewed these assessments and provided a technical memorandum June 5th, 2020 which can be found as **Appendix H.**

5.2.4.1 Oakley Sediment Basin

The Oakley Sediment Basin is composed of a series of perimeter and interior dikes used to store dredged sediments. The perimeter embankments have been raised a total of 20 feet since 2005 in order to increase storage capacity of the basin. The method of raising the perimeter embankment is typically referred to as the "upstream method" because the dam raise section is placed predominantly upstream of the existing embankment.

The condition assessment performed by Hansen Professional Services did not indicate any abnormal conditions or concerns. One area was noted to have minor seepage, but the recommendation was to continue to monitor the seepage for the presence of any turbidity which would indicate soil transportation through the embankment. The seepage was observed along the south side toe of the facility near the east end of the basin. The condition assessments indicated that slopes are well maintained and in good condition.

In 2017, Klingner and Associates entered into an agreement with the Water Utility to review plans to increase the height of the basin walls by six feet. Klingner and Associates performed a slope stability analyses to confirm the proposed modifications would meet the USACE 's minimum allowable Factor of Safety (FS) requirement. The material properties used in the analyses were assumed values based on the boring logs and empirical values. No lab testing of soil samples was accomplished to establish soil properties such as density, shear strength data for long-term and short-term strengths, and permeability values. Hence, there could be some variation in the FS values expected. There appears to be some inconsistency in the slope stability results shown; specifically,

- End-of-construction case appears to show a phreatic surface within the embankment.
- Steady-state case does not show a phreatic surface within the dam.
- This is backwards from normal analyses for these load cases. End of construction usually does not assume any water behind the dam; i.e., no phreatic water surface. Steady state conditions are for long-term seepage conditions where there is a phreatic surface established within the dam. Maybe they are just labeled incorrectly, but they should be reviewed, and corrected, if necessary.
- The failure surfaces were all generated for downstream failure surfaces. The FS values for the downstream just meet recommended FS values.
- No failure surfaces were generated for upstream failure surfaces. Because of portions of the raised embankment being placed over soft sediments, it is highly likely that upstream failure surfaces will have lower FS values that will go into the soft sediments. Upstream failure surfaces should also be checked to see what the FS values are relative to the required FS values.

As such, CDM Smith recommends that an analysis be conducted using data gathered from lab testing of soil samples.

5.2.4.2 Lake Decatur Dam

Several phases of rehabilitation have been performed on different areas of the dam since it was constructed in 1922. It appears normal maintenance is being performed by the Water Utility. Annual inspections prepared by Hanson Professional Services Inc. in general presented some areas requiring a level of maintenance over the years. The areas noted by the inspections should continue to be monitored and scheduled for maintenance as necessary.

The only area of concern is related to the bascule gates (crest gates). The bascule gates are over 50 years old, and their repairs and maintenance appear to be occurring at shorter intervals. During inspections performed between 2016 – 2019, no operation of the bascule/crest gates was performed during the visual inspections. It was noted in the reports that the Water Utility had reported to the inspector that the facility mechanism (i.e., gates) were functioning properly. In the 2019 inspection report, the Water Utility reported that repairs to the North Bascule gate machinery were in progress. As of May 2020, the Water Utility indicated that repairs to the north bascule gate have not been completed, but that the replacement of the bearings and pins is taking place.

In the absence of any operational / testing reports on the Bascule Gates, it is very difficult to adequately assess the overall condition of the bascule gates. All of the information provided was primarily visual, and the inspector did not appear to be present to physically observe the operation of the gates during the inspections.

As such, CDM Smith recommends the following be performed:

- It is our understanding that the Water Utility has a contract with Hanson Engineering. It is recommended that periodic inspections of the Lake Decatur Dam and operation of the gates continue.
- Have an independent third-party testing firm or utilize Hanson Engineering to oversee the testing and operation of both bascule gates to verify the full range of gate operations and that the gates are fully functioning. A report documenting the results of the testing should be prepared by the testing firm to document the gate testing.

5.2.5 Pump Performance Assessment

CDM Smith and the Water Utility staff conducted pump performance testing on all of the SWTP's low-lift and high service pumps as well as the pumps located at the William Street Pump Station. A detailed testing procedure and pump testing results can be found in **Appendix I** as a technical memorandum.

Performance testing is a common procedure that determines how well a pump operates compared to its original manufacturer specifications and allows for assessment of the condition of its internal components without any disassembly. Data collected from the performance tests is compared with the original factory performance test curves generated for each pump. The pump efficiencies are also calculated and compared with the factory test efficiencies. Knowledge of the efficiency at different operating conditions can be utilized by the Water Utility to determine their ideal pump operation.

The low-lift raw water pump station has four vertical turbine pumps. There are five horizontal split case high service pumps and four vertical turbine transfer pumps at the SWTP. The William Street Pump Station has three vertical turbine pumps. Preliminary information on each pump tested can be found in **Table 5-6**.

Table 5-6. Pump Overview

Pump Type	Pump Number	Pump Information
Raw Water Pump	Pump #1	Vertical turbine pump with 21-inch diameter impeller. Single stage. Operating head 46.5-feet.
Raw Water Pump	Pump #2	Vertical turbine pump with 16.10-inch diameter impeller. Operating head is 50-feet.
Raw Water Pump	Pump #3	Vertical turbine pump with a 14.5-inch diameter impeller.
Raw Water Pump	Pump #4	Vertical turbine pump with a 21.0-inch impeller.
High Service Pump	Pump #1	Horizontal split case centrifugal pump with a rated flow of 12,500 gpm at a rated total head of 194-feet with an impeller of 22.5-inches in diameter.
High Service Pump	Pump #2	Horizontal split case centrifugal pump with a rated flow of 9,345 gpm at a rated total head of 194-feet with an impeller of 22.5-inches in diameter.
High Service Pump	Pump #3	Horizontal split case centrifugal pump with a rated flow of 6,260 gpm at a rated head of 194-feet with an impeller of 21.5-inches in diameter.
High Service Pump	Pump #4	Horizontal split case pump with a rated flow of 12,500gpm at a rated head 194-feet with an impeller of 22.5-inches in diameter.

Pump Type	Pump Number	Pump Information
High Service Pump	Pump #5	Horizontal split case pump with a rated flow of 12,500 gpm and a rated head of 194-feet with an impeller of 22.75-inches in diameter.
Water Treatment Plant Transfer Pump	Pump #1	Vertical turbine pump. No rated flow or head indicated. The impeller is 21-inches in diameter.
Water Treatment Plant Transfer Pump	Pump #2	Vertical turbine pump. Rated flow of 9,380 gpm at a rated head of 45-feet using a 16.10-inch diameter impeller.
Water Treatment Plant Transfer Pump	Pump #3	Vertical turbine pump. Rated flow is 6,250 gpm at the rated head of 45-feet with an impeller diameter of 14.5-inches.
Water Treatment Plant Transfer Pump	Pump #4	Vertical turbine pump. Rated flow is 12,250 gpm at the rated head of 45-feet with an impeller of 21-inches.
William Street Pump Station and Reservoir Pump	Pump #1	Vertical turbine pump with open line shaft. Two (2) stage pump with a rated flow of 3,400 gpm at a rated head of 88-feet per stage, or total head of 176-feet.
William Street Pump Station and Reservoir Pump	Pump #2	Vertical turbine pump with open line shaft. Two (2) stage pump with a rated flow of 3,400 gpm at a rated head of 88-feet per stage, or total head of 176-feet.
William Street Pump Station and Reservoir Pump	Pump #3	Vertical turbine pump with open line shaft. Two (2) stage pump with a rated flow of 1,600 gpm at a rated head of 81.5-feet per stage, or 163-feet total head.

Raw Water Pumps #1 and #4 showed no decrease in achievable total dynamic head (TDH) but showed the greatest drop in efficiency. Raw Water Pump #3 showed a minor decrease in performance at less than 10% below its original factory test. Pump #2 showed the largest decrease in performance at 15% lower TDH than original factory test.

The High Service Pumps showed very little degradation in performance and did not show any decrease in efficiency from the factory testing.

The Water Treatment Plant Transfer Pumps showed minor degradation in performance. The greatest degradation to achievable TDH was 10% below the factory performance tests. The efficiency of Transfer Pumps #2 and #3 was 10-15% lower than their factory testing efficiencies. Transfer Pumps #1 and #4 showed 20-25% decreases in efficiencies.

The William Street Pump Station Pumps #2 and #3 showed no degradation in performance from factory testing. Pump #1 showed around 10% degradation in performance. All William Street Pump Station Pumps showed minor decrease in efficiency at about 10% lower than their factory testing efficiencies.

5.2.6 Filter Assessment

CDM Smith conducted a filter assessment at the SWTP which included assessment of media, underdrains, and backwash procedures. A detailed procedure for each assessment and results are included in **Appendix J**.

The filter media was assessed in a filter dig assessment which included media sampling and analysis. The media was visually inspected and appeared level and mostly clear. Foreign material found in the filter included plant material and some small strips of plastics. The filter media was further analyzed through lab testing by Bowser-Morner, Inc. The results of media observations and analysis indicate that the filter media is generally in good condition, and the plant is operated and maintained well.

5.2.7 North and South Clearwell Inspection

Structural inspections were performed on the north and south clearwells at the SWTP. The inspections assessed the concrete clearwell, concrete masonry unit (CMU) baffle walls, and influent piping. The inspections were a combination of visual observations and audible sounding. A summary of findings with photographs is included in **Appendix K** and summarized here. A visual observation of the clearwells' base slabs indicated that they were in good condition. The top slabs of the clearwells were also found to be in good condition. The CMU walls were found to have minor leakage and one isolated area of delamination. The leakage should be addressed by Water Utility maintenance staff or an outside contractor hired by the Water Utility to prevent the leaks from expanding and the delamination should be removed and patched. There is also deterioration on the filter influent piping. This piping should be re-primed and painted to continue to decrease the rate of corrosion and prolong the life of the piping.

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Section 6

Distribution System Evaluation and Modeling, Storage and Pumping Optimization

This section summarizes the evaluation of the distribution system with review and updates to the hydraulic model, simulating distribution system performance for different water demand conditions, available storage, and pumping capacity of the existing distribution system.

6.1 Hydraulic Model

6.1.1 The Water GEMS Model

The water distribution system hydraulics model (the model) was developed in Bentley WaterGEMS software. It was last calibrated as part of the *Water System Master Plan Update* (Strand, 2015) and validated/updated in the *EPS Modeling and Chlorine Residual Maintenance Plan* (Strand, 2020). **Figure 6-1** is a screenshot of the received model and **Table 6-1** summarizes the number of model elements. The model represents the SWTP (7.5-MG reservoir and its high service pumping station), elevated tanks, reservoir, booster pump stations, the Water-Utility-owned water mains, hydrants and valves. It does not include water service laterals.

Table 6-1. Element Count from Received WaterGEMS Model

Type	Count	Note
Junction	24,757	11,085 has positive water demand assigned
Pipe	30,215	536 miles of water mains; average model pipe length is 93 ft, diameter ranging from ¾ inches to 54 inches
Tank	5	Division St tank, Franklin St tank, Garfield Ave tank, William St reservoir, SWTP reservoir
Reservoir	1	at SWTP
Hydrant	4,137	
Pump	12	SWTP, SZBS and WSPS
Check valve	4	3 at SZBS, 1 used to separate North/South zone
Isolation valve	10,124	1 used to separate North/South zone; others were left open in model simulation
Flow control valve	4	
Pressure reducing valve	2	
Throttle control valve	1	

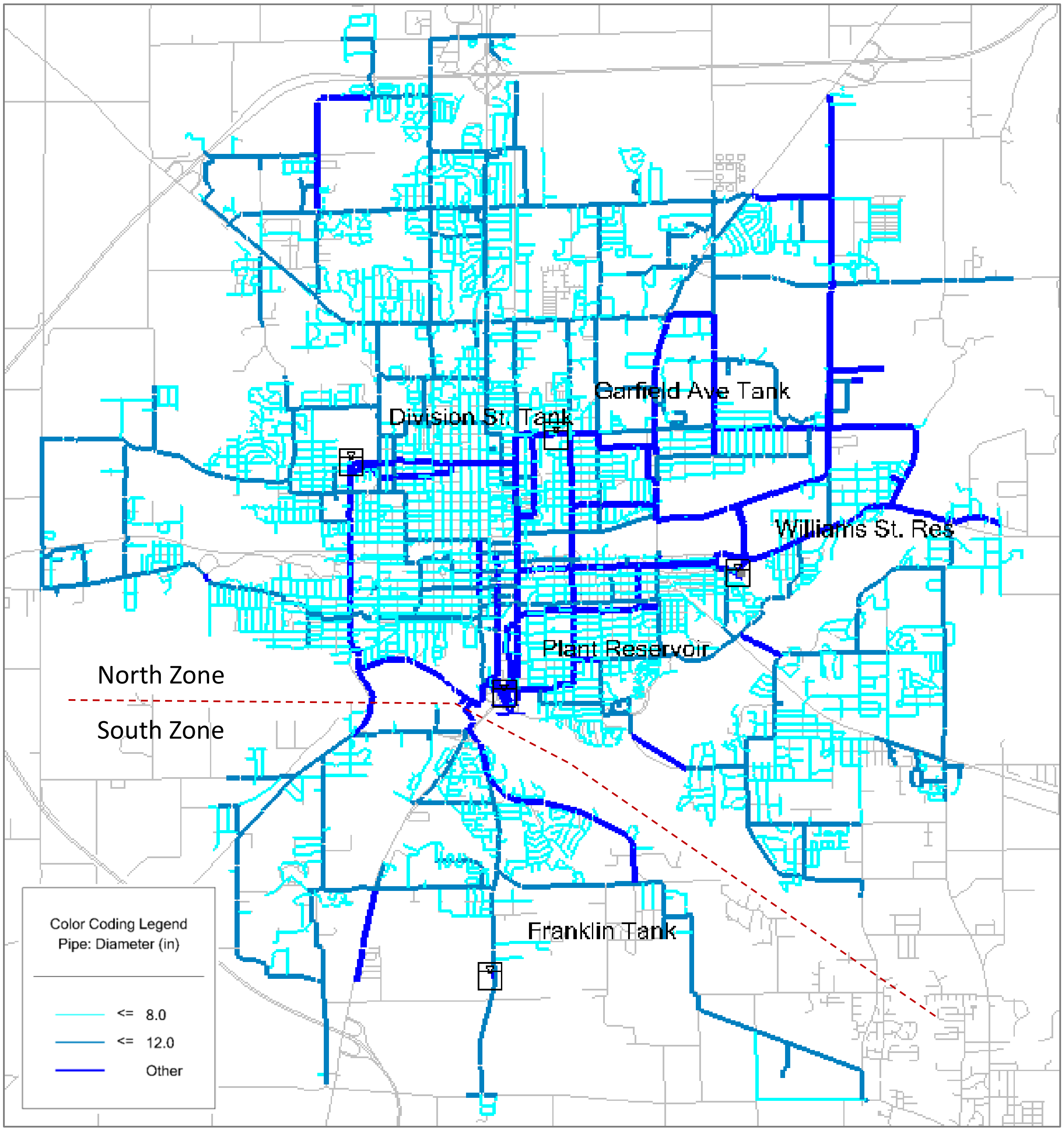

Figure 6-1. WaterGEMS Model Screenshot

6.1.2 Model Review

6.1.2.1 GIS Comparison and Connectivity Checks

The initial model review and update was performed in August 2020. Model pipe diameter and connectivity were compared against the Water Utility's water system GIS. WaterGEMs built-in tools were also used to check the model pipe crossings for inconsistencies. Junction elevations were validated against the county LiDAR bare earth elevation data. Upon review, the received model did not include water mains replaced since 2018. **Figure 6-2** highlights the changes made to bring the model up to date. This included updating connectivity, pipe diameter, pipe material, C factor (assume 130 for new pipe) and installation year. Duplicate model pipes were removed.

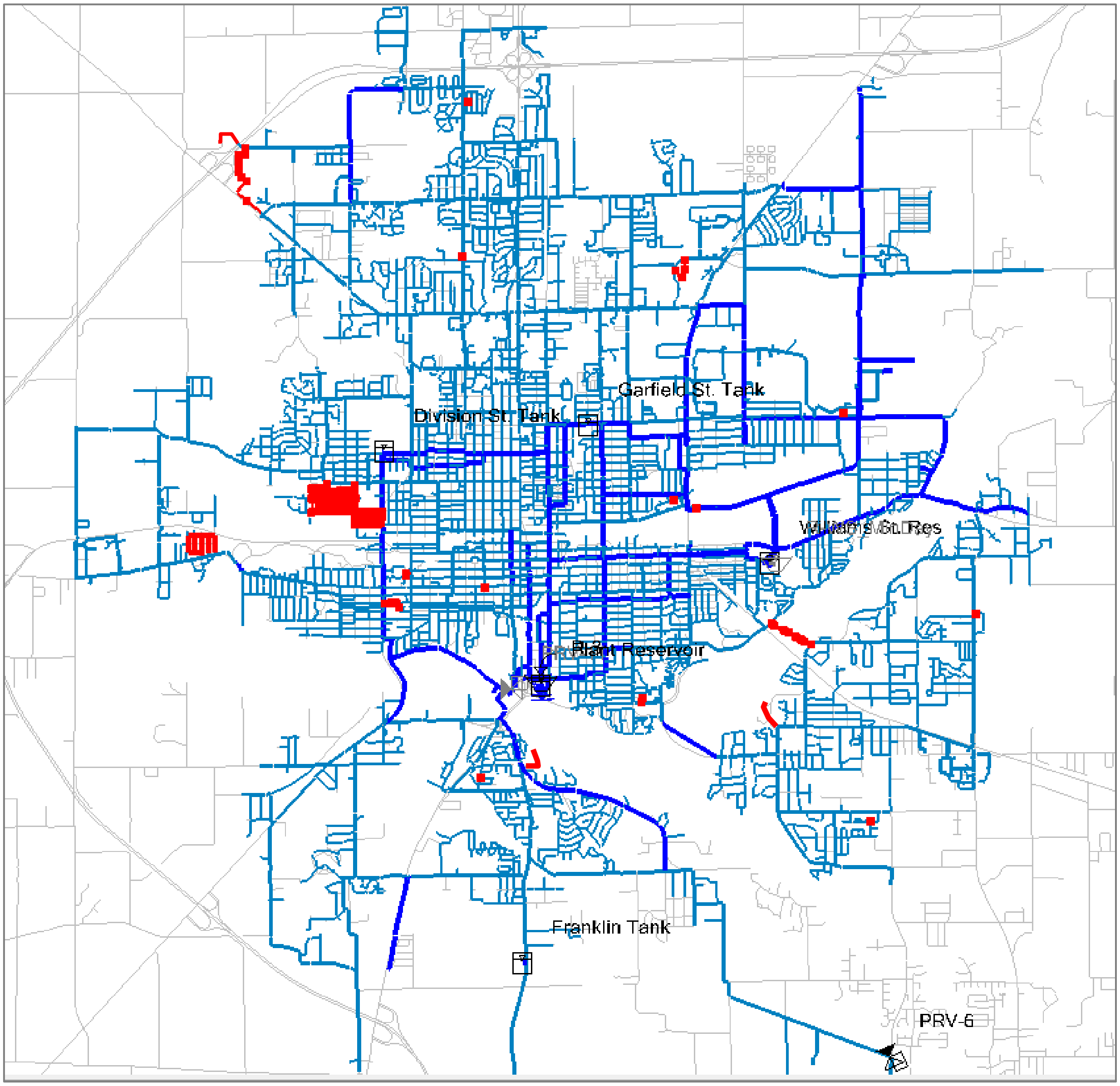

Figure 6-2. Model Update with New/Replaced Water Mains and Connectivity Fixes (Highlighted in Red)

6.1.2.2 Facility and Operation Review

Drawings and operation data of WTP, booster stations, reservoirs and tanks were reviewed. The model representations of these facilities and their operations were last updated by Strand (Strand, 2020). They are consistent with the drawings and data provided by the Water Utility.

6.1.2.3 Water Demand Allocation

The model has 11,085 junctions with positive water demand assigned. They were populated as part of the work performed by Strand (Strand, 2020). Besides the top ten largest users, the remaining demands were geospatially allocated with the August 26 – 29, 2019 water meter data.

These locations appear reasonable in the general vicinity and adequate for system evaluation purpose. Future refinement could include moving water demand junctions away from hydrant leads (20% of 11,085) as part of model updates and future demand adjustments.

For the model simulations in this study, water demand in different demand scenarios were distributed to these model junctions by the same allocation percentage.

6.1.3 Model Validation and Update

After presenting the preliminary model simulation results in the October 2020 distribution system workshop, additional checks to the model were recommended. The Water Utility performed six additional fire flow tests in the east and north side of the system, and measured system pressures in the vicinity of Manchester Court and Johns Hill Magnet School on 10/30/2020. The Water Utility also provided an ISO *Public Protection Classification Summary* report (ISO, 2017). The report included 27 additional fire flow tests performed some time in 2016.

The model validation and update process included replicating the fire flow test results (2015 Water Master Plan tests [23], 2017 ISO report tests [27], and 2020 tests [6]) in the model. The model pipe's Hazen-Williams roughness coefficient (C factor) were refined when necessary.

6.1.3.1 Hazen-Williams Coefficient

According to the 2015 Master Plan, the model pipe's Hazen-Williams roughness coefficient (C factor) was calibrated using fire flow tests, C factor tests and hydrant pressure recorder data (Strand, 2015). C factor is a dimensionless coefficient for friction/roughness of pipes. A smoother pipe has higher C factor, and can pass higher flow rate. The original C factors appear to be dependent on pipe diameter and vary slightly between the North and South pressure zones. The east side of North Pressure Zone has lower C factor than the rest of the system.

6.1.3.2 Model Validation with Fire Flow Tests

The fire flow test data include measured static pressure, residual pressure, hydrant flow and calculated available fire flow at 20 psi. For the 2020 test data with known test time, the model replicated the system condition and simulated the residual pressure. For the ISO tests without known test time, the model simulated a range of water demands and system operations conditions. The model results were then used to interpolate the available fire flow and residual pressure with the recorded test static pressure.

6.1.3.3 Model Pipe C Factor Update

The C factors were refined, within typical range, so that the model-simulated residual pressure is within 5 psi of the test readings. **Table 6-2** to **Table 6-4** summarize the model validation results after C factor adjustment. **Figure 6-3** maps the location of the fire flow tests.

Incorporating the age vs. C-factor findings from the 2015 Master Plan update (Strand, 2015), water main pipe capacity generally reduces by 10% every 10 years. This is translated to the following in updating the model C factors:

- C=130 for pipes installed since year 2010 (less than 10 years old)

- C factor drops by 10 per decade for pipes built before year 2010
- C factor for small diameter water mains (6 inch or smaller) is 15-20 psi less than large diameter mains of similar age
- Minimum C factors for small and larger diameter mains are 60 and 80. Exception was the three areas with fire flow tests (FF4, FF9 and FF12) suggesting lower C factors. This might indicate unknown local blockage and valve closing.

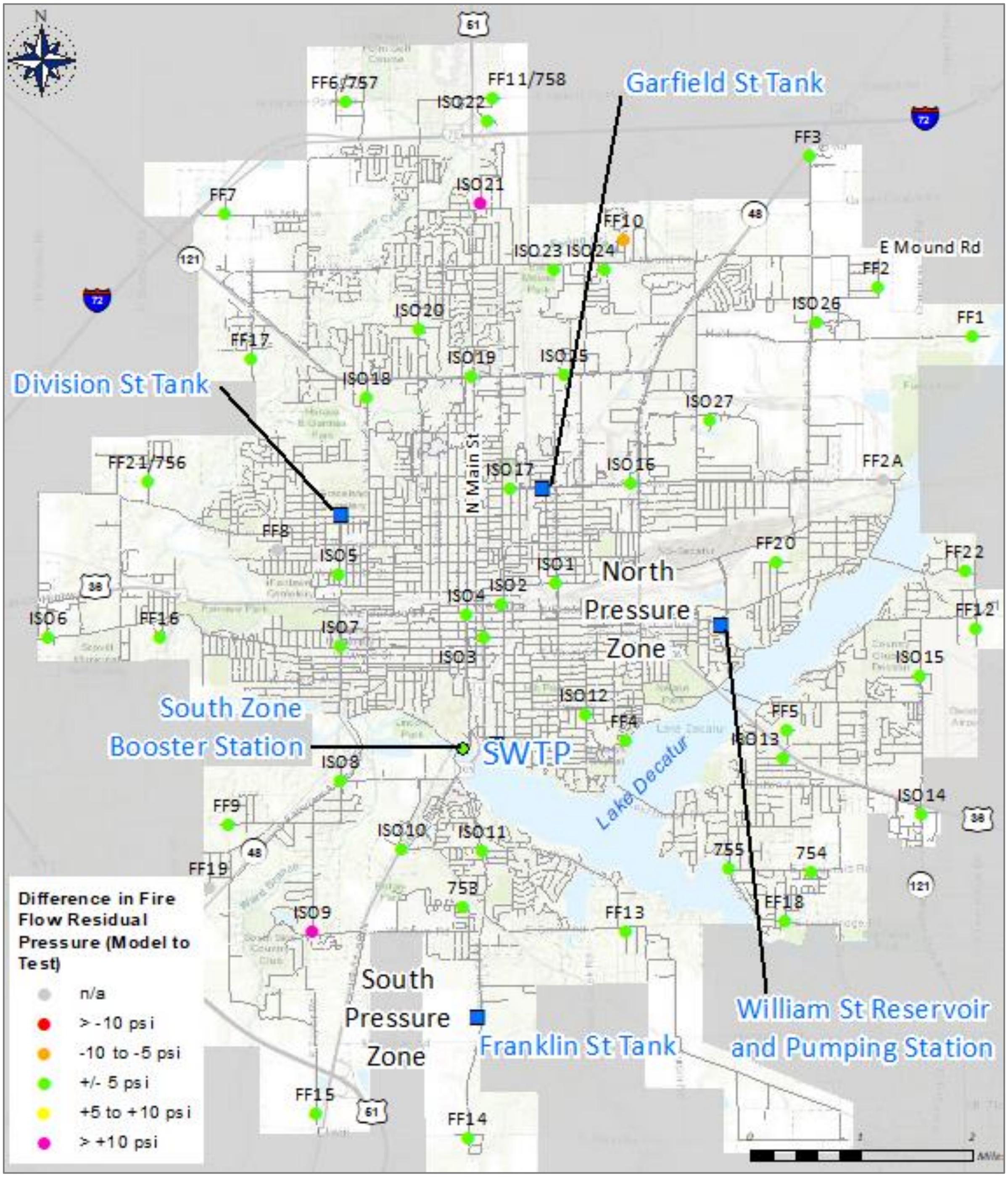

Figure 6-3. 2015 Water Master Plan, 2017 ISO Study and 2020 Fire Flow Tests Locations

Table 6-2 Model Fire Flow Test After Update (2015 Master Plan Tests)

2015 WMP Test Location		Area		Test Data				Model			Note
		Zone	Water Main Size (in)	Total Flow (gpm)	Static (psi)	Residual (psi)	Q_R20 (gpm)	Model Hydrant ID	Residual (psi)	Model - Test (psi)	
1	Rea's Bridge Road	N	6	2,085	64.6	40.6	2,779	670-005	37.9	-3	
2	East Progress Avenue	N	8	965	49.7	39.7	~~2,713~~ 1,713	134-001	40.8	1	typo in original report
2A	East Faries Parkway	N	16	1,700	59.8	44.8	~~1,733~~ 2,848	167-001	52.7	n/a	typo in original report; n/a - noted closed valve during test
3	Brush College Road near Route 48	N	8	1,094	52.7	48.7	3,347	643-001	50.1	1	
4	21st Street	N	6	400	61.7	15.7	369	058-022	15.9	0	Lowest C factor in system: 51
5	Sadowski Ct	N	6	565	68.7	20.6	511	094-012	22.9	2	local main C factor increased to 69
6	Hickory Point Road	N	12	1,032	57.5	50.5	2,448	374-008	47.8	-3	Oakland Ave water main C factor increased to 120; tested again in 2020
7	Needle Road	N	6	923	48.8	40.8	1,781	414-002	39.01	-2	
8	W Leafland Avenue	N	8	816	50.0	40.0	1,577	012-009	48.4	n/a	water main replaced in 2018
9	Rock Springs	S	12	863	61.4	37.4	1,149	202-001	40.3	3	available fire flow seems low for the water main size
10	Camelot Drive	N	8	1,076	55.4	44.4	2,087	216-006	39.0	-5	
11	Hickory Point Road	N	12	965	54.3	38.8	1,421	365-002	38.7	0	
12	Excelsior	N	6	447	53.6	11.6	374	177-015	14.0	2	
13	Wheatland Road	S	6	1,082	58.7	45.7	2,105	237-003	43.4	-2	
14	South Franklin Street	S	8	1,695	60.9	40.9	2,438	576-001	39.2	-2	transmission main C factor increased to 130
15	South Taylor Road	S	12	782	51.6	30.6	939	569-008	32.5	2	available fire flow seems low for the water main size

2015 WMP Test Location		Area		Test Data				Model			Note
		Zone	Water Main Size (in)	Total Flow (gpm)	Static (psi)	Residual (psi)	Q_{R}20 (gpm)	Model Hydrant ID	Residual (psi)	Model - Test (psi)	
16	Hillcrest	N	6	692	62.9	28.9	780	050-014	29.4	0	water man C factor increased to 80 to match pipe age
17	Westlawn Avenue	N	12	1,460	59.9	40.9	2,040	207-007	42.9	2	
18	Lost Bridge Road and 34th St	N	12	863	56.9	44.9	1,488	099-014	49.1	4	
19	File Drive	S	8	258	62.9	10.9	234	203-001	61.4	n/a	
20	Condit and 32nd Street	N	6	1,076	59.7	45.7	1,959	027-013	45.0	-1	
21	Elizabeth Street	N	12	1,945	54.7	37.7	~~3,334~~ 2,828	212-012	28.1	-10	possible typo in 2015 in report; tested again in 2020
22	E William Street	N	16	2,224	59.7	51.7	5,459	176-003	47.6	-4	

Table 6-3 Model Fire Flow Test After Update (2017 ISO Report Tests)

2017 ISO Report Test Location		Area		Test Data				Model			Note
		Zone	Water Main Size (in)	Total Flow (gpm)	Static (psi)	Residual (psi)	Q_{R}20 (gpm)	Model Hydrant ID	Residual (psi)	Model - Test (psi)	
1	Jasper & Sangamon	N	12	2,570	61	52	5,800	019-025	52	0	increased model hydrant lead C factor to match test
2	Cerro Gordo & MLK Drive	N	16	2,200	59	53	6,000	037-004	52	-1	
3	Franklin & Prairie	N	12	3,530	63	55	8,800	037-035	55	0	
4	Eldorado & Main	N	16	3,420	60	40	5,000	042-061	42	2	
5	Marietta & Fairview	N	8	2,510	56	51	7,300	011-011	51	0	
6	Fairway & Main	N	12	1,380	55	36	1,900	221-003	38	2	

2017 ISO Report Test Location		Area		Test Data				Model			Note
		Zone	Water Main Size (in)	Total Flow (gpm)	Static (psi)	Residual (psi)	Q_R20 (gpm)	Model Hydrant ID	Residual (psi)	Model - Test (psi)	
7	Main & Fairview	N	16	1,670	62	57	5,300	046-018	57	0	increased model hydrant lead C factor to match test
8	Taylorville & Fairview	S	6	1,330	83	62	2,400	075-003	62	0	
9	Taylor & West Grove	S	12	1,590	85	46	2,100	191-003	69	23	
10	Rt 51 & Imboden	S	12	1,890	80	64	3,900	081-003	64	0	
11	Imboden & Franklin	S	6	1,240	62	50	2,400	085-012	49	-1	
12	16th & Lincoln	N	6	1,340	66	59	3,700	058-001	58	-1	
13	34th & Maplewood	N	6	1,060	54	44	2,100	095-001	45	1	test possibly performed at 32nd and Maplewood
14	Mt Zion Rd & Maryland	N	12	1,590	64	40	2,200	110-001	44	4	
15	Airport Rd & Powers	N	6	1,430	49	30	1,800	103-019	28	-2	
16	21st & Garfield	N	12	2,760	52	43	5,500	021-019	42	-1	
17	Morgan & Garfield	N	8	2,760	52	45	6,300	017-013	46	1	result suggests hydrant lead connects to 16-inch transmission main
18	Oakland & Corson	N	12	1,780	63	52	3,700	066-015	53	1	
19	Pershing & Water NW corner	N	6	2,580	55	42	4,400	119-033	39	-3	test suggests water main diameter could be 8- or 10-inch
20	MacArthur & Diane	N	8	2,140	52	38	3,300	119-001	35	-3	model connectivity fixed
21	Ash & Prospect	N	8	1,180	60	40	1,700	157-001	53	13	

2017 ISO Report Test Location		Area		Test Data				Model			Note
		Zone	Water Main Size (in)	Total Flow (gpm)	Static (psi)	Residual (psi)	Q_R20 (gpm)	Model Hydrant ID	Residual (psi)	Model - Test (psi)	
22	Hickory Pt Frontage Rd, 1st S of Wingate Dr	N	12	1,160	65	40	1,600	368-005	39	-1	
23	Educational Park & Mound	N	6	1,380	49	30	1,700	122-014	27	-3	water main C factor increased to 125 to match pipe age
24	Moundford Ave, 1st S of Mound	N	6	1,330	52	32	1,700	125-002	34	2	
25	Charles & Pershing	N	12	2,980	48	33	4,200	002-009	36	3	
26	Brush College Rd, 1st S of College Park	N	6	1,060	40	30	1,500	136-007	30	0	
27	Parkway Dr & Parkway Ct	N	12	1,750	48	37	2,900	161-008	38	1	

Table 6-4 Model Fire Flow Test After Update (October 30, 2020 Tests)

2020 Tests		Area		Test Data				Model				Note
		Zone	Water Main Size (in)	Total Flow (gpm)	Static (psi)	Residual (psi)	Pressure Drop (psi)	Model Hydrant ID	Static (psi)	Residual (psi)	Pressure Drop (psi)	
753	Wildwood Dr	S	8	1,870	58	35	23	F: 083-013; M: 083-016	58	35	23	
754	Fitzgerald Rd W of Baltimore Av	N	12	1,820	56	26	30	F: 109-001; M: 112-001	54	27	27	
755	Lost Bridge Road & Fitzgerald Rd	N	12	1,060	65	56	9	F: 097-034; M: 097-028	66	58	8	
756	Elizabeth Street	N	12	2,150	54	23	31	F: 212-011; M: 212-012	56	25	31	
757	Hickory Pt Rd & Oakland Av	N	12	1,000	57	47	10	F: 374-007; M: 374-008	57	48	9	Oakland Ave water main C factor = 120
758	Hickory Pt Rd & Wingate	N	12	1,040	50	40	10	F: 365-003; M: 365-002	52	38	14	

6.1.3.4 Updated Model Statistics

Table 6-5 summarizes the number of model elements after model update. **Figure 6-4** and **Figure 6-5** show the distribution of model pipe diameter and C factor, respectively. 6-inch local water mains make up 305 miles, or roughly 57%, of the distribution system. On the other hand, 16-inch diameter and larger transmission mains add up to 50 miles or 9% of the system.

Table 6-5. Element Count for the Updated WaterGEMS Model

Type	Original Count	After Update Count	Note
Junction	24,757	24,637	Water demand is distributed to 11,085 model junctions across the system.
Pipe	30,215	30,154	532 miles of water mains; average model pipe length is 93 ft, diameter ranging from 0.8 to 54 inches; assigned associated GIS-ID
Tank	5	5	Division St tank, Franklin St tank, Garfield Ave tank, William St reservoir, SWTP reservoir
Reservoir	1	1	at SWTP
Hydrant	4,137	4,197	added new model hydrants
Pump	12	12	SWTP, SZBS and WSPS
Check valve	4	4	3 at SZBS, 1 used to separate North/South zone
Isolation valve	10,124	10,021	1 closed to separate North/South zone; 1 closed to separate WSPS inflow/outflow line; others left open in model simulation
Flow control valve	4	4	
Pressure reducing valve	2	2	
Throttle control valve	1	1	

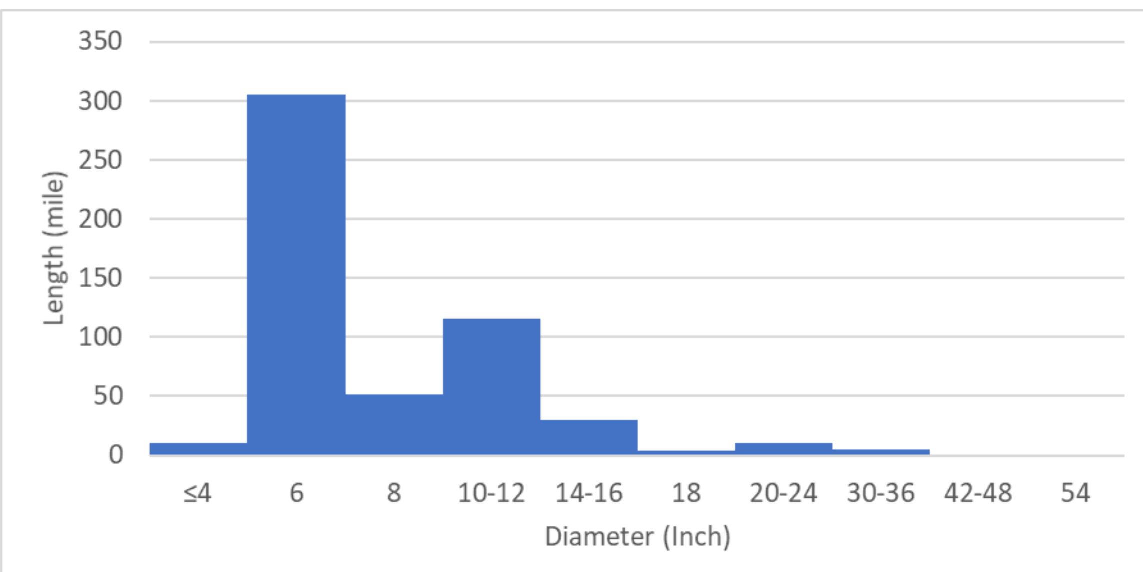

Figure 6-4. Distribution of Model Water Mains Diameter

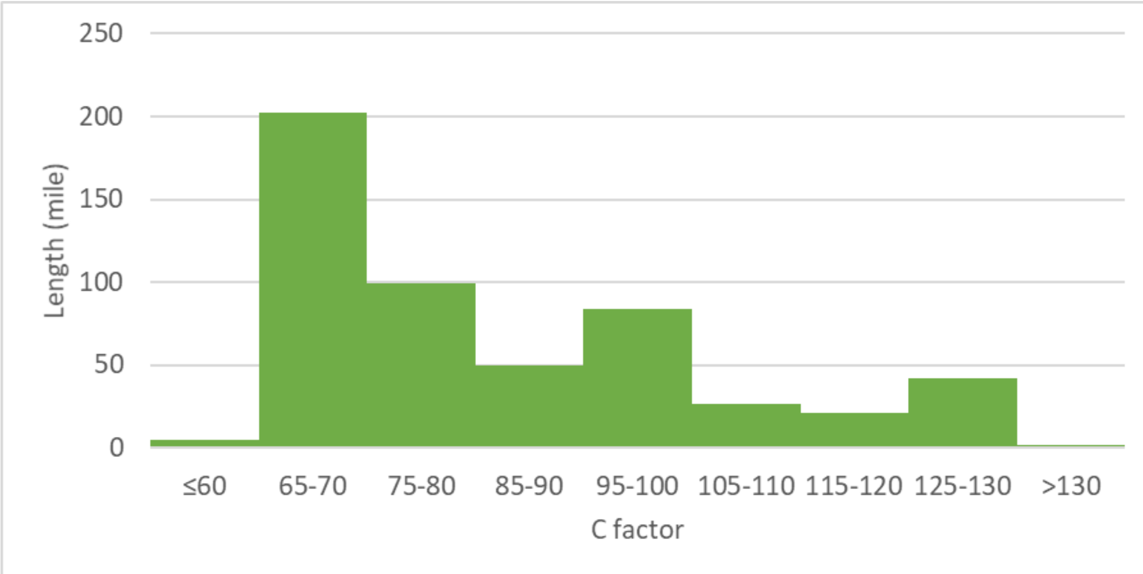

Figure 6-5. Distribution of Model Water Mains Roughness (C Factor)

Figure 6-7 color-codes the model pipe by C factor in the system map. Pipes with higher C factors (blue lines in map) are found at the outskirts of the system where newer water mains are. They are also found in pockets in the system where water mains were replaced. Inherited from the previous model version, the C factors for the local water mains are mostly 70 (orange) in the North Pressure Zone and 80 (yellow) in the South Pressure Zones.

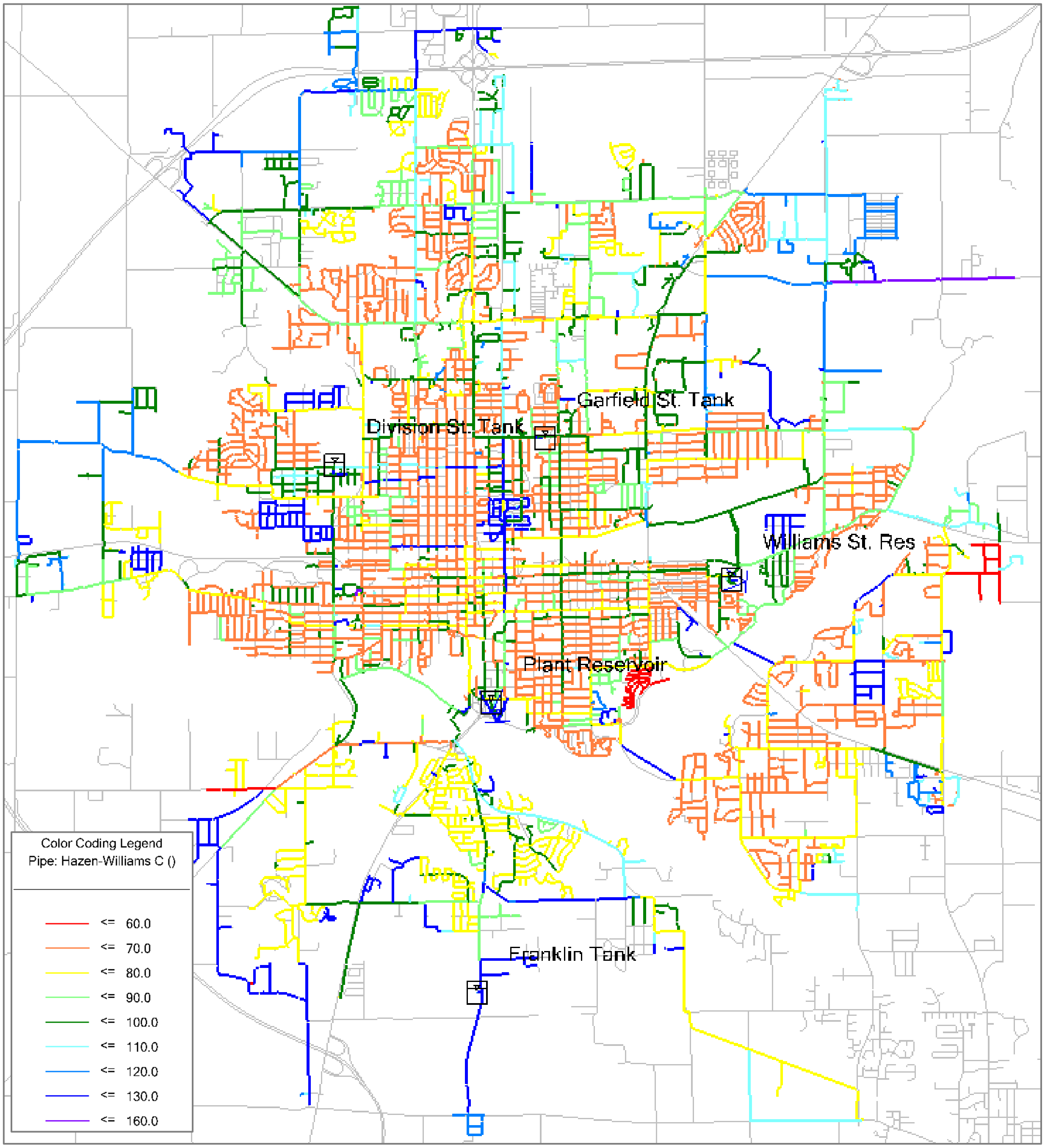

Figure 6-6. Updated Model C Factor in the Water Distribution System

6.2 System Performance Evaluation

The updated hydraulic model was utilized to evaluate system performance for water pressure, available fire flow, water age and residual chlorine concentration. The water demand projections applied depend on the evaluated metrics.

All model simulations in this study were performed in WaterGEMS CONNECT Edition Update 3.

6.2.1 Water Demand Used in Model Simulation

Table 6-6 summarizes the water demand applied to the model simulations. **Section 3** discusses the development of water demand forecast. Because the 2020 – 2050 projections vary little for normal weather and expected growth condition, the projected 2020 demands were used for all simulations. It ranged from 14.1 MGD for the minimum day demand to 29.3 MGD for the peak hour of maximum day demand. The North and South Pressure Zones were assumed to consume 93.5% and 6.5% of the system demand, respectively, which is based on historical data. ADM and Tate & Lyle, the largest users in the system, were assumed to use 56% of the system demand.

Table 6-6. System Water Demand (in MGD) by Scenario/Year

Year	Average Day	Maximum Day	Peak Hour	Minimum Day
2020	18.6	23.4	29.3	14.1

6.2.2 Pressure

6.2.2.1 Model Setup

Both steady state and extended period simulation models were set up to simulate system pressure. That includes simulating the average day, maximum day and peak hour demands. **Table 6-7** summarizes the water demand and pump operation of the scenarios. In steady state simulations, pumps were turned on to meet average or maximum day demand. Tank water elevations were set at average level. In extended period simulations, pumps were operated to maintain tank water levels in normal ranges.

Table 6-7. Model Setup to Evaluate System Pressure

Scenario	Demand (MGD)	Pump Operation			Tank/Reservoir Elevation (ft)		
		HSPS	WSPS	SZBS	Division/ Garfield	Franklin St	William St
Average Day – Steady State	18.6	#2	off	off	800	830	667
Average Day – Extended Period	18.6	#3, #4	#1	#1	794 - 805	821 - 839	663 - 671
Maximum Day – Steady State	23.4	#2	#1	off	800	830	667
Peak Hour (WSPS on) – Steady State	29.3	#2	#1	#1	800	830	667
Peak Hour (WSPS off) – Steady State	29.3	#4	Off	#1	800	830	667

CDM Smith

6.2.2.2 Simulation Results and Findings

6.2.2.2.1 Pressure

Figure 6-7 is a box plot[1] illustrating the distribution of pressures across the North and South Pressure Zones during average day, maximum day and peak hour of maximum day conditions. **Figure 6-8** and **Figure 6-9** present the pressure contours during average day and peak hour of maximum day conditions, respectively. Pressures are maintained between 35 and 100 psi for the entire system except at Manchester Court during peak hour condition. Pressures in the South Pressure Zone are on average 10-15 psi higher than in the North Pressure Zone.

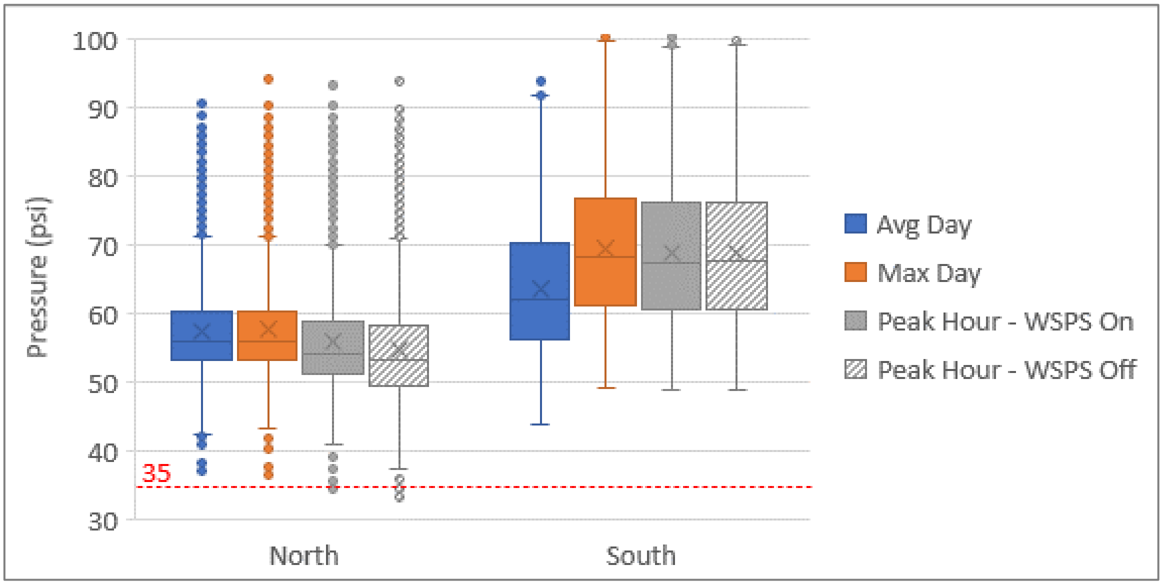

Figure 6-7. System Pressure in Average Day, Maximum Day and Peak Hour Demand

6.2.2.2.2 Hydraulic Grade Line (HGL)

The hydraulic grade line (HGL) drops (head loss) as water travels away from the SWTP because of friction loss in the pipe and minor loss due to transition, elbows, fittings and valves in the system. The head loss is steeper close to SWTP and SZBS and levels out after a distance of approximately 4 miles from the SZBS. Head loss is also higher in days with higher water demand. In the North Pressure Zone, the HGL at the furthest point is roughly 20 feet lower than HSPS in average day condition, and up to 30 feet lower in maximum day condition (**Figure 6-10**). In the South Pressure Zone, the HGL drops by 10 feet at the furthest point in average day, and 20 feet in maximum day (**Figure 6-11**). The drop in minimum HGL beyond 4 miles from SZBS is at the southeast corner of the South Pressure Zone. It has a 12-inch diameter transmission main serving Mt Zion and an industrial user.

[1] Box plots (or box-and-whisker plots or box-whisker plots) give a good graphical image of the concentration of the data. The box is drawn from first to third quartile, meaning 50% of the data is in the box. The horizontal line in the middle to denote the median, and the marker denotes average. The whiskers extend to the smallest and largest data points within 1.5 times the interquartile range below the first and above the third quartiles. Dots are outside the whiskers are considered outliers.

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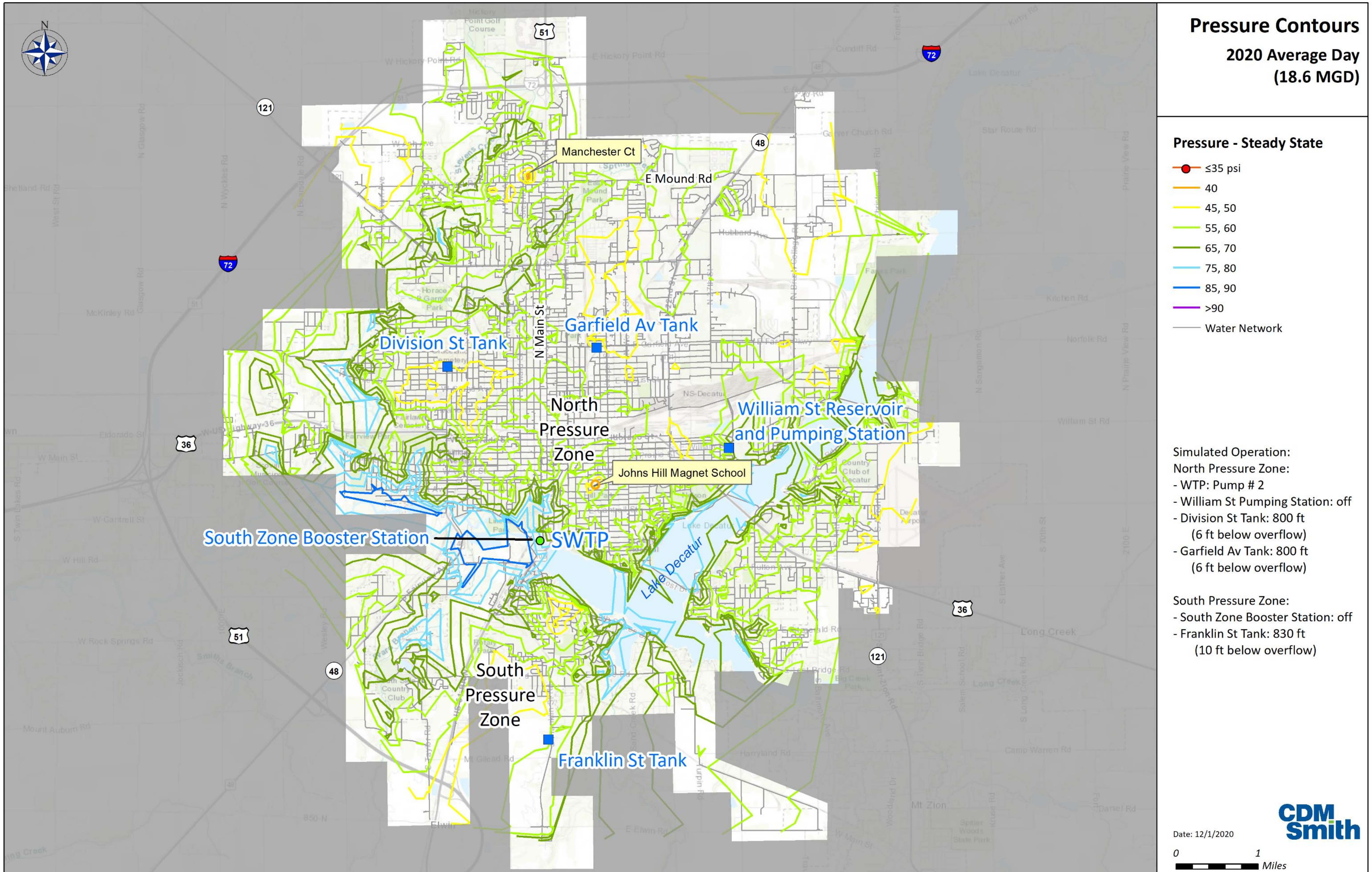

Figure 6-8. Pressure Contours (2020 Average Day Condition)

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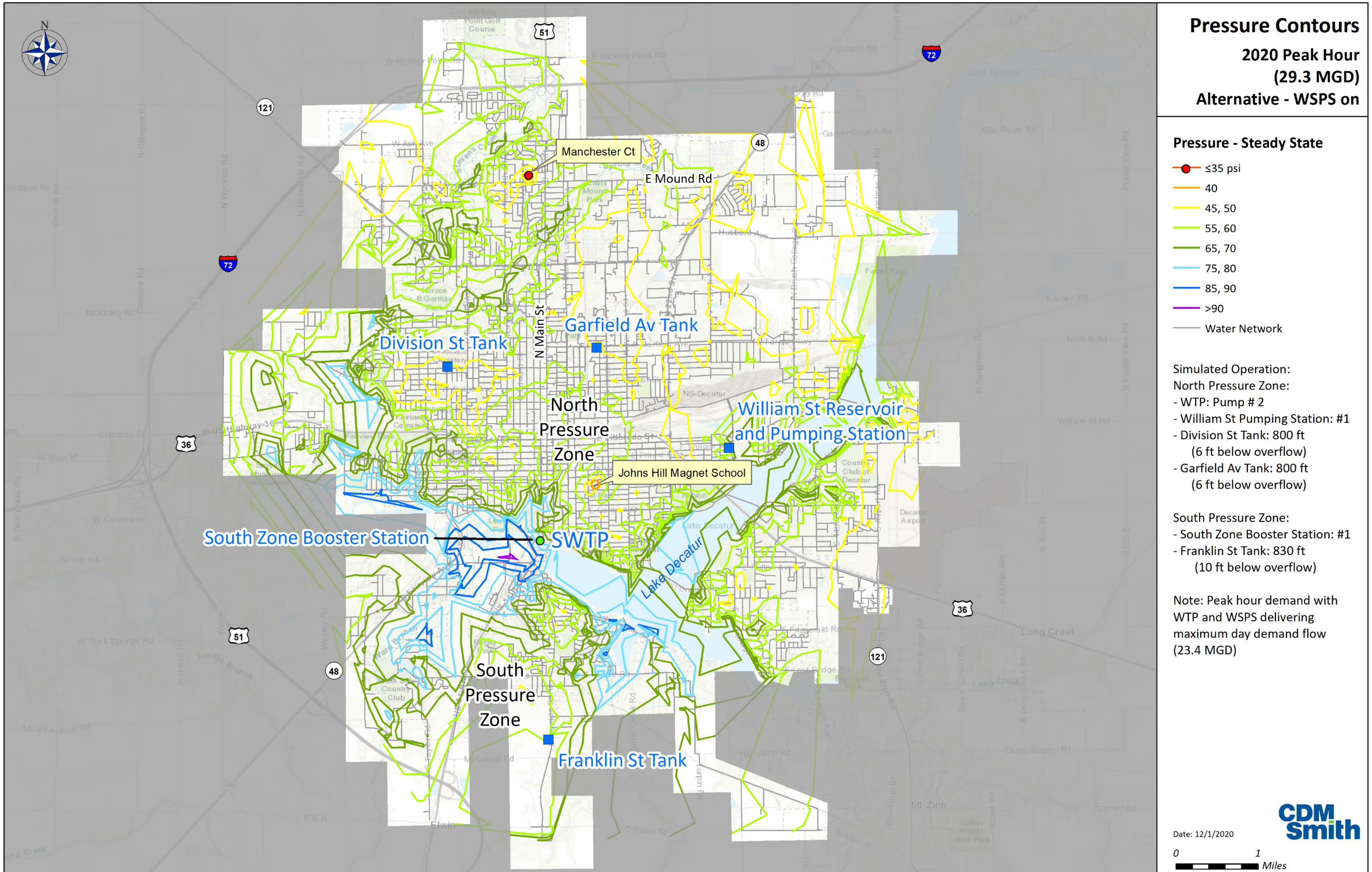

Figure 6-9. Pressure Contours (2020 Peak Hour Condition – WSPS On)

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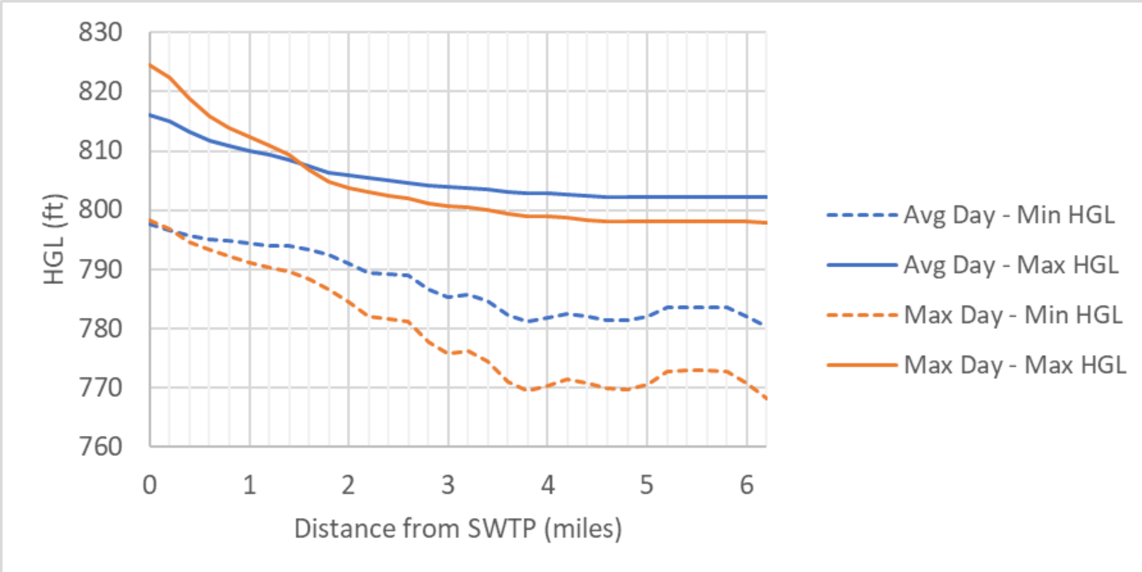

Figure 6-10. HGL vs. Distance from SWTP in North Pressure Zone

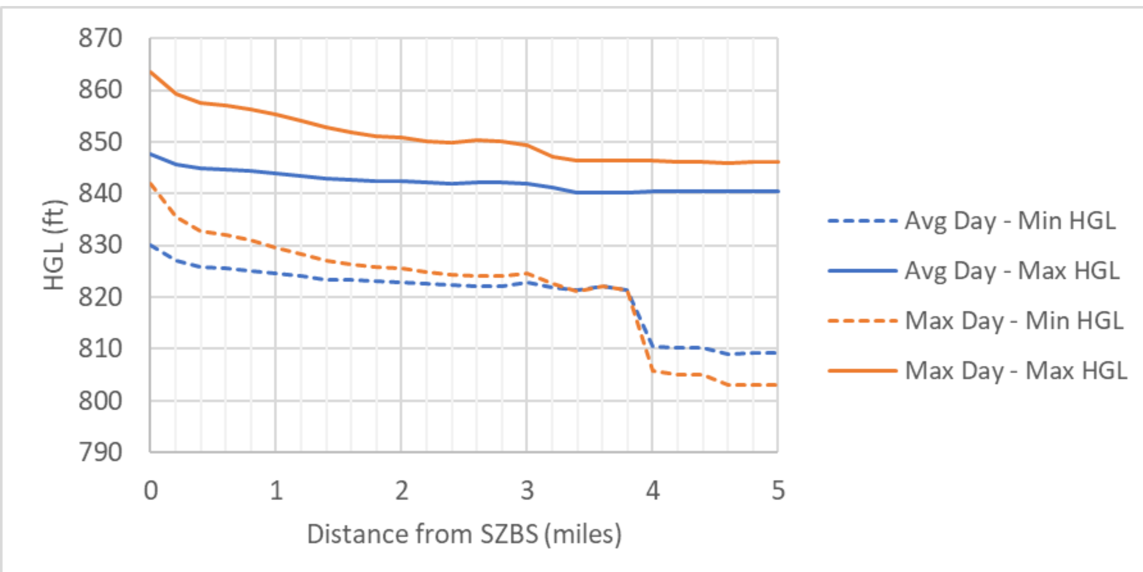

Figure 6-11. HGL vs. Distance from SZBS in South Pressure Zone (when SZBS in operation)

6.2.2.2.3 Locations with Pressures Below 40 psi

The lowest pressures in the system are at Manchester Court and Johns Hill Magnet School, which are among the highest points in the North Pressure Zone. Pressure at Manchester Court was below 35 psi in peak hour steady state simulations, and below 35 psi for approximately an hour per day in average day extended period simulation. **Figure 6-12** is a profile view illustrating the gradual reduction in HGL (blue line) from SWTP to Manchester Court during peak hour condition. The HGL drops by 10 feet from SWTP to Garfield Avenue Tank, and by another 15 feet to Manchester Court. The yellow dash line indicates the elevation equivalent to 35 psi, and it is above the HGL at Manchester Court. Once the HGL in the Garfield Avenue Tank drops below 798 ft (or 27 ft in terms of tank water level), the pressure at Manchester Court may drop below 35 psi.

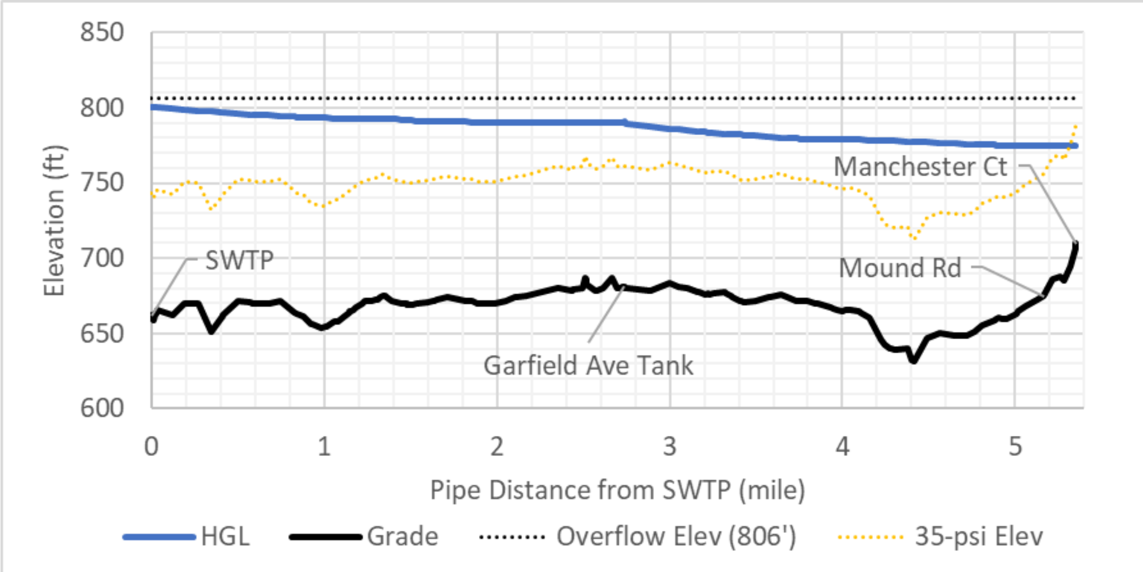

Figure 6-12. HGL Profile from SWTP to Manchester Court

Table 6-8 Model vs. Recorded Pressure at Manchester Ct and Johns Hill School (October 30, 2020 Data)

2020 Tests		Area		Test Data	Model		
		Zone	Water Main Size (in)	Pressure (psi)	Model Hydrant ID	Pressure (psi)	Difference (psi)
P1	Johns Hill School	N	6	36	039-015	36	0
P2a	Manchester Ct	N	6	38	155-013	37	-1
P2b	Olympic Ct	N	6	44	155-003	43	-1
P2c	Mound Rd	N	12	48	155-002	48	0

On the other hand, pressures at Johns Hill Magnet School were between 36 and 40 psi for the simulated scenarios.

Additional pressure monitoring took place on 10/30/20 afternoon at both locations. The recorded pressures were 38 psi at Manchester Court, 39 psi at Olympic Court and 36 psi at Johns Hill Magnet School. The model was able to replicate these measurements (**Table 6-8**). It is noted that the Water Utility did not receive residents' complaints of low pressure in these areas.

6.2.2.2.4 Locations with Pressures Approaching 100 psi

The highest pressures are in the South Pressure Zone. They are located between SZBS and South Side Drive, close to the Sangamon River and boundary between the two pressure zones. Pressure is high because of low elevation. Pressure may exceed 100 psi when SZBS is in operation.

6.2.2.2.5 William Street Pumping Station

The two peak hour demand simulations suggest that the William Street Pumping Station is critical in boosting pressure in the east side of North Pressure Zone. Pressure would drop below 40 psi in the furthest corner in the east if the pumps were off during peak hour condition.

6.2.2.2.6 Transmission Mains to Largest Water Users

The existing distribution system has sufficient redundancy to the largest water users located in the North Pressure zone. However, significant users in the South Pressure Zone (i.e. PPG and the Village of Mt. Zion are both served by a 12-inch diameter main. The 2015 Master Plan Report (Figure 5.05-1) proposed adding a 12-inch diameter main to provide looping in the south east corner of the South Pressure Zone to improve redundancy in this pressure zone, and for these significant users (Strand, 2015).

6.2.3 Available Fire Flow

6.2.3.1 Model Setup

Available fire flow (at 20 psi) at each model hydrant was calculated by simulating the model in maximum day water demand condition. **Table 6-9** summarizes the model setup. The normal operation scenario turns on one pump at HSPS and SZBS to satisfy maximum day demand, while the max pump scenario turns on all available pumps at HSPS (minus pump #5) and SZBS. Pumps #1 and #3 were turned on at William Street Pumping Station to boost available fire flow for the east side in both scenarios. Tank water elevations were set at average of normal operating range.

Table 6-9. Model Setup to Evaluate Available Fire Flow

Scenario	Demand (MGD)	Pump Operation			Tank/Reservoir Elevation (ft)		
		HSPS	WSPS	SZBS	Division/ Garfield	Franklin	William St
Maximum Day – Normal	23.4	#4	#1, 3	#1	800	830	667
Maximum Day – Max Pump	23.4	#1, 2, 3, 4	#1, 3	#1, 2	800	830	667

6.2.3.2 Simulation Results and Findings

Figure 6-13 is box plots showing the distribution of available fire flows at the hydrants in North and South Pressure Zones in both scenarios. **Figure 6-15** and **Figure 6-16** are maps of available fire flow across the system. Hydrants connected to the transmission mains have the highest available fire flow, more often exceeding 6,500 gpm (blue dots in maps). Areas with low fire flows (red dots) are located at dead end and at residential neighborhoods with older 4- or 6-inch diameter water mains. 1% (or 44) of the hydrants have available fire flow less than 500 gpm, which is the minimum ISO fire flow protection requirement for public water supply system. These hydrants are all served by 4- or 6-inch diameter water mains (**Figure 6-14**).

The impact of turning on more pumps at the SWTP diminishes away from the SWTP. Available fire flow could be boosted by 10-30% for hydrants within 1-mile radius of SWTP in the North Pressure Zone. At the edge of the system, the boost would be between 5% and 15%. In the South Pressure Zone, turning on both SZBS Pumps #1 and #2 would yield 15% higher available fire flows compared to turning on one pump.

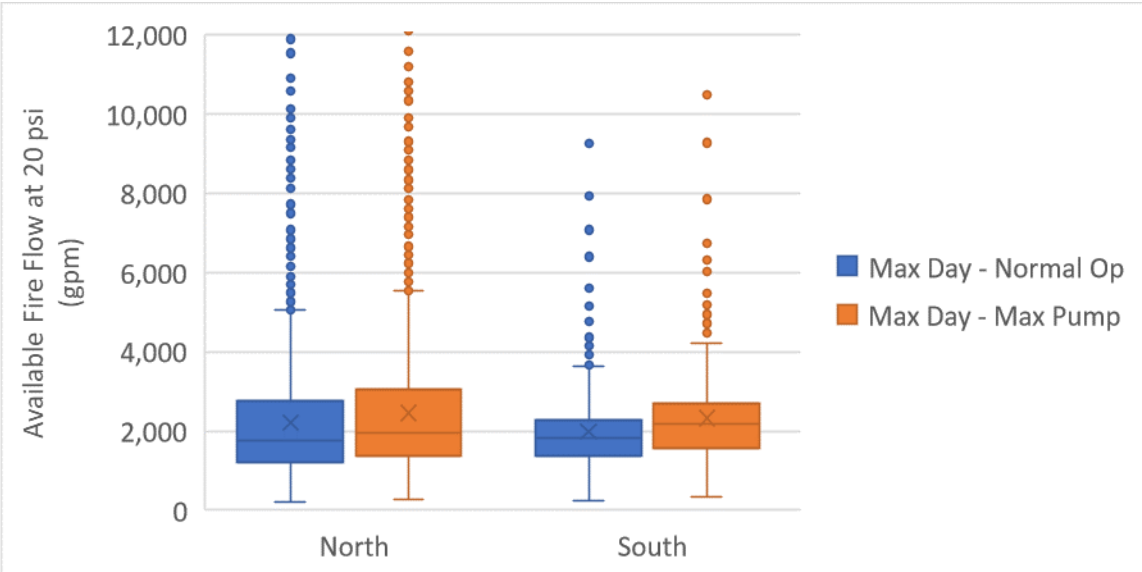

Figure 6-13. Available Fire Flow during Maximum Day Demand

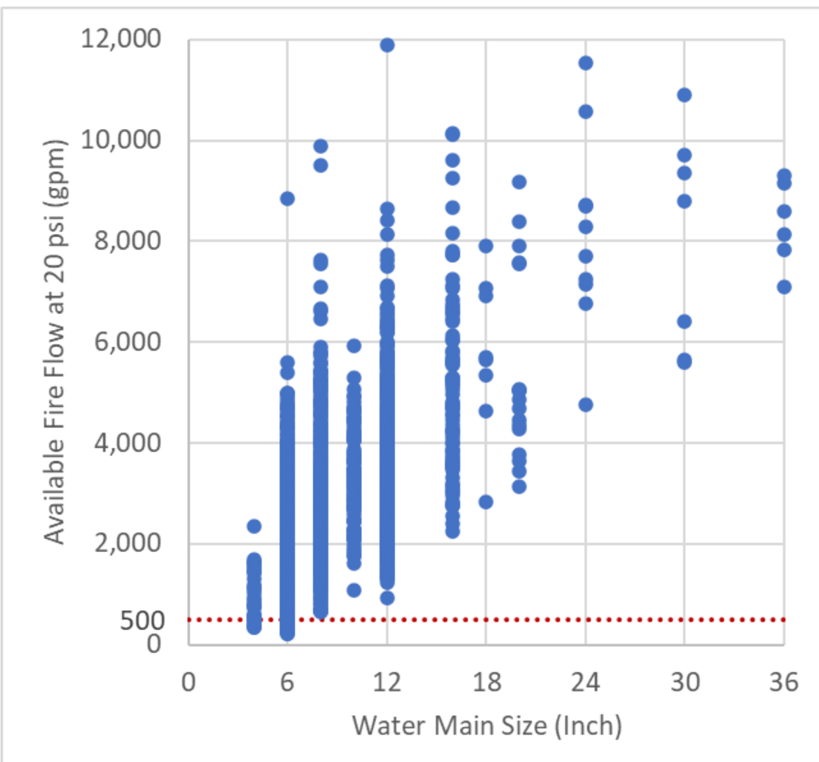

Figure 6-14. Available Fire Flows vs. Water Main Diameter

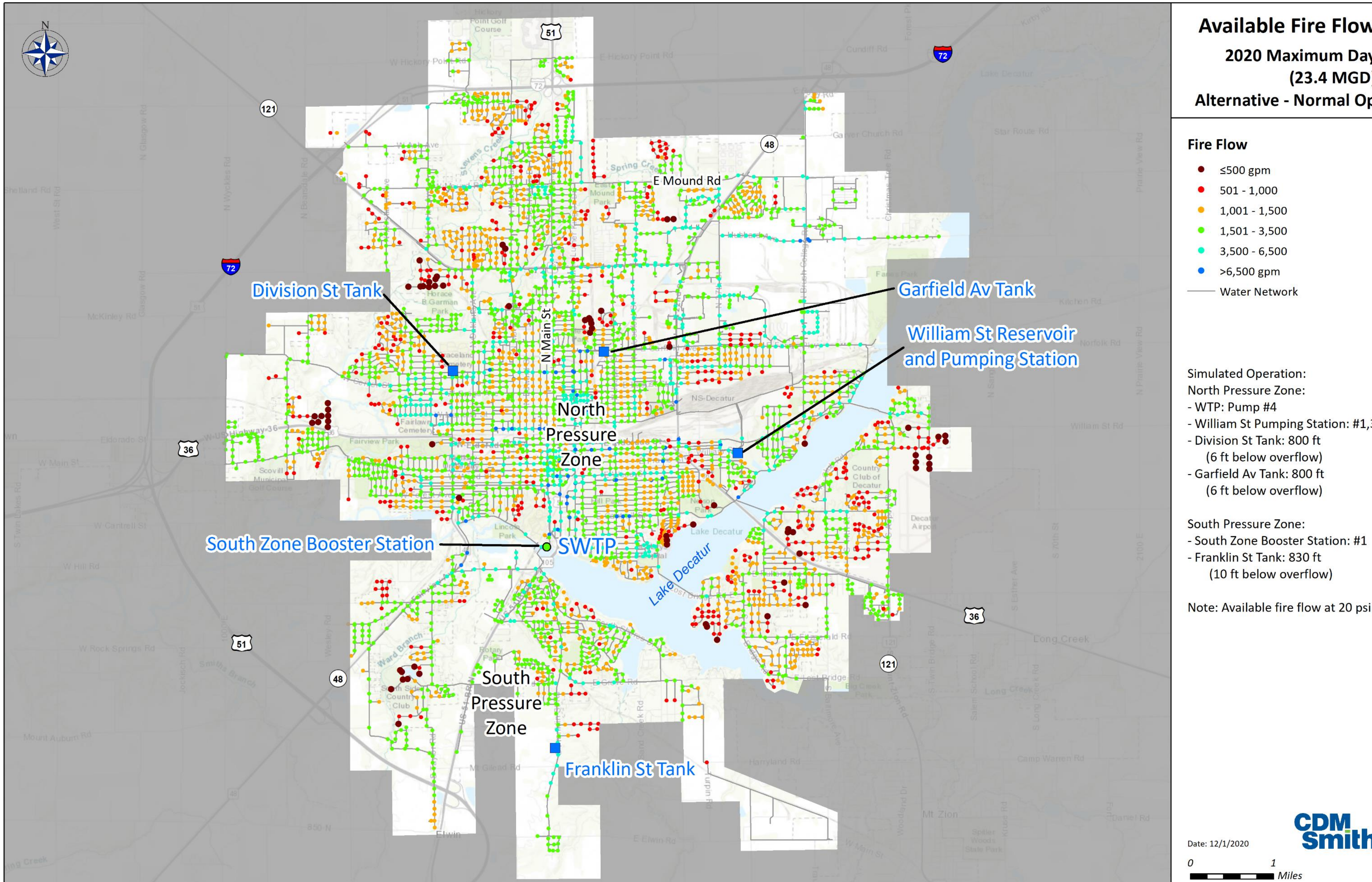

Figure 6-15. Available Fire Flow for North Pressure Zone Hydrants in Max Day – Normal

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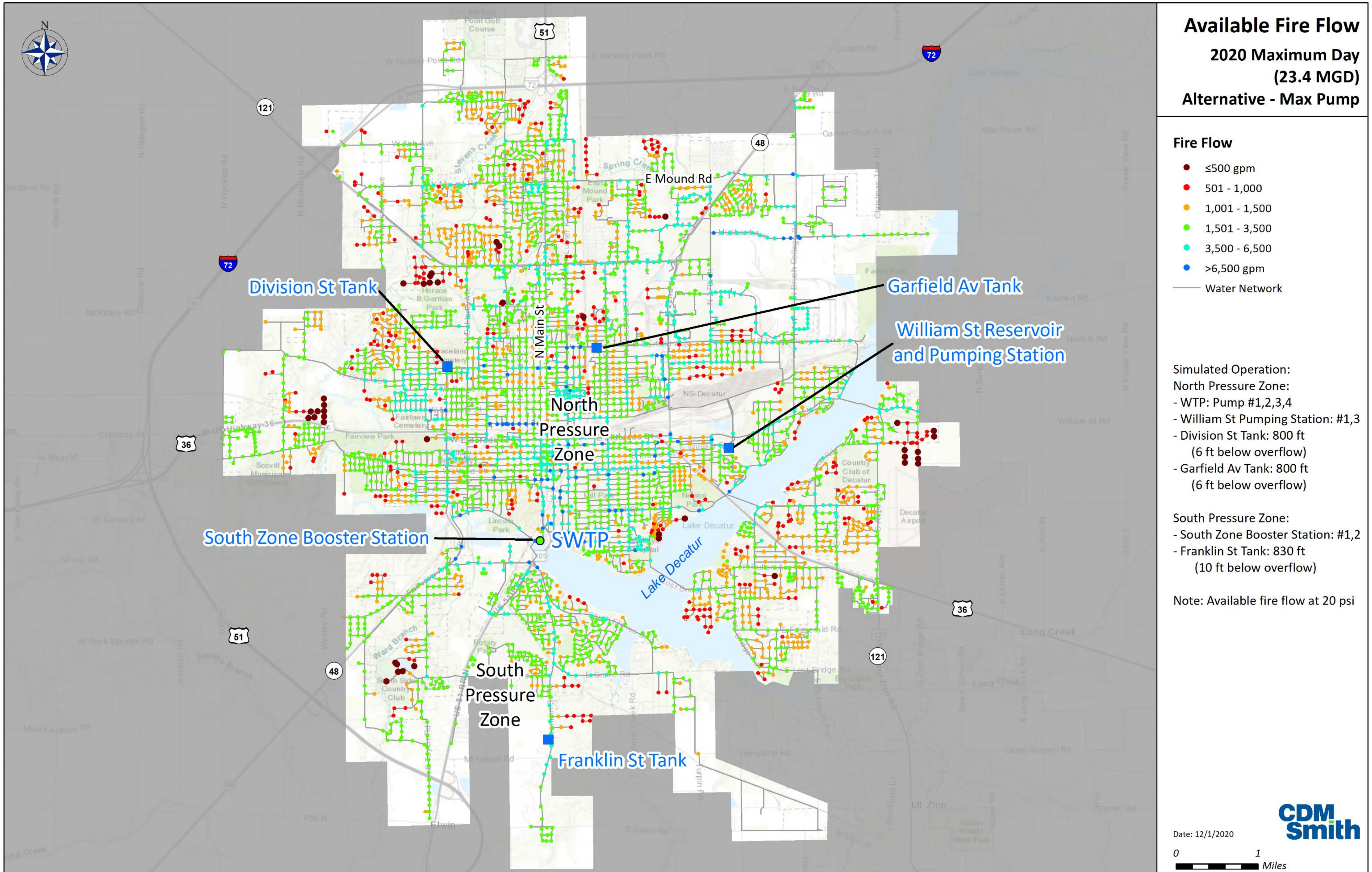

Figure 6-16. Available Fire Flow for North Pressure Zone Hydrants in Max Day – Max Pump

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6.2.3.3 Available vs. Desired Level of Fire Flow

The ISO has prepared guidance on estimating the amount of water needed for municipal fire protection. It suggests a public water supply system should provide 500 to 3,500 gpm for fire protection. For individual properties, the Needed Fire Flow depend on construction type, occupancy type, building area and exposure. In previous Master Plan Update report (Strand, 2015), a performance standard was established for desired fire flow capacity based on zoning. The listed standard was 1,500 gpm for residential, 2,500 gpm for commercial, 3,500 gpm for light industrial and 7,000 gpm for heavy industrial areas.

Model simulation suggests that up to 55% of the hydrants would satisfy the previous Master Plan Update performance standard. These criteria are higher than what are typically used for water distribution master planning and are impractical for the Water Utility due to the large list of deficiencies. Therefore, a range of criteria were evaluated to look for achievable fire flow standards and are summarized in **Table 6-10**. Similar to the ISO suggestion, these criteria range between 500 and 3,500 gpm and vary by land use. Currently, available fire flow is less than 500 gpm for 1% of the hydrants, and less than 1,000 gpm for at 10% to 13% of the hydrants.

Table 6-10. Hydrants Exceeding Various Desired Level of Available Fire Flow

#	Desired Level of Fire Flow by Land Use Type (gpm)				Percent of Hydrants Exceeding Criteria	
	Residential	Commercial	Light Industrial	Heavy Industrial	Normal Op	Max Pump
1	500				98%	99%
2	1,000				87%	90%
3	1,000	2,000			77%	82%
3	1,000	2,000		3,500	75%	81%
4	1,500				62%	69%
5	1,500		3,500		55%	63%
6	1,500	2,500	3,500	7,000	48%	55%

#6 is the *2015 Water Master Plan Update* standard

Section 6.6.2 discusses the incremental improvements needed to bring the available fire flows of all the hydrants above 500 gpm and 1,000 gpm. Further improvement may be financially infeasible for the Water Utility. It is recommended to perform fire flow tests at these low flow hydrants for verification and determining the need for future enhancement.

6.2.4 Water Age

6.2.4.1 Model Setup

Extended period simulations were performed to determine the range of water age experienced across the system. Models were assembled to simulate the 2020 minimum, average and maximum demand day conditions. For each scenario, the models were run for a total of 384 hours (16 days) with the same daily demand and 96-hour diurnal curves to allow the water age to stabilize. Average water ages were calculated from the last 24 hours of simulation for the model junctions with water demand. **Table 6-11** summarizes the model setup for water age simulations. The range of operating water levels of the elevated tanks and reservoirs were based on August 2019 data.

Table 6-11. Model Setup to Evaluate Water Age

<table>
<tr><th rowspan="2">Extended Period Simulation Scenario</th><th rowspan="2">Demand (MGD)</th><th colspan="3">Pump Operation</th><th colspan="3">Tank/Reservoir Elevation (ft)</th></tr>
<tr><th>HSPS</th><th>WSPS</th><th>SZBS</th><th>Division/ Garfield</th><th>Franklin</th><th>William St</th></tr>
<tr><td>Minimum Day</td><td>14.1</td><td>#3</td><td>off</td><td rowspan="3">#1</td><td rowspan="3">794 - 805</td><td rowspan="3">821 - 839</td><td>refill mode*</td></tr>
<tr><td>Average Day</td><td>18.6</td><td>#3, #4</td><td rowspan="2">#1</td><td rowspan="2">663 - 671</td></tr>
<tr><td>Maximum Day</td><td>23.4</td><td>#2, 3, 4</td></tr>
</table>

* the minimum capacity at HSPS is higher than minimum day demand. The EPS model was set up to use the excess supply by refilling the William Street reservoir

6.2.4.2 Simulation Results and Findings

Figure 6-17 is the water age exceedance curves for the simulated scenarios. Demand-weighted average of the water ages range from 19 hours in maximum day to 27 hours in minimum day scenarios. Water ages for 95% of the system demand were less than 3.0, 2.2 and 1.8 days in minimum, average and maximum demands scenarios, respectively (5% exceedance). **Figure 6-18** shows the respective exceedance curve for North and South Pressure Zones. Between the two pressure zones, water age in the South Pressure Zone is on average 24 hours higher than the North Pressure Zone. The South Pressure Zone has half the water demand per inch-mile[2] of water main than the North Pressure Zone. Therefore, water in the South Pressure Zone stays in the system longer before being consumed and are more sensitive to change in water demand.

Figure 6-19 to **Figure 6-21** are water age maps in minimum, average and maximum day conditions. In the North Pressure Zone, water age is generally less than 2 days within 3-mile radius from the SWTP (blue and green dots in figures). The highest water ages were found in dead end water mains and outskirts of the system. That includes the residential neighborhoods north of Center Street and Elizabeth Street on the west side, Hickory Point Road area in the north, IL-48 and I-72 to the northeast, and Country Club Road and William Street Road east of Lake Decatur.

In the South Pressure Zone, water age is the lowest adjacent to the SZBS, and increase towards south. The Taylor Road branch and Franklin Street Road branch extending south to Elwin Road has the highest water age, exceeding 7 days in all scenarios (black dots in figures).

Section 6.6.3 revisited some of the proposed water main loops at the outskirt of the system in the previous Master Plan report (Strand, 2015), and evaluated their impact in water age. In 2020, Strand recalibrated the model used in the previous master plan report, updated the model, and performed chlorine residual samples as several locations to model and evaluate additional scenarios to improve chlorine residual in the system (Strand, 2020).

[2] Inch-mile (inch of diameter per mile of pipe) is pipe diameter in inches multiply by pipe distance in mile

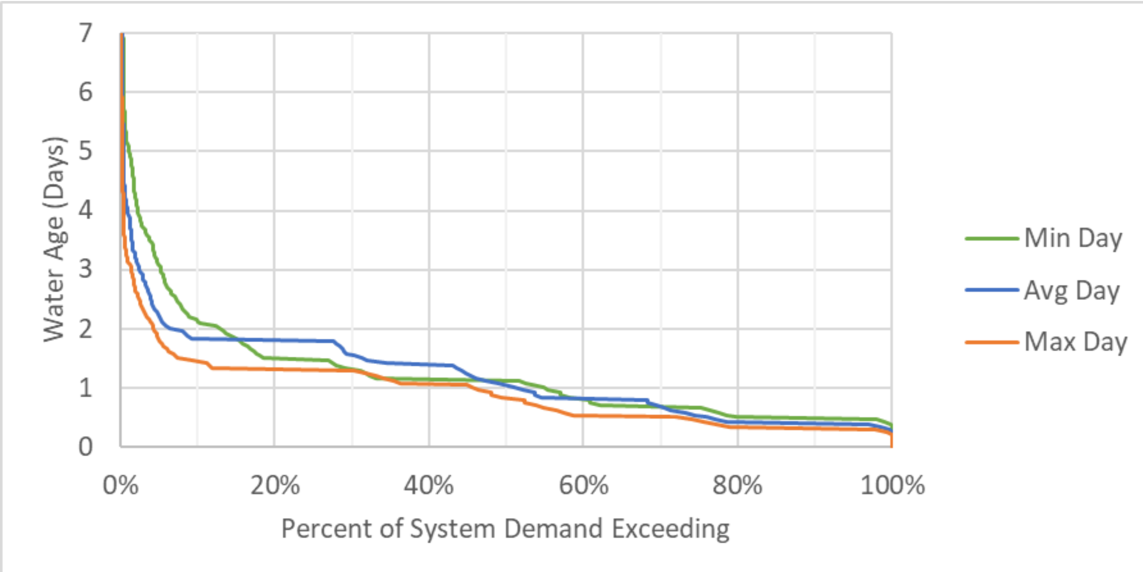

Figure 6-17. Water Age Exceedance Curve for Minimum, Average and Maximum Day Conditions

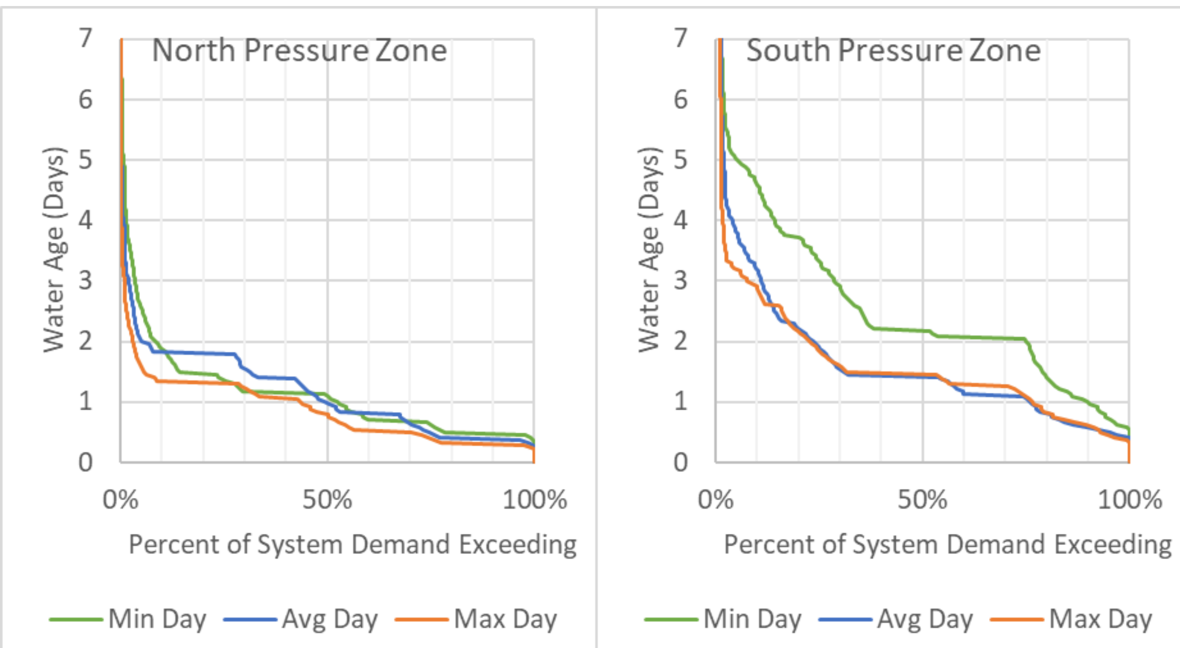

Figure 6-18. Water Age Exceedance Curve for Minimum, Average and Maximum Day Conditions for North and South Pressure Zones

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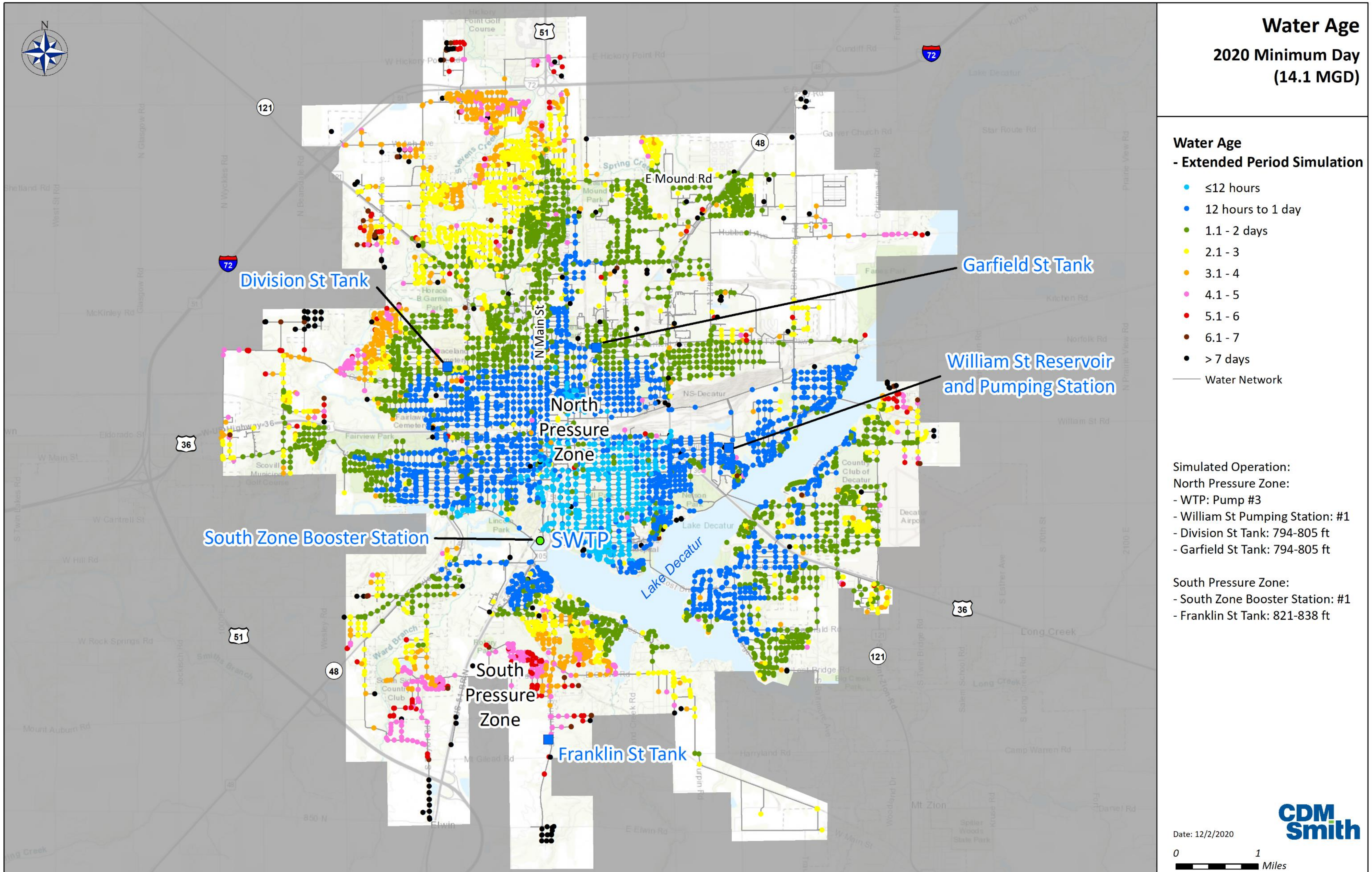

Figure 6-19. Water Age with Minimum Day Demand

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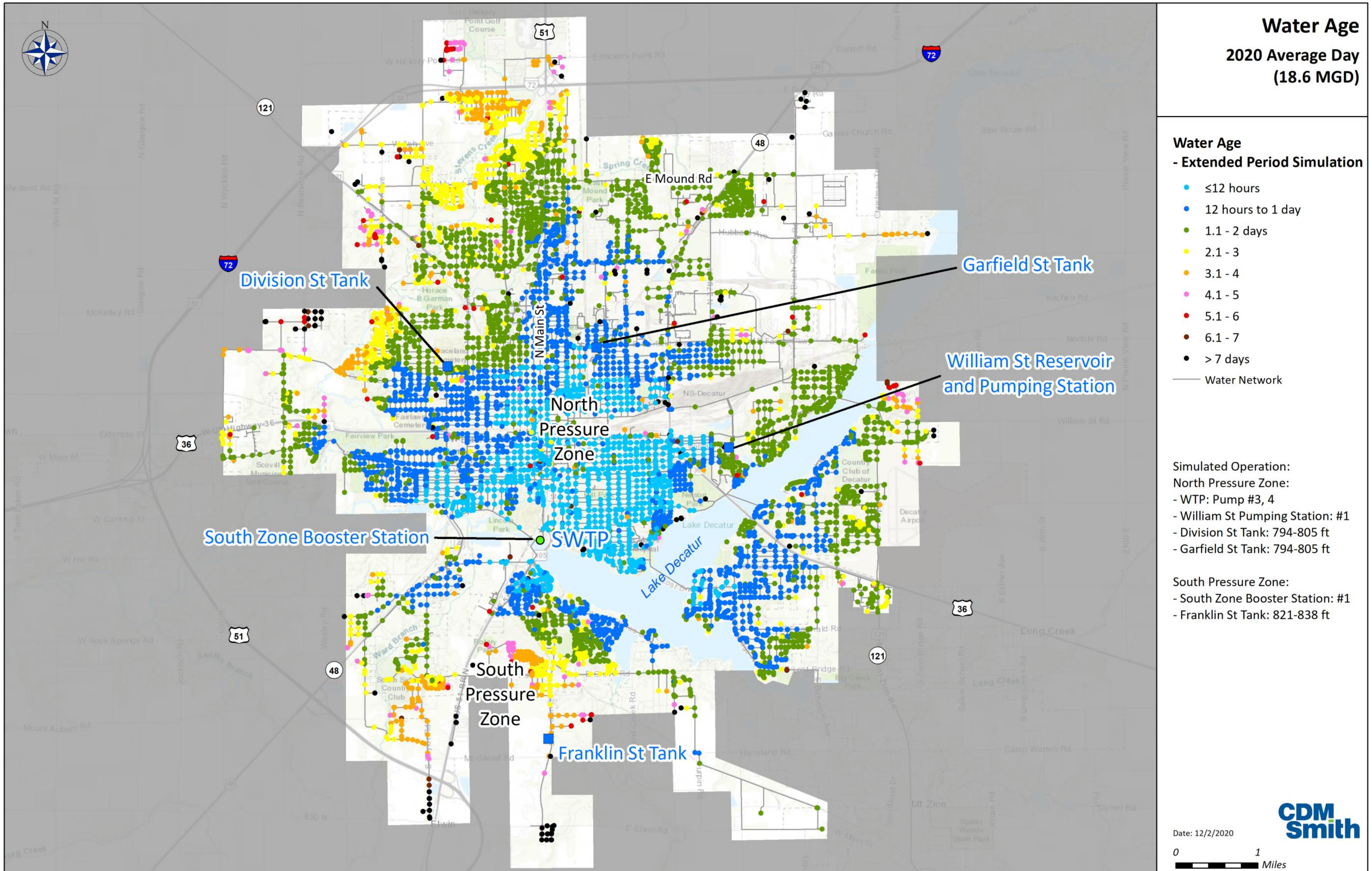

Figure 6-20. Water Age with Average Day Demand

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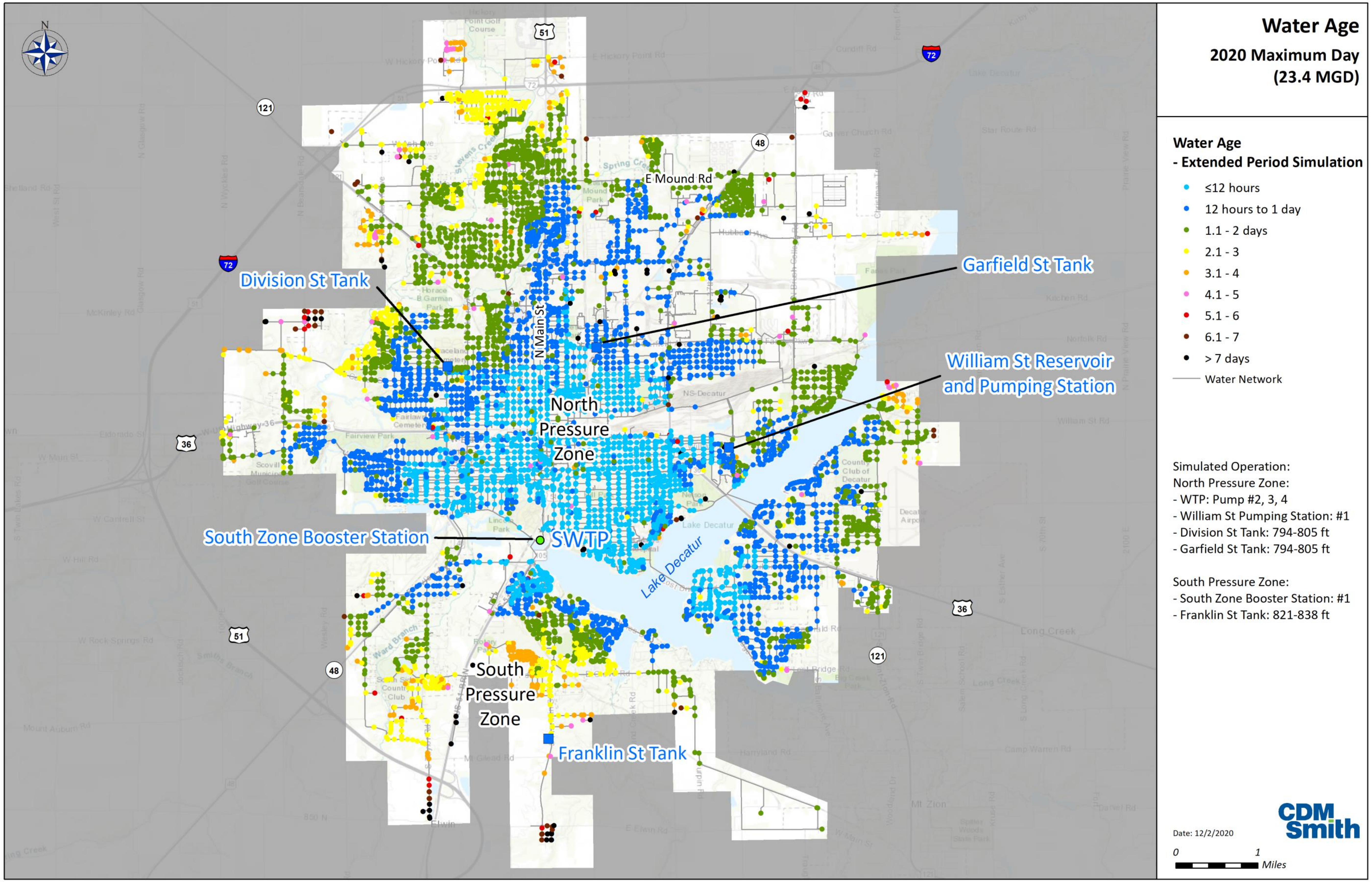

Figure 6-21. Water Age with Maximum Day Demand

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6.2.5 Chlorine Residual

6.2.5.1 Model Setup

Table 6-12 summarizes the model setup for chlorine residual simulations. Similar to the water age analysis, the chlorine residual concentrations were evaluated using extended period simulations. Chlorine residual concentrations were simulated for minimum, average and maximum day conditions. For each scenario, the models were run for a total of 384 hours (16 days) to allow the water age to stabilize.

It is assumed that the free chlorine concentration is 1.52 mg/L leaving the SWTP HSPS pumps. The *EPS Modeling and Chlorine Residual Maintenance Plan* (Strand, 2020) has derived the decay coefficient in the system from 26 samples. The average decay coefficient is -0.492 per day, with 99.7% confidence level between -0.648 per day and -0.336 per day. To be conservative, the lower confidence value of -0.648 per day was used for model simulations.

Average chlorine residual concentrations were calculated from the last 24 hours of simulation for the model junctions with water demand.

Table 6-12. Model Setup to Evaluate Chlorine Residual Concentration

Scenario	Demand (MGD)	Pump Operation			Tank/Reservoir Elevation (ft)			SWTP Chlorine Conc (mg/L)
		HSPS	WSPS	SZBS	Division/ Garfield	Franklin	William St	
Minimum Day	14.1	#3	off	#1	794 - 803	821 - 839	refill	1.52
Average Day	18.6	#3, #4	#1				663 - 671	
Maximum Day	23.4	#2, 3, 4						

6.2.5.2 Residual Chlorine Standard

The residual chlorine standard is listed in Illinois Administrative Code Title 35, Subtitle F, Chapter 1, Part 604 Section 604.725. The current standard is that "a minimum free chlorine residual of 0.5 mg/L or a minimum combined chlorine residual of 1.0 mg/L must be maintained in all active parts of the distribution system at all times."

Using the first-order decay reaction equation ($C_t = C_0 e^{-kt}$), it is calculated that if initial concentration is 1.52 mg/L, chlorine residual concentration would begin to drop below 0.5 mg/L for water staying in the system longer than 1.7 or 3.3 days (for decay coefficient of -0.648 and -0.336 per day, respectively). If the initial concentration is 1.8 mg/L, it would be 2.0 or 3.8 days.

6.2.5.3 Simulation Results

Figure 6-22 and **Figure 6-23** are the chlorine residual exceedance curves for the simulated scenarios. In minimum day demand, which is the worst-case scenario, chlorine residual concentrations would be above 0.5 mg/L for 79% of water demand model junctions and 92% of total system demand. On an average day, that would be 85% of water demand locations and 95% of total system demand. **Figure 6-24** through **Figure 6-26** highlights locations with concentration less than 0.5 mg/L for the simulated scenarios. Similar to the water age analysis, low chlorine residual concentrations were found at dead end mains and outskirts of the system. The South Pressure Zone has more areas below the standard. They are located in areas with water age exceeding 2 days.

It is noted that the Water Utility has already made operational adjustments to increase free chlorine residual in the system and the further improvements were recommended in *EPS Modeling and Chlorine Residual Maintenance Plan* (Strand, 2020).

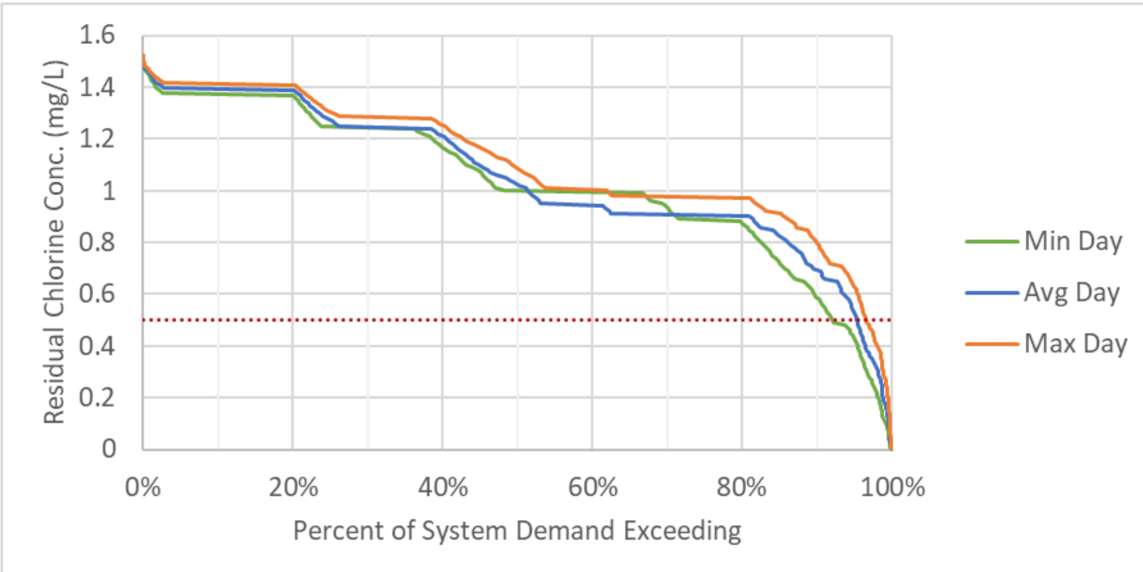

Figure 6-22. Chlorine Residual Concentration Exceedance per System Demand

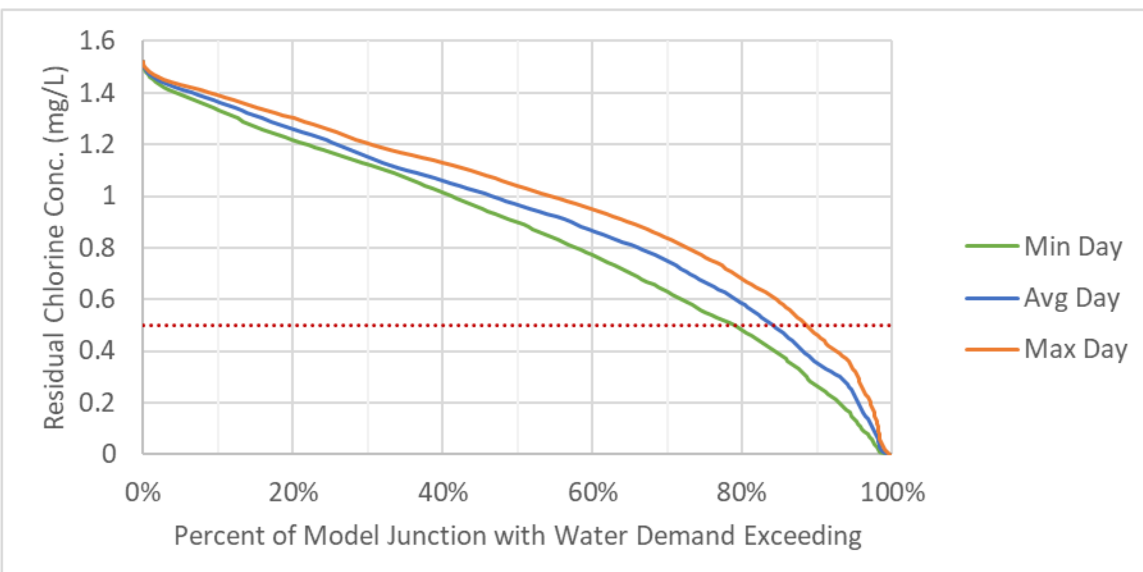

Figure 6-23. Chlorine Residual Concentration Exceedance per Model Junctions with Water Demand

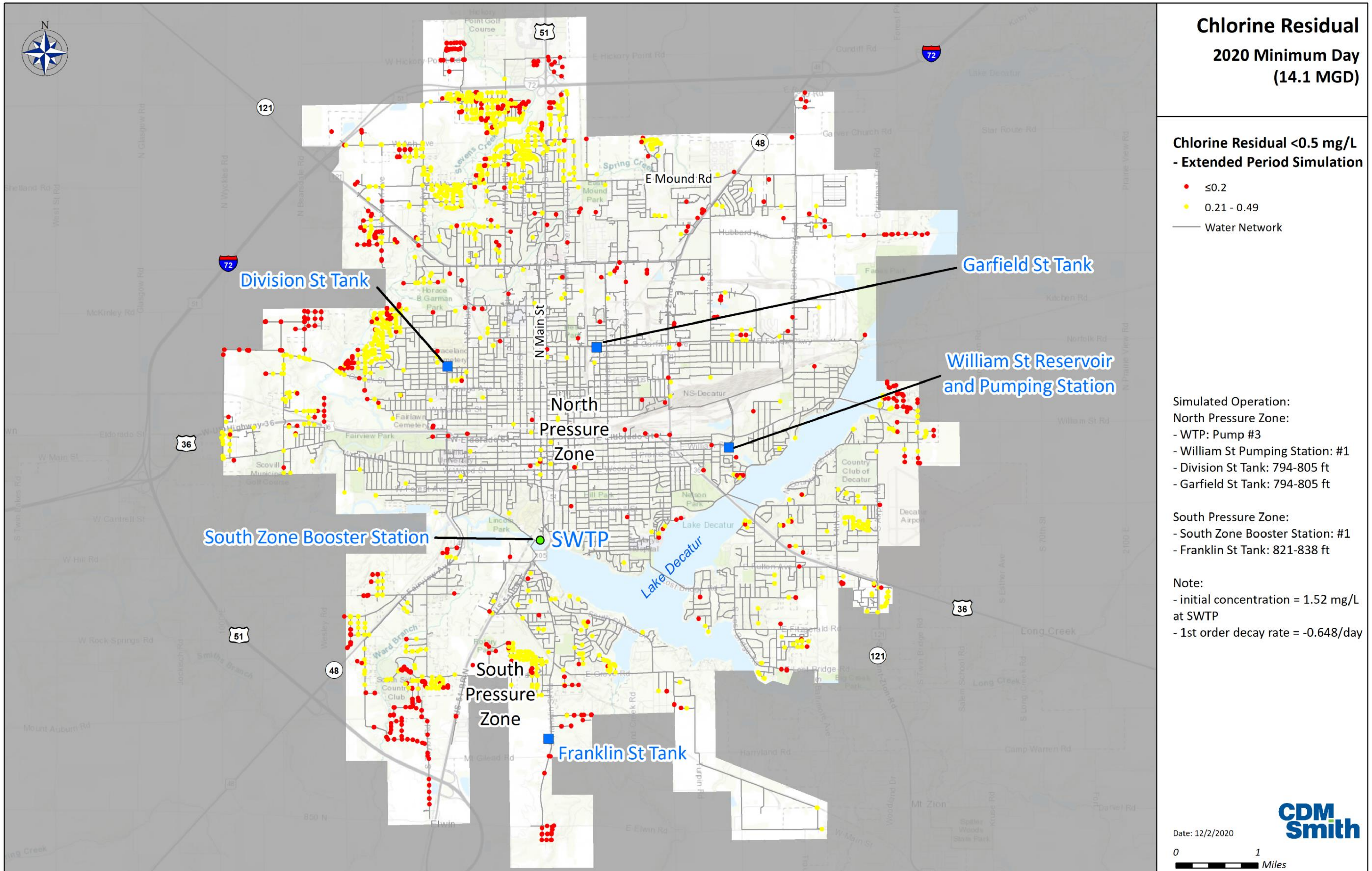

Figure 6-24. Chlorine Residual Concentration with Minimum Day Demand

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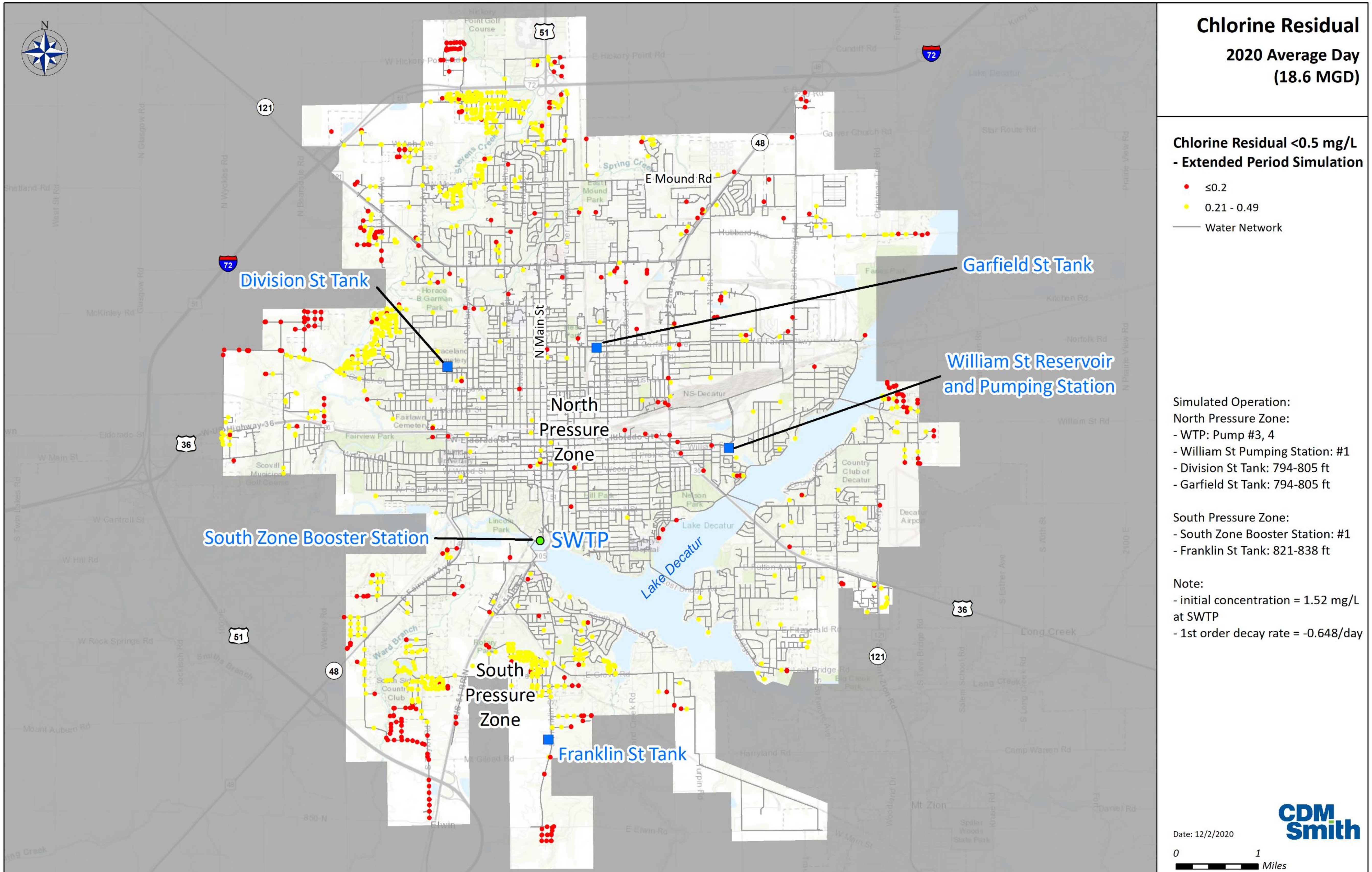

Figure 6-25. Chlorine Residual Concentration with Average Day Demand

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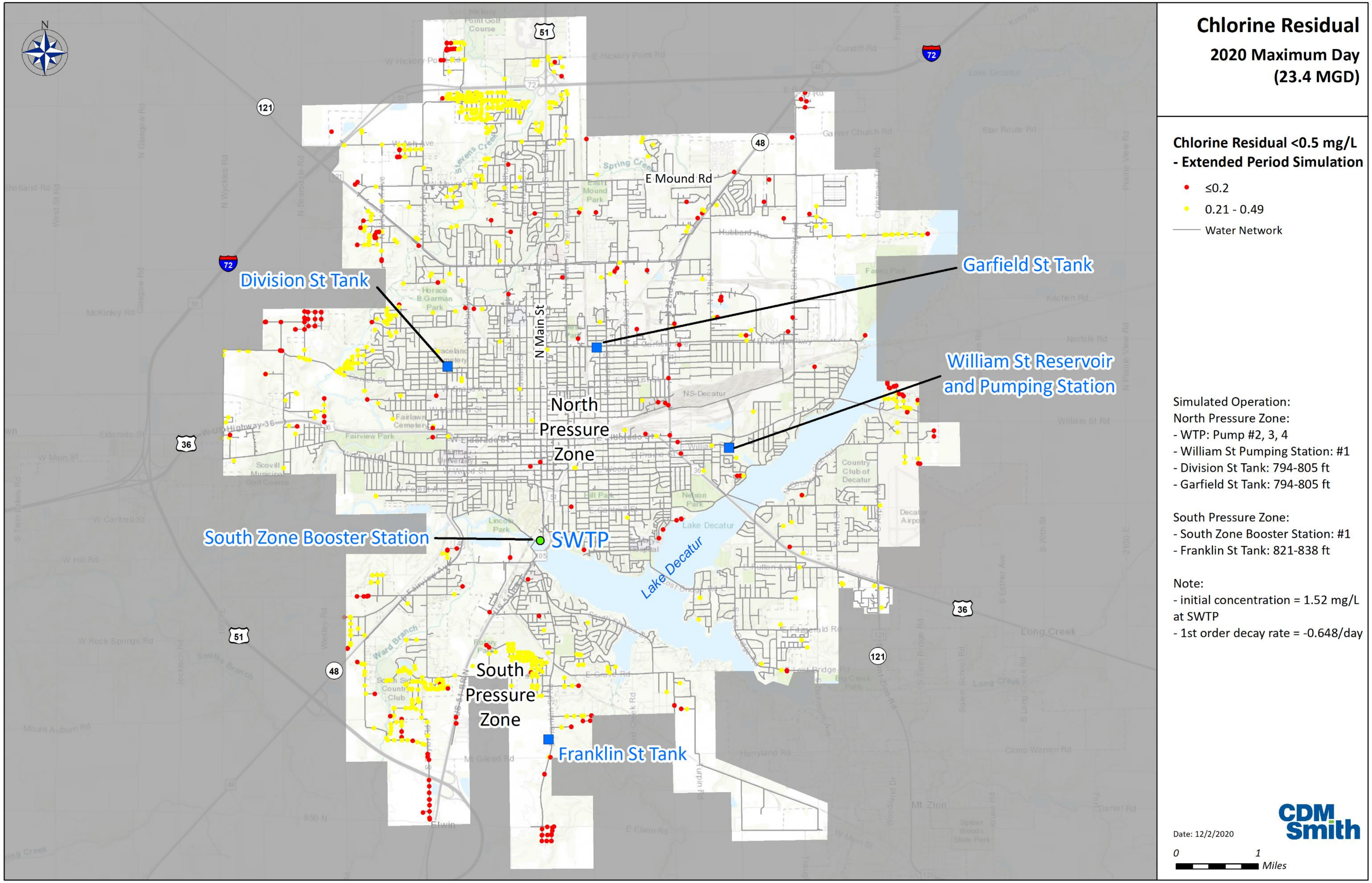

Figure 6-26. Chlorine Residual Concentration with Maximum Day Demand

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6.3 Storage Analysis

The current and future storage need is evaluated and compared against existing available storage.

6.3.1 Existing Storage

Table 6-13 details the Water Utility's existing storage tanks in the distribution system with capacity, construction material and pressure zone.

The Water Utility's water storage is made up of five structures. The Williams Street Reservoir and the SWTP Reservoir are both ground storage reservoirs. Garfield Avenue, Division Street, and Franklin Elevated Storage Tanks are elevated tanks intended to maintain pressure in the system and provide backup storage. These structures form a combined maximum total storage of 16.0 MG. However, some storage volume is not available. The *High Service Pump Station and Water Treatment Plant Storage Study* (Strand, 2008) stated that the SWTP reservoir needs to maintain a minimum level of 13.8 feet (or 3.25 MG) for sufficient chlorine contact time during maximum demand day. 2.67 MG at the William Street reservoir is also not available because the booster pumps cannot operate properly below water depth of 16 feet. As a result, the available storage volume is 10.1 MG. In case of prolonged power outage and exhausted standby power, only the 3.5 MG of storage in the elevated tanks are available.

Table 6-13. Existing Storage Facilities

Tank (Name or Location)	Storage Volume (MG)	Available Storage Volume (MG)	Shape	Material	Pressure Zone
SWTP Reservoir	7.5	4.25	Above ground tank	Prestressed wire-wrapped	North
William Street Reservoir	5.0	2.33	Above ground tank, domed roof	Prestressed Concrete	North
Garfield Avenue Elevated Storage Tank	1.5	1.5	Elevated tank with support columns	Steel	North
Division Street Elevated Storage Tank	1.0	1.0	Elevated tank with support columns	Steel	North
Franklin Street Elevated Storage Tank	1.0	1.0	Elevated tank	Welded steel vessel, concrete pedestal	South
Total	**16.0**	**10.1**	with power		
	3.5	**3.5**	without any power, all from elevated tanks		

According to inspection reports from 2018 by Utility Service Co., Inc., the Division Street elevated tank would require monitoring of spot corrosion and minor rust deficiencies on the interior coating. The Garfield Avenue tank will require an exterior renovation due to the age and condition of the coating. The Garfield Avenue tank will also require monitoring of the access hatch interior, the interior ladder, and the roof of the tank as each spot is beginning to show corrosion. A 2012 inspection report by Utility Service Co., Inc., stated that the Franklin Street elevated tank requires repair of coating on vessel belly, coating on vent sidewall, and coating on wet interior beam surfaces. The 2012 report also recommended that a heavy gauge screen be installed on the interior portion of the roof vent stack.

6.3.2 Storage Needs

Storage volume requirements for the Water Utility are evaluated using two different criteria: Ten State Standards and standard industry practice that has been widely used within the midwestern states of the US.

6.3.2.1 Ten States Standards Storage Recommendations

The Ten States Standards suggests the minimum storage capacity for systems to be equal to the average day demand in addition to any fire flow demands that the system may require. The proposed tank volumes required based on average day demands and ISO recommended fire flows are shown in **Table 6-14** below.

Table 6-14. Required Storage Volume based on Ten State Standards

Year	Average Day Demand (MGD)	Fire Flows (MGD)	Recommended Storage Volume (million gallons)
2020	18.6	0.63	19.23
2030	18.5	0.63	19.13
2040	18.4	0.63	19.03
2050	18.3	0.63	18.93

The Ten States Standards also states that this requirement may be reduced when the source and treatment facilities have sufficient capacity with standby power to supplement peak demands of the system. The high service pump station's firm capacity with standby power is greater than both maximum day demand and peak hour demand, both of which are greater than average day demand and fire flows combined. The treatment plant and distribution system pump stations all also have backup power on site. The system's capacity is sufficient for supplementing any peak demands due to fire flow and average demand.

6.3.2.2 Industry Standard Storage Recommendations

Based on industry standard practice[3], water is stored in tanks for three purposes: equalization, firefighting and emergency storage. Each of these elements were evaluated separately.

6.3.2.2.1 Equalization Storage

Equalization storage is water that is used to supplement pumping operations during periods when demands are higher than pumping rates. The amount of equalization storage required is unique to every water system. For systems such as Decatur, it is common for the pumped supply into the system to be designed to meet the peak day demand. Equalization storage would then provide additional water to meet usage in excess of the peak day demand such as the peak hour demand. The larger the system, the more difficult and expensive it becomes to provide enough

[3] Water Distribution System Handbook by Larry W. Mays Published in 2000; Water and Wastewater Engineering Vol. 1 by Gordon Fair, John Geyer, and Daniel Okun, Published in 1966.

elevated storage to meet the balancing storage requirements. Thus, in large systems, pump stations are designed to supply water in excess of the peak day demand.

Equalization storage is commonly estimated to be 25 percent of the peak day demand. Utilizing the peak day demands, the equalization storage required was calculated and shown in **Table 6-15** below.

Table 6-15. Equalization Storage Required

Demand	Peak Day Demand (MGD)	0.25 x Peak Daily Demand (million gallons)
2020	23.4	5.85
2030	23.3	5.83
2040	23.1	5.78
2050	23.0	5.75

6.3.2.2.2 Fire Flow Storage

Fire-fighting storage is water stored to provide a specific fire flow for a specified duration. Specific fire flow and specific time durations vary significantly by community and regulating authority. Fire flow requirements are dependent on the type of customer (residential, commercial or industrial) and the building construction.

Fire flow requirements are typically based on Needed Fire Flows guidelines established by the Insurance Services Office (ISO), a nonprofit association of insurers that evaluate relative insurance risks in communities. According to ISO, a water system is not required to supply any fire flows above 3,500 gpm. If the Needed Fire Flow for a particular customer is in excess of 3,500 gpm, the customer must provide additional systems (such as sprinkler systems) to lower the needed fire flow requirement or provide on-site facilities to supply the additional fire flow. The Water Utility has earned an ISO Public Protection Classification (PPC) of Class 2. For maximum PPC credit, the Water Utility should have the ability to provide Needed Fire flows of 2,500 gpm for two hours and 3,000-3,500 gpm for three hours. Based on the largest value for flow requirement, the required fire flow storage is assumed as follows:

Fire Fighting Storage = 3,500 gpm x 3 hours x 60 min/hr
= 630,000 gallons

On the other hand, 2008 and 2015 Water Master Plan have quoted fire flow requirements published by the National Board of Fire Underwriters (NBFU), a bureau founded by fire insurance underwriters which merged with the American Insurance Association in the 1960s. The required fire flow storage according to NBFU is as follows:

Fire Fighting Storage = 7,000 gpm x 10 hours x 60 min/hr
= 4,200,000 gallons

The time was reduced to 6 hours after discussions between Water Utility staff and the City of Decatur's Fire Chief in 2015. The resulting fire flow storage equation is as follows:

Fire Fighting Storage = 7,000 gpm x 6 hours x 60 min/hr
= 2,520,000 gallons

6.3.2.2.3 Emergency Storage

Emergency storage is water stored for emergency situations such as source of supply failures, major transmission main failures, pump failures, electrical power outages, or natural disasters. The amount of emergency storage included in a particular water system is at an owner's discursion, typically based on an assessment of risk and the desired degree of system dependability. During emergency situations, customers will significantly decrease water usage to the benefit of the water system. On average, a reserve volume of 15% to 25% of the tank volume is an adequate amount. CDM Smith used 20% of the existing active volume (10.1 MG) as emergency storage for this analysis.

6.3.2.3 Storage Analysis Summary

The total storage volume recommended for the conditions evaluated above is summarized in **Table 6-16.**

Table 6-16. Summary of Storage Analysis

	Storage (MG)			
	2020	**2030**	**2040**	**2050**
Criteria 1: Ten State Standards				
(1) Average Day Demand + ISO Fire Flow	19.23	19.13	19.03	18.93
(2) *Current Available Active Storage*	*10.08*	*10.08*	*10.08*	*10.08*
Gap (MG)	**9.15**	**9.05**	**8.95**	**8.85**
Criteria 2: Industry Standard				
Equalization (25% of Peak Day Demand)	5.85	5.83	5.78	5.75
Fire Flow (ISO and NBFU)	0.63 – 4.2	0.63 – 4.2	0.63 – 4.2	0.63 – 4.2
Emergency (20 % of available storage volume)	2.02	2.02	2.02	2.02
(3) Required Total Storage	8.50 – 12.07	8.48 – 12.05	8.43 – 12.00	8.40 – 11.97
(4) *Current Available Active Storage*	*10.08*	*10.08*	*10.08*	*10.08*
Gap (MG)	**0 – 1.99**	**0 – 1.97**	**0 – 1.92**	**0 – 1.89**

According to **Table 6-16,** the current available storage volume is approximately 9 MG less than average day plus fire flow demand based on Criteria 1: Ten States Standards. This storage would only be required if the system did not have sufficient capacity with standby power to supplement peak demands of the system. Decatur's system capacity with backup supplemental power is sufficient for supplementing any peak demands due to fire flow and average demand, therefore additional storage is not recommended based on the Ten States Standards.

Based on Criteria 2: Industry Standards, if the ISO required fire flows are considered, there is no additional storage recommended for the system. If the original NBFU fire flows were considered, up to 2 MG of additional storage is recommended for Decatur's system. Both fire flow calculation methods are viable means of determining storage recommendations, but additional considerations such as stored water age should be considered. Additional storage has the potential to increase detention times, leading to loss of disinfectant residual, microbial growth, formation of disinfectant byproducts, taste and odor problems, and other water quality problems.

Additional storage is not recommended by CDM Smith at this time based on industry standards and implication on water age.

6.4 Pumping Capacity and Operation

Table 6-17 summarizes the firm and total pumping capacities in 2020 average demand condition. They depend on the system curve and therefore different from the rated capacity of individual pumps.

Table 6-17. Pumping/Booster Station Capacities in Average Demand Condition

Location	# of Pumps	Firm Capacity (MGD)	Total Capacity (MGD)
SWTP HSPS	5	45.3 (#1,2,3,4)	49.3
WSPS	3	7.3 (#1,3)	11.8
SZBS	2	1.7	2.5

6.4.1 High Service Pumping Station

The capacity of HSPS depends on system curve that vary by demand conditions. In normal operations, water demands can be easily fulfilled by running one pump at a time and alternating between smaller and larger pumps to drain and refill the elevated tanks during the diurnal cycle. The HSPS could deliver between 14.4 MGD using Pump #3 and 25.4 MGD using Pump #4 in one-pump operation.

There is no immediate need to expand pumping capacities. Both firm and total capacities exceed the sum of peak hour demand (29.3 MGD) and ISO fire flow demand (3,500 gpm, or 5.0 MGD), which is the worst-case scenario, by at least 11 MGD.

The smallest pump, Pump #3, is operating at the upper end of its design curve. It is rated 9 MGD at 195 feet TDH but is pumping at 14.4 MGD in one-pump mode. This flow rate is slightly higher than the projected minimum day demand (14.1 MGD). To keep at least one pump operating at HSPS in minimum day, system operation is limited to using the excess pumping capacity to refill the William Street Reservoir, overflowing the elevated tanks or flushing the hydrants.

6.4.2 William Street Pumping Station

The William Street Pumping Station helps boost pressure and available fire flow in the east side of the North Pressure Zone. As discussed in **Section 6.2.2.2.4**, the pumps help maintain system pressure above 40 psi during peak hour condition.

6.4.3 South Zone Booster Station

The booster station can easily maintain pressure in the South Pressure Zone with one pump during average demand day. Operating both pumps speed up the on/off cycle and increase turnover at the Franklin Street elevated tank. On an average demand condition, the tank would be refilled and drained once every 1.5 days with one pump operating and once per day with two pumps operating. Water age of the tank would be reduced by 8 hours, while water age in the distribution system would be increased by 8 hours on average. Running two pumps at SZBS during consecutive low demand days could pose short-term issue to water age and chlorine residual in the distribution system.

6.5 Summary of Distribution System Evaluation

Table 6-18 summarizes the evaluation of distribution system.

Table 6-18. Summary of Distribution System Evaluations

Metrics	Description
Pressure	▪ Mostly maintained between 35 and 100 psi ▪ Could drop below 35 psi at Manchester Court if Garfield Avenue Tank HGL is less than 798 ft (27 feet of water in tank). Did not receive complaints of low pressure from residents ▪ William Street Pumping Station is critical in maintaining pressure in east side of North Pressure Zone during peak hour
Available Fire Flow at 20 psi	▪ Less than 500 gpm, the ISO minimum, for 44 hydrants. All are served by 6-inch or smaller diameter water mains. It is recommended for the Water Utility to consider a long-term plan to eliminate 4-inch diameter mains by increasing them to a larger diameter. 6-inch mains should also be evaluated for increasing to a larger diameter especially when not looped. In all cases, the evaluation to upsize should consider water age.
Water Age	▪ 95% of system is less than 2.2 days with average demand. ▪ Water age in the South Pressure Zone is 24 hours higher than that in North Pressure Zone. ▪ Consider hydrant flushing as an interim solution, and perform in accordance with Scenario 1 of the Strand report (Strand, 2020).
Residual Chlorine	▪ 79 – 89% of system exceeds 0.5 mg/L between minimum and maximum demand conditions ▪ Improvements are evaluated in *2020 EPS Modeling and Chlorine Residual Maintenance Plan*
Storage Volume	▪ Existing operating storage volume (10.1 MG) is more than the required volume based on industry standard calculation with ISO fire flow (8.5 MG) ▪ Additional storage volume is not recommended due to water age implication
Pump Capacity	▪ Water demand can be easily managed by running one pump at HSPS and one pump at SZBS ▪ Additional pump capacities are not recommended

6.6 System Optimization / Improvement

This section looks at possible improvements to the identified issues.

6.6.1 Maintaining Pressure Above 35 psi

Manchester Court has the lowest pressure in the system. While the Water Utility did not receive any complaints from residents about low water pressure, the model simulates water pressure dropping below 35 psi at Manchester Court during peak hour of the day. Such occurrence could be mitigated by maintaining water level at Garfield Avenue tank above 798 ft during peak hour demand. For reference, the water level at Garfield Avenue tank typically maintained between 794 ft and 805 ft.

6.6.2 Available Fire Flow

This section looks at the possible incremental improvements to bring the available fire flows of all the hydrants above 500 gpm or 1,000 gpm. The maximum day – max pump operation model scenario was utilized for the analysis. The strategies of system improvements were applied as follows:

- Upsize 4-inch or smaller diameter water mains to a minimum of 6-inch diameter main
- Extend dead end water mains to form a loop; a 6-inch diameter water main to complete the loop is usually sufficient to increase available fire flow above 1,000 gpm; new easement will be needed for new loops running along parcel boundaries; looping will also improve water age in local system
- If looping is not feasible, replace the entire section of dead-end water main with a 6- or 8-inch diameter

Table 6-19 sums up the length of water main needed to be installed or replaced to meet the two thresholds. Roughly 5.6 miles, equivalent to 1% of system length, of new water mains would be needed to raise available fire flows of all hydrants above 500 gpm. To elevate above 1,000 gpm, an additional 41 miles (7.6% of system) of new water mains would need to be installed. **Figure 6-27** and **Figure 6-28** show the locations with water main replacement.

It is recommended to raise available fire flows for all hydrants to be above 500 gpm. Before design, fire flow tests at these locations are needed for further validation.

Table 6-19. Water Main Replacement for Fire Flow Improvement

Improvement Scenario	Length of New/Replaced Water Main (miles)				
	6-in	8-in	10-in	12-in	Total
> 500 gpm	3.7	1.7	0.2	0	5.6
> 1,000 gpm*	35.2	4.7	0	0.6	40.5

* in addition to new/replaced water mains for the >500 gpm scenario

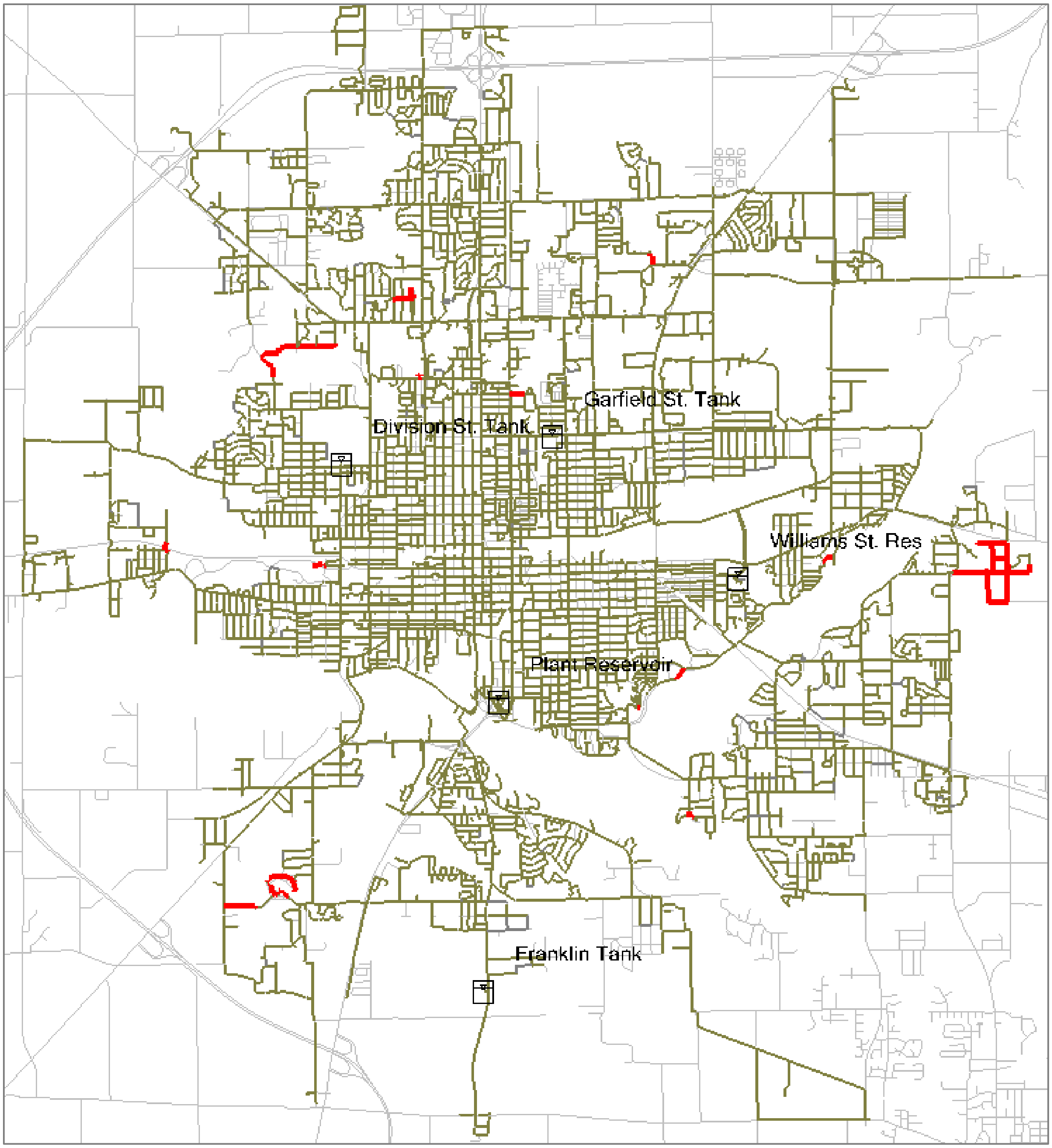

Figure 6-27. Possible Water Main Improvement to Provide A Minimum Available Fire Flow of 500 gpm for Entire System (Highlighted Red)

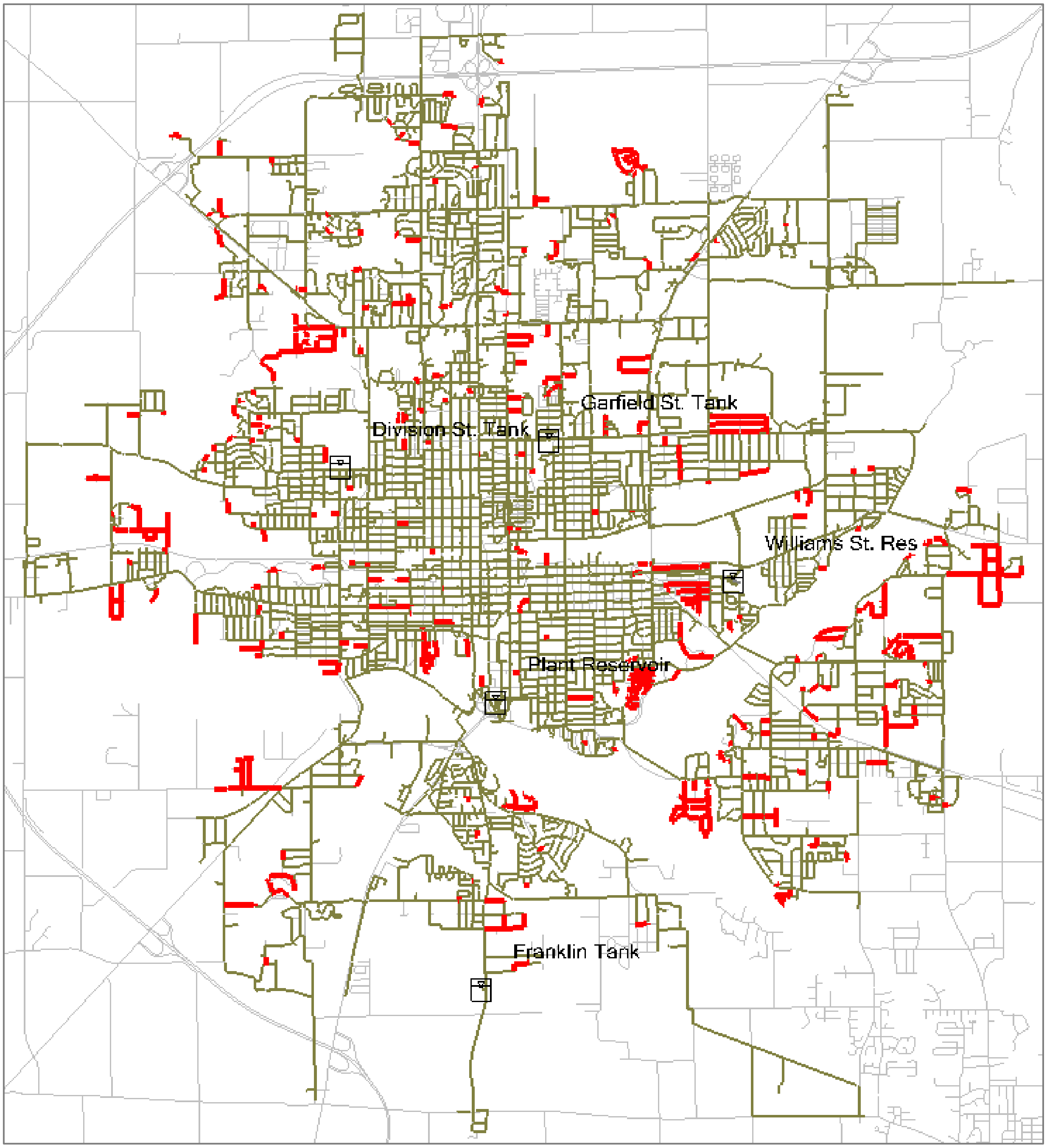

Figure 6-28. Possible Water Main Improvement to Provide A Minimum Available Fire Flow of 1,000 gpm for Entire System (Highlighted Red)

6.6.3 Additional Distribution System Improvements for Water Age and Chlorine Residual

The *2020 EPS Modeling and Chlorine Residual Maintenance Plan* (Strand, 2020) has recommended operational improvements to meet the IEPA free chlorine residual requirement of at least 0.5 mg/L in the system. That includes plans to flush hydrants throughout the system and increase the chlorine dosing at the SWTP.

This section looks at other solutions to reduce water age and increase residual chlorine concentration. That includes installing new water main loops at the edge of the system, which were considered in the 2015 *Water Master Plan Update* (Strand, 2015). Looping does not reduce overall water age. It could reduce water age at the local level by allowing fresher water to pass through the area. **Figure 6-29** highlights the location of new water main loops and the resulting water age in minimum day demand.

In North Pressure Zone, new water mains along Westlawn Avenue between Pershing Road and Ravina Park Road (#1) opens a new route to the northwest corner of the system. This moves fresher water into the area. Water age in the residential neighborhood south of the new water main would drop below 2 days. On the other hand, a new water main connecting the areas along Hickory Point Road (#2 and #3) would reduce water age in the area by at least 2 days.

The water age impact offset each other in the South Pressure Zone. Completing the loop along BR US 51 (#4) reduces water age for the west side but increases water age for the east side of the Wildwood Drive neighborhood. This new route also reduces water movement for the water main along IL 48 to the west, and water age in St Louis Bridge Road and Forest Crest Drive would increase by 1 day. Similar situation occurs at Taylor Road (#5) where water age reduces towards Elwin Road but increases along Heritage Road.

Table 6-20 summarizes the water age improvement for these areas. It is recommended to move forward with proposed improvement in (#1 and #2).

Table 6-20. New Water Main for Water Age Improvement

New Water Main Area #	Neighborhood Area	Water Age (Days) in Minimum Demand Condition	
		Existing	with New Water Main
1a	Summit Ave and Ravina Park Rd	>3	<2
1b	Norwood Ave and Pershing Rd	>2	<2
1c	Westlawn Ave south of IL 121	3 – 7	2 – 4
1d	Elizabeth St and Burt Dr	>7	2 – 4
2a	Hickory Point Rd and Oakland Ave	>5	3 – 5
3a	Hickory Point Rd and US 51	>4	2 – 4
3b	I 72 and IL 48	>7	<5
4a	Wildwood Dr (west)	5 – 6	1 – 3
4b	Wildwood Dr (east) / Franklin St Rd	3 – 6	4 – 7
4c	Grove Rd and Taylor Rd	3 – 6	1 – 5
4d	St Louis Bridge Rd	2 – 6	3 – 7
5a	Heritage Rd	4 – 5	>6
5b	Taylor Rd	>7	5 – 7
6a	Elwin Rd and Franklin St Rd	>7	>7

Area with worse water age in red

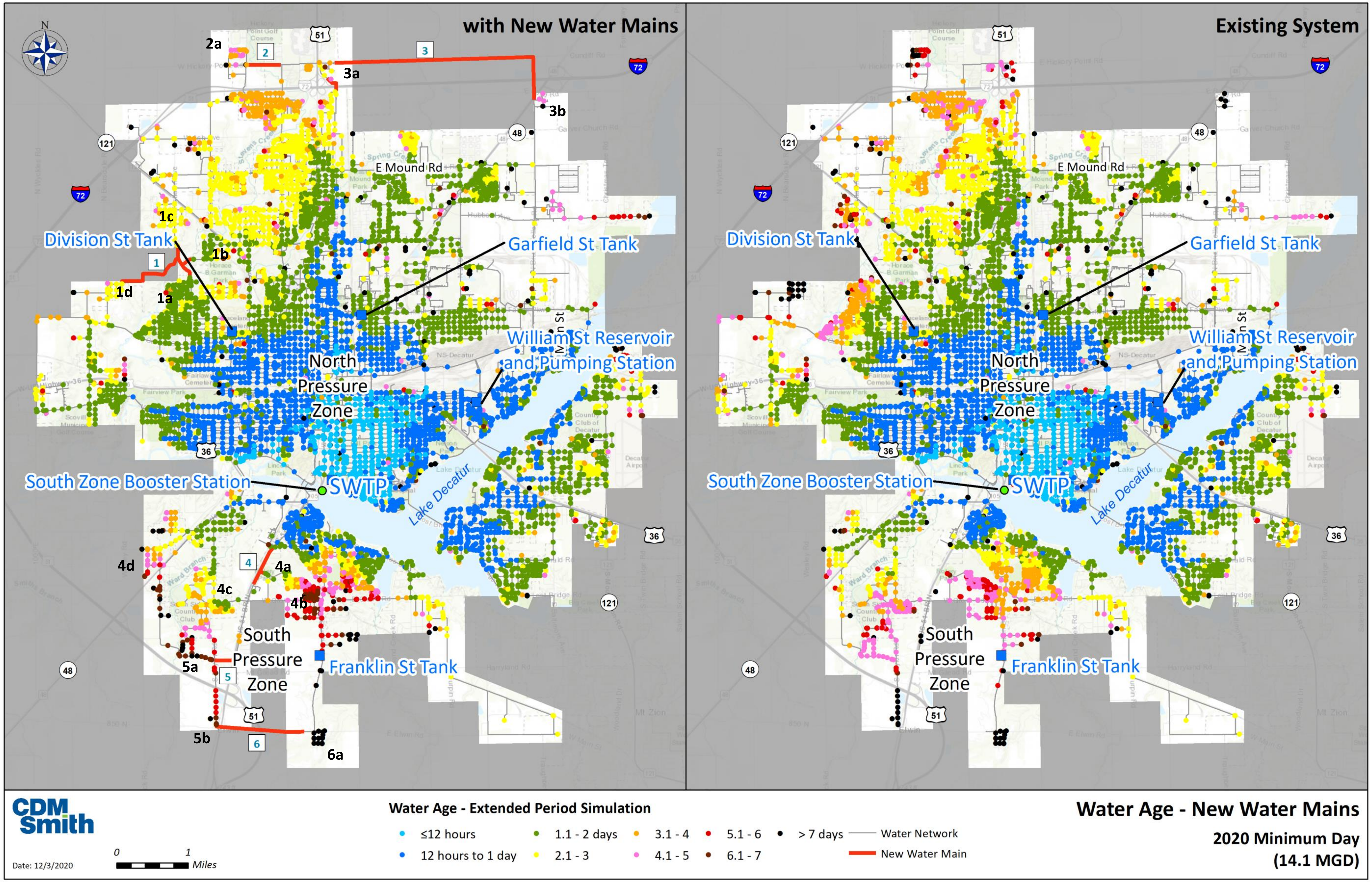

Figure 6-29. Possible Water Main Improvement at Edge of System and Water Age Impact in Minimum Day Demand

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6.6.4 Summary of Recommended Improvements

6.6.4.1 Available Fire Flow

Total of 5.6 miles of new or replaced water mains are recommended to provide a minimum available fire flow of 500 gpm to all hydrants in the system. **Appendix L** contains maps of each project area (Projects FF1 to FF13).

6.6.4.2 Water Age and Chlorine Residual

Up to 1.7 miles of new water main along S Westlawn Avenue between W Pershing Road and N Summit Avenue, and along Trump Hill Lane between S Westlawn Avenue and Burt Drive (#1 in **Figure 6-29**) are recommended to improve water age in the vicinity. These are respectively phases 2 and 3 in Project FF1/WA1, which phase 1 is to provide adequate available fire flow to the Highland Place/Court neighborhood. Refer to **Figure 1** in **Appendix L** for the project map.

In addition, 2,700 ft of new water main along W Hickory Point Road between N Oakland Avenue and N MacArthur Road (#2 in **Figure 6-29**) is incorporated into Project A of high-risk pipes projects. The water main improves water movement, enhances water age and provide redundancy for the neighborhoods north of I-72. Refer to **Figure 2** in **Appendix M** for the project map.

Prior to the installation of improvements, the Water Utility could consider hydrant flushing as an interim measure to reduce water age. Hydrant flushing is referred to as Scenario 1 in the Strand Report (Strand, 2020). This strategy produced preliminary modeling results that flushing hydrants in high water age areas decreases water age, however, there are operation and maintenance costs to the Water Utility to perform hydrant flushing.

6.7 References

Insurance Services Office (ISO), Inc. (February, 2017). *Public Protection Classification (PPC™) Summary Report.*

Strand & Associates (January 2008). *High Service Pump Station and Water Treatment Plant Storage Study.*

Strand & Associates (March, 2015). *Water System Master Plan Update*.

Strand & Associates (September, 2020). *EPS Modeling and Chlorine Residual Maintenance Plan.*

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Section 7

Distribution System Risk Assessment

CDM Smith conducted a risk-based assessment of the Water Utility's water distribution system. This evaluation supported development of a capital strategy to replace or rehabilitate water mains to address condition-based risks and recurring hydraulic issues.

7.1 Methodology

CDM Smith used the Water Utility's geodatabase and staff knowledge to conduct a risk assessment of all pipes in the distribution system. Risk ratings were assigned to approximately 536 miles of distribution pipe (i.e., 29,906 pipe segments in the geodatabase). Risk ratings are assigned using a five-point scale to identify pipes with high risk due to the probability of failure (PoF) and consequence upon failure (CoF).

- **Probability of Failure (PoF) Evaluation** – A probability of failure rating is the rating of a pipe's potential to fail (e.g., leak or break) in the near-term. Probability of failure is related to a pipe's known or estimated condition.

- **Consequence of Failure (CoF) Evaluation** – A consequence of failure rating is an evaluation of a pipe's criticality in the system. If there is a high consequence to the community or environment if a pipe fails, it will be rated as 'highly critical' and receive a high consequence of failure rating.

The process of calculating PoF and CoF scores are detailed in **Sections 7.1.1** through **7.1.5**. Results are detailed in **Section 7.1.6**.

7.1.1 Probability of Failure Evaluation

The PoF evaluation involves scoring each pipe in the system using evaluation criteria, or PoF factors, with the goal of identifying pipes with a higher likelihood of failing relative to other pipes in the system. PoF factors for this study are based upon available GIS data, hydraulic model data and staff knowledge.

PoF factors used for this evaluation are as follows:

- *Material* –Certain pipe materials perform better than others. Water Utility staff identified that ductile iron pipe constructed before 1969 have more risk than cast iron constructed before 1969 or ductile constructed after 1970. Plastic pipe tends to be more resilient than ferrous pipe.

- *Break History* – The Water Utility provided GIS data plotting the location of 3,047 water main leaks occurring between 1974 and 2020. Pipe segments with more breaks received a higher PoF score.

- *Rail Induced Vibration* – Pipes located below railroads received a higher PoF score because the vibrations and other deterioration from the rail traffic raised the risk of damage.
- *Under Water Resource* – Pipes below water bodies received a higher PoF score due to the elevated risks associated with difficult access pipes that are difficult to assess condition.

Table 7-1 presents PoF factors and weighting for the Water Utility distribution system. Refer to **Section 7.1.3** for additional information on how these weights/scores are used to calculate a PoF score for each pipe.

Table 7-1. Probability of Failure Factors

PoF Factor	Weighting	Breakpoint Score					
		5	4	3	2	1	0
Material	10	CIP >1950	NA	DIP, CIP <1950	NA	Copper, HDPE, PVC	NA
Break History	20	10+	NA	5 - 9	NA	1 - 4	NA
Rail Induced Vibration	3	Crosses Rails	NA	NA	NA	NA	Does not Cross Rails
Under Water Resource	5	Yes	NA	NA	NA	NA	No

7.1.2 Consequence of Failure Factors

The CoF evaluation involves assessing each pipe using a series of CoF factors to identify pipes with a severe consequence upon failure. CoF factors are identified based on GIS data and hydraulic model results. CoF factors are selected to be consistent with a triple bottom line (e.g., social / ecological / financial) understanding of system risk.

CoF factors used in this study are as follows:

- *Proximity to Roads* – Failed pipe under or adjacent to roads have a higher consequence of failure because of the potential for public disruption, required multi-agency coordination, and cost for an emergency repair.
- *Proximity to Railroads* – Failed pipe under and adjacent to railroads have a higher CoF because of the potential for public disruption, required multi-agency coordination, and cost to repair.
- *Proximity to Sensitive Receptors (Schools and Hospitals)*– Sensitive receptors are pipes serving people who may be sensitive to water outages. Hospital and school location data was digitized into the GIS system; pipes between the nearest isolation valve and the facility were targeted as a higher CoF.
- *Proximity to High Use Facility* – Facilities that are particularly vulnerable or have high water use were identified by Water Utility staff. These facilities receive a higher CoF score.

- *Difficult Access Upon Failure* – Staff identified pipes that are difficult to access. If it will be difficult and/or costly to access a section of pipe upon failure, the pipe is rated as being more critical to protect.

Table 7-2 presents CoF factors and weighting for the City of Decatur Water Utility distribution system. Refer to **Section 7.1.3** for additional information on how these weights/scores are used to calculate a PoF score for each pipe.

Table 7-2. Consequence of Failure Factors

PoF Factor	Weighting	Breakpoint Score					
		5	4	3	2	1	0
Local Roads	2	0 – 20 ft	NA	NA	NA	NA	>20 ft
Collector Roads	4	0 – 20 ft	NA	NA	NA	NA	>20 ft
Arterial Roads	6	0 – 20 ft	NA	NA	NA	NA	>20 ft
Freeway Roads	8	0 – 20 ft	NA	NA	NA	NA	>20 ft
Interstate	10	0 – 20 ft	NA	NA	NA	NA	>20 ft
Railroads	10	0 – 50 ft	NA	NA	NA	NA	>50 ft
Difficult Access	10	Yes	NA	NA	NA	NA	No
Schools	5	DS of Valve	NA	NA	NA	NA	US of Valve
Hospitals	5	DS of Valve	NA	NA	NA	NA	US of Valve
High Use Facilities	5	DS of Valve	NA	NA	NA	NA	US of Valve

7.1.3 Scoring Approach

PoF and CoF ratings are calculated for each pipe segment as follows:

7.1.3.1 Identify Assets to Evaluate

- *Identify Discrete Pipe Assets* – Pipe assets are identified in Decatur's water system geodatabase.

7.1.3.2 Conduct the Probability of Failure Evaluation

- *Evaluate Each Segment for Each Probability of Failure Factor* – Assign a score to each asset for each PoF factor listed in **Table 7-1**.

- *Calculate a Raw Probability of Failure Score* – Sum the scores from each PoF factor to develop a single raw PoF score for each asset. Refer to the calculation in **Section 7.1.4** for an example of calculating a raw score.

- *Develop a Probability of Failure Rating for Each Asset* – Consider the raw PoF scores for each asset. Break these scores into five scoring ranges of PoF (e.g., raw scores of 0 to 50 receive a rating of 1 – Low Probability of Failure). Assign a PoF rating of 1 to 5 for each asset.

7.1.3.3 Conduct the Consequence of Failure Evaluation

- *Evaluate Each Segment for Each Consequence of Failure Factor* – Assign a score to each asset for each CoF factor listed in **Table 7-2**.

- *Calculate a Raw Consequence of Failure Score* – Sum the scores from each CoF factor to develop a single raw CoF score for each asset. Refer to the calculation in **Section** 7.1.4 for an example of calculating a raw score.

- *Develop a Consequence of Failure Rating for Each Asset* – Consider the raw CoF scores for each asset. Break these scores into five ranges of CoF (e.g., raw scores of greater than 15111 receive a rating of 5 – High Consequence of Failure). Assign a CoF rating of 1 to 5 for each asset.

7.1.3.4 Develop a Risk Matrix

- *Develop a Risk Matrix* – Combine the five-point PoF and CoF scores into a 5x5 risk matrix. **Section 8.1.6** details the risk matrix. If a section of pipe requires work AND its PoF and CoF ratings are high, this pipe would present a large risk to the system and should be addressed in the near-term.

7.1.4 Example Raw Score Calculation

Figure 7-1 shows an example raw CoF score calculation. This calculation process is identical for calculating PoF scores.

Water Pipe X is:

- Located more than 50 ft from a railroad (0 points with a weight of 10)
- Located more than 20 ft from an Interstate Road (0 points with a weight of 10)
- Located more than 20 ft from Arterial Roads (0 points with a weight of 5)
- Located below a Local Road (5 points with a weight of 5)
- Located downstream of the nearest isolation valve serving a medical facility / sensitive receptor (5 points with a weight of 5)
- Has not been identified as having difficult access upon failure (0 points with a weight of 10)

RAW CoF SCORE = (0 x 10) + (0 x 10) + (0 x 5) + (5 x 5) + (5 x 5) + (0 x 10) = 50 points

Figure 7-1. Example Raw Consequence of Failure Score Calculation

7.1.5 Converting Raw Scores to Five-Point Rating

Raw scores were converted into a five-point PoF and CoF rating. Breakpoints for each rating were set to provide an inverse exponential distribution that would identify a small percent of pipes at the highest rating (i.e., PoF or CoF = 5) and more than 50 percent of the pipes at the lowest rating (i.e., PoF or CoF = 1). **Table 7-3 and Table 7-4** summarize raw score breakpoints.

Table 7-3. Probability of Failure Raw Score Breakpoints

PoF Rating	Raw Score
1	< 51
2	51 – 70
3	71 – 105
4	106 – 110
5	> 110

Table 7-4. Consequence of Failure Raw Score Breakpoints

CoF Rating	Raw Score
1	< 11
2	11 – 20
3	21 – 36
4	37 – 50
5	> 51

7.1.6 Risk Matrix

Individual PoF and CoF scores for each pipe were plotted in a 5 x 5 risk matrix. The risk matrix is developed with the following assumptions:

- Pipes with a high PoF and CoF rating (i.e., scores of 4 or 5) are assumed to be high risk. High-risk pipes are identified in the risk profile by the color red. Pipes that have scores that plot in the red box should be considered for a capital project.
- Pipes with a high probability of failure but low consequence of failure have a medium-high overall risk. These pipes are identified in the risk profile by the color pink. These pipes should be addressed with an asset management plan.
- Pipes with a moderate probability of failure and moderate consequence of failure have a moderate overall risk. These pipes are identified in the risk profile by the color orange. Pipes that plot in the orange box should be watched for future damage.
- Lower risk pipes are identified in the risk profile by the colors blue and green. Based on risk, these pipes are a lesser priority for rehabilitation or replacement.

Pipes with the associated risk ratings were plotted in GIS. Individual pipe ratings were adjusted based on common-sense evaluation factors such as:

- If a pipe segment had a lesser risk score but is between two high risk pipes, the intermediate pipe was also rated a high risk. While the intermediate pipe likely did not have damage, it was subjected to similar stressors that damaged the adjacent pipes.

- An adjacent pipe might receive a higher risk score if it is located near a long run of pipes with a high-risk score. The adjacent pipe was added so the recommended capital project would address an entire block (i.e., from cross street to cross street).
- If the pipe had a high risk that was driven by water age or low pressure only, the pipe was reviewed further. Several pipes are extended into private systems; however, the systems are not part of the model. These segments likely do not have pressure or water loss issues.
- Pipes that were either rehabilitated in 2019 or 2020 are assumed to have a risk score of 1.

Figure 7-2 presents the risk matrix distribution. A map of risk ratings for the entire system is provided in **Appendix P**.

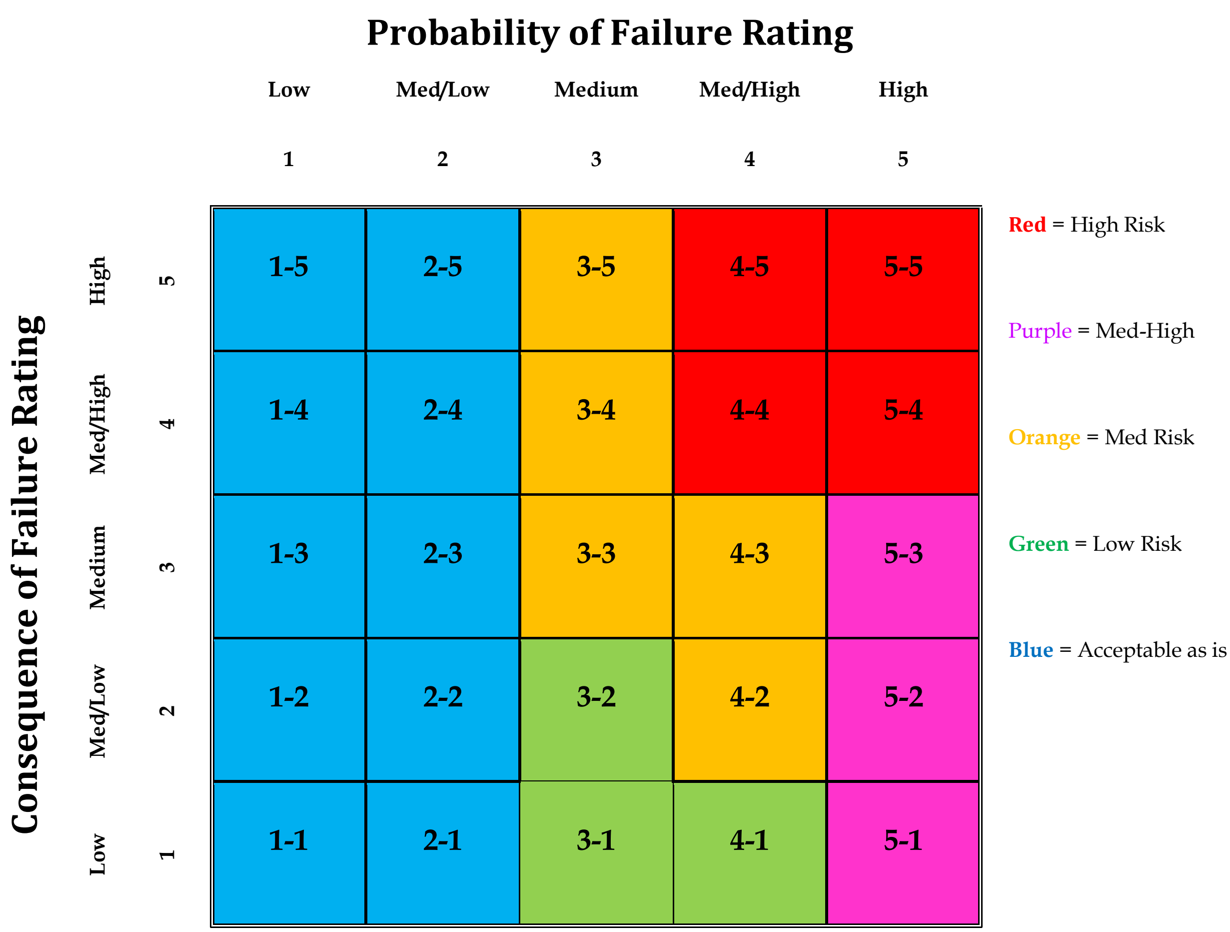

Figure 7-2. Risk Matrix

7.2 Risk Profile

The CDM Smith Team assessed risk for approximately 536 miles of water pipes. Assessed pipes were identified by the Water Utility as 'Active' (LIFECYCLES field in geodatabase) and owned by the City of Decatur Water Utility. **Table 7-5** and **Figure 7-3** present the risk profile for the Water Utility's distribution system.

Table 7-5. Decatur Risk Profile

Risk Level	Miles of Pipe	% of System
5	2.0 mi	0.5%
4	4.0 mi	0.7%
3	10.7 mi	2.0%
2	21.1 mi	3.9%
1	498.2 mi	92.9%

7.3 Capital Projects

Pipe risk, as defined by this study's risk assessment should be a key consideration when developing capital projects. **Table 7-6** provides a summary of distribution system projects that address high-risk projects. Refer to **Appendix B** for maps showing each project, and Section 8 for their incorporation into the Capital Improvements Plan (CIP).

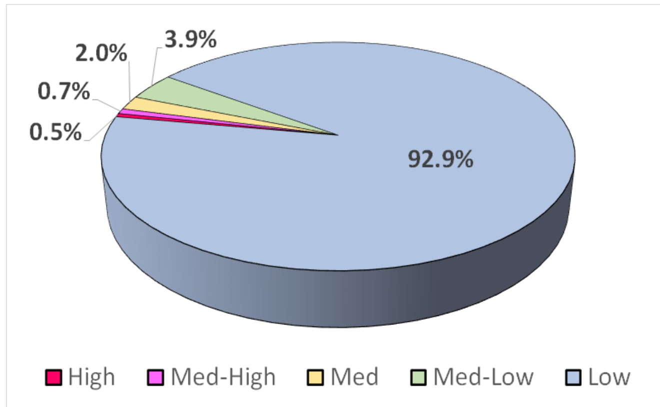

Figure 7-3. City of Decatur Water Utility Distribution System Risk Profile

Table 7-6. Distribution System Projects to Address High-Risk Pipes

Project ID	Street Reference	Pipe Size		Risk Index	Project Type	Present Value Cost		
		Length (ft)	Dia (in)			Construction Cost	Engineering Cost	Total Cost
A	• North Oakland Ave • North MacArthur Road	1,550 ft	10-in and 12-in	5.0	Replace or rehabilitate mains to provide additional redundancy and minimize high consequence of failure	$430,000	$110,000	$540,000
B	• North Moundford Ave • Hummingbird Drive	3,200 ft	6-in	4.0	Replace or rehabilitate mains to minimize high consequence of failure	$900,000	$200,000	$1,100,000
C	• East Garfield Ave • East Locust St • East Logan St • East Olive St • North 22nd St	8,500 ft	6-in and 12-in	4.2	Replace or rehabilitate mains to minimize high consequence of failure and minimize disruptions to traffic	$2,500,000	$600,000	$3,100,000
D	• North Brush College Road	2,600 ft	12-in	4.7	Replace or rehabilitate mains to minimize high consequence of failure and minimize disruptions to traffic	$960,000	$240,000	$1,200,000
E	• East Cantrell St • South 44th St	3,200 ft	6-in	3.5	Replace or rehabilitate mains to minimize high consequence of failure	$900,000	$200,000	$1,100,000
F	• East Cantrell St • South Lake Ridge Ave	8,000 ft	6-in and 12-in	3.3	Replace or rehabilitate mains to minimize high consequence of failure	$2,600,000	$700,000	$3,300,000
G	• West Forest Ave	800 ft	6-in	4.0	Replace or rehabilitate mains due to age and materials of main construction and in order to minimize disruptions	$220,000	$60,000	$280,000
H	• West Legion Drive • West Sesom Drive • South Taylorville Road • Rock Drive	5,100 ft	6-in and 12-in	4.2	Replace or rehabilitate mains to minimize high consequence of failure and minimize disruptions to traffic	$1,500,000	$400,000	$1,900,000
I	• South Franklin Street Road • West Wayside Ave • Medial Drive	2,700 ft	6-in and 12-in	4.2	Replace or rehabilitate mains to minimize high consequence of failure	$900,000	$200,000	$1,100,000

7.4 Risk Mitigation Activities

Capital projects can be refined through additional risk mitigation studies and inspections. Common risk mitigation activities include:

- Assessment of Non-Revenue Water and Leak Detection Studies
- Implementing Smart Technologies to Monitor the System
- Implementing a Valve Exercise Program
- Implementing a Hydrant Exercise and Replacement Program
- Implementing a Meter Assessment and Replacement Program

- Implementing a Lead Service Line Replacement Program

7.4.1 Assessment of Non-Revenue Water and Leak Detection

One common method to focus water main replacement is to first conduct a water audit and then target areas that are expected to have excessive water loss through leaks. A city-wide water audit can be conducted in accordance with the procedures outlined in AWWA M36 – Water Audits and Loss Control Programs. This procedure will quantify water loss due to leakage, meter error and unbilled water consumption. Once water loss due to leakage is quantified, the Water Utility can then conduct targeted leak detection inspections.

7.4.1.1 Leak Detection

In the USEPA report *Water Audits and Water Loss Control for Public Water Systems*, the USEPA recommends implementing a water loss control program through the process of 1) conducting a water audit, 2) initiating steps for an intervention and 3) evaluating the success of the intervention.

Intervention steps include:

- Implementing preventative measures to avoid water loss
- Meter installation, testing and replacement
- Leakage management
- Pipe repair and replacement

7.4.1.2 Leakage Management

A key component of leakage management is to conduct inspections to locate leaks and then repair the defective areas. Leak detection is a rapidly evolving field; technology options and accuracy improve each year. An overview of commonly used technologies in the regional market are summarized as follows.

Acoustic Leak Detection (Exterior Sensors)

Conventional acoustic leak detection involves attaching sensors devices to the exterior of water mains or on hydrants to listen for water leakage in the main. One technology to consider is Echologics, owned by Muller Corporation. Echologics involves attaching sensors to the water main. An acoustic signal is generated in the system; the easiest method to generate an acoustic signal is to open fire hydrants near the testing area. Propagation velocity of the acoustic signal is computed between sensors. This data, coupled with pipe stiffness, will identify an average and minimum wall thickness of the pipe segment. In addition to measuring pipe thickness, any leaks between the two sensors will create an anomaly in the data. These leaks can then be identified and repaired.

Typical distances for evaluation are about 300 to 500 ft between sensors. A limitation to this technology is that pipe wall thickness may vary along the stretch of pipe evaluated. If one keeps the length between sensors short, the results will be more consistent with actual pipe condition.

Acoustic Leak Detection (Interior Sensors)

An acoustic sensor can be inserted in the pipeline and track the location of leaks from the inside of the pipe. This method tends to be more expensive than an exterior acoustic leak detection test; however, the location of leaks can be identified more accurately.

Tethered Leak Detection

One tool for interior leak detection is the Sahara sensor by Pure Technologies. The Sahara unit is inserted through a 2-inch+ tap in the main while it is in service. The sensor includes a small parachute that will catch the flow of water and send the sensor through the pipe. The sensor is tethered so its travel through the pipe is controlled. The position of the sensor is tracked at ground surface. When leaks are detected, this technology can pinpoint the leak within about 18-inches of its actual location. Challenges with this technology are that the sensor cannot maneuver through butterfly valves and the distance investigated is limited by the drag of the cable.

Untethered Leak Detection

Another tool for interior leak detection is SmartBall by Pure Technology. SmartBall is a free swimming (i.e., untethered) acoustic sensor that is inserted into live mains that are 10-in or larger. The sensor is a foam ball that is inserted and extracted through 4-in openings in the pipeline. The sensor is capable of freely traveling for 15 hours on battery power. During this time, it communicates with receivers throughout the system. The SmartBall system will calculate the location of leaks by detecting the location of acoustic pulses relative to the receivers. When the ball is retrieved at a down-gradient location, the information about the location of leaks can be downloaded and processed. The SmartBall system has the advantage of traveling long distances and passing through open valves. Accuracy of results can be influenced by unsteady flow rates that impact the travel speed of the ball through the pipe.

7.4.1.3 Previous Leak Detection Studies in Decatur

The Water Utility has conducted an annual leak detection study on a portion of the system ranging from 153 mi to 176 mi (29% - 33% of the system). Leak testing results were normalized by feet of pipe evaluated. Results are summarized in **Figure 7-4.**

Year	Length Inspected (mi)	Leakage (gpd/mi)
2008	176	4,460
2009	176	1,318
2010	176	1,045
2011	176	2,284
2012	176	2,034
2013	177	491
2014	176	772
2015	182	4,208
2016	153	2,457
2017	165	2,961
2018	165	1,695
2019	165	1,962

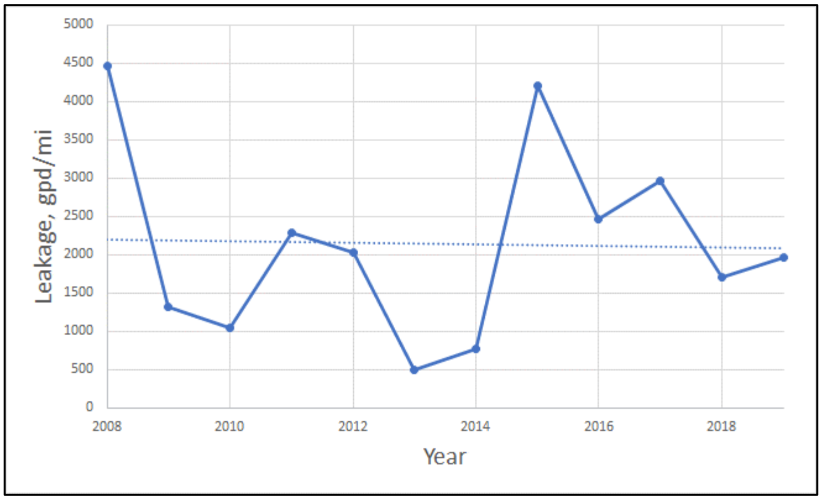

Figure 7-4. Leakage Testing Results

The data set in **Figure 7-4** shows variability per year in leakage rate. Because only half to one-third of the system was evaluated, it is difficult to make any definitive conclusions about the entire system. However, leakage should show a downward trend indicating less leakage as leakage is both lost revenue and lost water. As outlined in AWWA M36, strong water loss control produces the following four benefits:

1. Water resource management: Limiting unnecessary or wasteful source water withdrawals resulting in better use of available water resources.
2. Finance: Optimizing revenue recovery and promoting equity among ratepayers.
3. Operations: Minimize supply efficiency and generating reliable performance data.
4. System Integrity: Reducing potential for contamination in the distribution system.

7.4.2 Smart Technology

An emerging trend in the water distribution industry is to use digital technologies to better monitor and respond to the system. This movement towards enhanced monitoring results in better risk identification and mitigation. With the advancements in remote communication technology and the affordable cost of Cloud-based storage, municipalities are beginning to identify cost-effective opportunities to leverage big data to improve their system. ‘Big Data’ is the industry term to refer to the large volume of data that can be collected, stored, and analyzed relatively quickly using emerging technologies.

7.4.2.1 Computer Maintenance Management Systems

A computerized maintenance management system (CMMS) is a software platform that can help a utility organize and implement maintenance on assets in plants and the distribution system. A CMMS can track maintenance concerns, work history, inventory, and preventative maintenance activities. Base CMMS systems allow for asset tracking; more sophisticated systems will also integrate with ArcGIS to make asset data available in GIS for buried infrastructure.

7.4.2.2 Automated Meter Reading

Most of the water meters throughout the City are connected to the Water Utility's Itron fixed network meter reading system. This type of system involves meters emitting a signal to fixed base networks spread throughout the city limits. The base hubs then transmit results to a master hub where all data is compiled. A fixed-network system can allow for daily water meter readings. Several fixed network technologies also include an application where residents can see their real-time water consumption by checking their property in the network database.

7.4.2.3 In-Line, Real Time Leak Detection

Companies such as Echologics can install acoustic monitoring devices throughout the distribution system. These systems are permanently installed and continually measure acoustic signals in the stem. Acoustic signal results are transmitted to a central data collection hub and are evaluated using artificial intelligence. When leaks are developing, the acoustic signal changes. This change is registered in a monitoring device. The system will identify the change and issue an alert that a new leak is occurring. The Echologics system is called the 'EchoShore DX'. This system monitors leaks on water mains between 4-in to 12-in.

7.4.2.4 Building Information Modeling and Augmented Reality

It is possible to upgrade existing CAD data to convert files to a Building Information Modeling (BIM) system. BIM data is a computer model of the facility with an integrated database of asset information. Database information, such as specifications, manuals and maintenance history can be accessed in the model.

An emerging step for BIM is to integrate 3D models into virtual or augmented reality. While this is not a widespread practice yet, advocates for this technology have suggested that virtual / augmented reality can help with coordinating construction and for staff training.

7.4.2.5 Technology Challenges

When implementing digital technologies in a distribution system, utilities must contend with four key challenges:

- Data Storage – Digital technologies typically generate a large volume of data. If this data is to be archived for further analysis, the utility should work with their information / technology (IT) department to confirm that adequate cloud-based or physical storage is available for the data.

- Communications Infrastructure – In-field technology will need to communicate back to a central data processing hub. For example, remote-read water meters will need to send water meter readings to a collection point. The Water Utility will need to have a reliable communication infrastructure network (e.g., cellular, wi-fi) to facilitate these data collection events.
- Information Technology Department Engagement – Installing additional technology in the distribution system will require additional coordination with the IT Department. Typically, IT Departments have responsibility to the equipment inside of water treatment and conveyance facilities. Digital technologies in the system may require additional IT needs in the distribution system. Consider if your IT department has enough staff and is properly trained to service equipment in the field. If not, it is often possible to contract with the equipment vendor to receive support.
- Cyber Security – Utilities should take precautions to promote Cyber Security of data that is collected and transmitted in the field. Security risks from new technology should be incorporated into future risk assessments of the water utility.

7.4.3 Implementing a Valve Exercise Program

Implementing an annual valve exercising program can help to ensure isolation valves can be located and will be operable if the Water Utility needs to isolate water mains for emergency repair. Developing a strategic program that identifies the valves targeted for exercising each year will help to ensure that the distribution system will be more resilient if a break occurs.

The program should set goals for the following:

- Identify the number of valves to be exercised annually based on the percentage of the total valves in the system. In addition, set a goal that all valves are tested within a certain period. A total of 9,776 valves are in the geodatabase and identified as being 'not private'. The Water Utility should assess staff available to conduct valve testing and establish a reasonable testing interval (e.g., test each valve every 10 years or 978 valves per year). If resources are limited, prioritize exercising the larger valves (e.g., test each valve >12-in every five years or 944 valves / 5 years = 189 valves per year).
- Identify critical valves that should be inspected annually.
- Develop a written inspection procedure and standard form to document results of the exercising activities.
- Identify and replace inoperable valves that are discovered during testing.
- Confirm valves that are exercised are correctly located in the geodatabase and are accessible.

7.4.4 Implementing a Hydrant Exercising and Replacement Program

Implementing a hydrant exercise program will ensure that fire hydrants are accessible and operable if needed for a fire emergency. Develop a program to test and replace hydrants.

The program should include the following:

- Set a goal to test and flush hydrants each year (e.g., test hydrants in the Spring and Fall each year). A total of 4,638 hydrants are in the geodatabase.
- Develop a testing protocol and standard form to document the results of inspection. Include the following in the testing protocol (at a minimum).
  - Confirm the hydrants are correctly located in the geodatabase.
  - Confirm the hydrants are visible and accessible for the fire department. It may be necessary to cut brush around the hydrant or touch-up paint.
  - If hydrants are in poor condition or inoperable, replace the hydrant.

7.4.5 Implementing a Meter Assessment and Replacement Program

A meter assessment and replacement program will ensure that flow meters are accurately accounting for water that is sold to customers. In general, flow meters should be replaced when inaccurate or every 10 to 15 years.

The meter assessment program should have the following components:

- Confirm agreements are in place that allows the Water Utility to test flow meters.
- Test the accuracy of the source water meter at least once per year.
- Test and calibrate water meters on a regular basis. AWWA recommends meters: 5/8-in to 1-in are inspected at least every 10 years, meters 1-in to 4-in are inspected at least every 5 years, and meters 4-in and larger are inspected at least annually.
- Identify and install unmetered connections.
- Replace meters when they are either: inaccurate and cannot be calibrated or when newer, more efficient meters are available.

Section 8 of the CIP includes a meter replacement program.

7.4.6 Implementing a Service Line Replacement Program

As indicated in Section 4 there are proposed revisions to the Lead and Copper Rule LCR. These proposed revisions to the LCR were published in the Federal Registrar, but are currently frozen and in review by EPA. There is also a new Senate Bill proposed that has additional requirements for lead service line replacement, which may have stricter timelines than the Federal Rule. As it is anticipated that the Lead and Copper Rule will take some final form over the coming months, it is important for the Water Utility to determine the inventory of lead and galvanized services and developing a plan for replacement.

The Water Utility has indicated that the water system has no lead service lines but has lead goosenecks with galvanized services. Under the proposed LCR-Revisions, lead goosenecks are required to be replaced when encountered in the field, and galvanized services are also required to be replaced if they are or ever were downstream of a lead service line. While the Water Utility's has reported that galvanized services are not (and were not) downstream of lead services, and thus may not require replacement under the LCR-Revisions, the rule is subject to change under this current review and galvanized services will likely be included in future regulations. Galvanized services can be a source of red water complaints and other water quality issues if left in the system. In addition, the Illinois EPA SRF Loan Program offers up to $4 million dollars of principal forgiveness for the replacement of lead and galvanized services.

As such, the Water Utility should develop a replacement plan that focuses on the following areas:

- Strategy for identifying unknown lines
- Replacing lead goosenecks when encountered in the field during repairs and construction activities.
- Customer notification strategy
- Proposed replacement goal rate for trigger exceedance
- Flushing procedures
- Prioritization considering disadvantaged consumers and sensitive populations
- Funding sources.

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Section 8

Water Infrastructure Capital Improvements Plan

This section provides an overview of the Water Utility's Capital Improvements Plan (CIP) based on a condition assessment of facilities performed by CDM Smith in year 2020, and CDM Smith's review of reports developed by other organizations as provided in Part 8.1 of this section. The projects included in this Section are further evaluated in Section 9 to plan the overall financial impact to the Water Utility over a 30-year period.

8.1 Overview

The Water Utility commissioned CDM Smith to assist in the development of a Long-Term Sustainability Plan. The plan included the development of a 30-year financial projection model which is presented in Section 9 of this report. The model was utilized to assess the economic and financial impact of alternative CIP projects and schedules. The development of the Long-Term Sustainability Plan included information field work performed by CDM Smith, and research by CDM Smith staff of information from the following sources:

- Interviews with operations and maintenance staff
- CDM Smith condition assessment of facilities throughout the Water Utility by a multi-discipline team of civil, process, mechanical, structural, and instrumentation and control engineers as well as architectural staff.
- AWIA Risk and Resilience Assessment
- Distribution system hydraulic assessment
- Distribution system risk assessment based on condition and criticality
- Water system regulatory review
- Review of Intera Additional Supplies Report
- Review of Concentric SCADA Master Plan
- Review of Strand Modeling Report and recommendations
- Review of Northwater Consulting scope of services proposal for Lake Decatur Watershed Management Plan
- Review of other available reports related to the existing Lake Decatur Dam and evaluation of ClariCones at the South Water Treatment Plant

8.2 Asset Management Systems

The Water Utility takes an active approach to maintaining its equipment resulting in facilities that were well maintained. To maintain equipment the Water Utility uses AllMax Antero software for asset management at the SWTP and pump station facilities. The software allows the Water Production Maintenance Supervisor to schedule maintenance activities, and add notes to support staff performing maintenance operations. The Water Utility currently does not have any means to export data globally from the platform to reevaluate maintenance trends with select pieces of equipment.

However, the age of equipment, material degradation, new technologies, and advances in the potable water treatment industry result in equipment and processes needed to be replaced as outlined in this Section.

8.3 Cost Estimate Development

As shown in **Table 8-1**, the American Association of Construction Estimating (AACE) recommends five "classes" of cost estimates. Estimates for this Sustainability Plan are considered "Class 5 – Study/Planning" level. It is anticipated that these costs will be further refined to a more precise estimate class as the project design advances.

Table 8-1. AACE Cost Estimate Classes

Estimate Class	Level of Completion	Source of Cost Information	Estimated Accuracy of Estimate
Class 5	0% to 2%	Professional Judgement	-50% to +100%
Class 4	1% to 15%	Materials x Factor, Cost Curves, Rough Quantities x Unit Cost	-30% to +50%
Class 3	10% to 40%	Refined Quantities x Unit Cost	-20% to +30%
Class 2	30% to 75%	Detailed Quantity Takeoffs x Unit Cost During Design	-15% to +20%
Class 1	65% to 100%	Detailed Takeoffs x Unit Costs at Final Design	-10% to +15%

8.3.1 Multipliers

Multipliers were applied to each project's estimate as follows:

- Engineering, Legal, and Administration – A multiplier of 20% is applied to the sum of the construction subtotal to CDM Smith generated cost estimates to account for engineering, legal, and administration activities required during design and construction for cost estimates developed by CDM Smith.
- CDM Smith did not add multipliers for engineering. Legal or administration to costs obtained from the other sources listed in Part 8.1.

8.3.2 Cost Escalation Over Time

CDM Smith estimated capital costs included in this section were approximated using 2021 pricing as the base year. The estimated capital cost pricing included in this Section and forecasted to occur in the future and were adjusted beyond the base year in Section 9.

The actual project cost would occur during bidding. Consequently, if there is a long bid period, the project cost at the time of design should be inflated to the future bid date.

8.4 Summary of Capital Projects

Capital Improvement Plan (CIP) projects were separated into three groups; immediate term, short-term, and long-term improvements. The projects' grouping was based on one or more of the following criteria: condition, regulation requirement, safety, security, fire flow requirement, redundancy, automation, monitoring, or another identified need. **Tables 8-2, 8-3,** and **8-4** detail the immediate, short-term, and long-term improvements. The Project Numbers are associated with the Spending Plan in **Appendix O**.

Table 8-2. Immediate Improvements – Begin 2021 to 2024

Project Number	Facility/System/Project	Driver/Need	Approximate Capital Cost
21	Development and Implementation of New Water Sources – Immediate	Water Supply/Feasibility	$700,000 (Intera 2019 Study Inflated to 2021)
23	Lake Decatur Watershed Plan – Phases 2 & 3	Water Supply	$780,000 (Northwater Consulting Budget Proposal)
26	Replace Bulk Water Purchase System	Condition/Revenue	$180,000
50	William Street Pump Station Valve and Meter Improvements	Monitoring	$720,000
28	East Clarifier ClariCone Conversion	Condition/Age/Performance/ Lower Power and Maintenance Costs/Reliability	$14,000,000 (ClariCone cost Inflated to 2021 from CMT Memorandum)
29	South Water Treatment Plant Improvements –Chlorine and Caustic System	Condition/Age/Performance/ Lower Power and Maintenance Costs/Reliability	$828,000
38	SCADA Improvements/Upgrades (Y1&2)	Upgrades/Security/Improve SCADA and Monitoring /Condition	$300,000 (Concentric 2020 SCADA Masterplan)
27	Physical Security at Various Facilities	Security	$190,000
52	Distribution System –Fire Flow Improvements	Meet Fire Flows	$2,850,000
		Total Immediate Improvements	**$20,548,000**

Table 8-3. Short-Term Improvements – Begin 2023 to 2029

Project Number	Facility/System/Project	Driver/Need	Approximate Capital Cost
22	Development and Implementation of New Water Sources - Short Term	Water Supply/Feasibility	$5,500,000 (Intera 2019 Study)
24	Lake Decatur Watershed Plan - Phase 4	Water Supply	$720,000 (Northwater Consulting Budget Proposal)
30	South Water Treatment Plant Improvements – Chemical System Improvements	Condition/Age/Performance/Lower Power and Maintenance Costs/Reliability	$3,540,000
31	South Water Treatment Plant Improvements – Architectural and HVAC Improvements	Condition/Age/Performance/Lower Power and Maintenance Costs/Reliability	$30,000
32	South Water Treatment Plant Improvements – Electrical Improvements	Condition/Age/Performance/Lower Power and Maintenance Costs/Reliability	$16,076,000

Project Number	Facility/System/Project	Driver/Need	Approximate Capital Cost
40	Additional Finished Water Storage	Reliability	$7,000,000
41	Finished Water Reservoir Aeration	Water Quality	$3,790,000
33	South Water Treatment Plant Improvements – Clarifier, Piping, and Valve Improvements	Condition/Age/Performance/Lower Power and Maintenance Costs/Reliability	$6,500,000
39	SCADA Improvements/Upgrades (Y3-8)	Age/Performance/Reliability	$1,280,000 (Concentric 2020 SCADA Masterplan)
42	Lime Lagoon Electrical Improvements	Condition/Age/Lower Power and Maintenance Costs	$110,000
44	Raw Water Pipeline Improvements	Reliability	$1,920,000
45	Raw Water Pump Station Architectural Improvements	Condition/Age/Performance/Safety	$70,000
46	Raw Water Pump Station Electrical Improvements	Age/Performance/Reliability	$2,400,000
48	South Booster Pump Station Improvements	Condition/Reliability	$80,000
49	William Street Pump Station Improvements II	Condition/Age/Performance	$150,000
55	Distribution System – Sampling Stations and Distribution Tank Mixing	Performance	$520,000 (Strand 2020 Study)
53	Distribution System – Fire Flow Improvements	Meet Fire Flows	$2,050,000
		Total Short-Term Improvements	**$51,800,000**

Table 8-4. Long-Term Improvements – Begin 2028 and Later

Project Number	Facility/System/Project	Driver/Need	Approximate Capital Cost
25	Development and Implementation of New Water Sources - Long Term	Water Supply/Feasibility	$19,100,000 (Intera 2019 Study)
34	South Water Treatment Plant Improvements	Condition/Age/Performance	$7,550,000
47	Raw Water Pump Station Improvements	Condition/Maintenance	$3,807,000
35	Miscellaneous Water System Improvements	Security/Condition/Age/Performance	$500,000
51	Distribution System - Condition Risk Pipes	Condition/Age/Performance/Lower Power and Maintenance Costs/Reliability	$15,380,000
56	Distribution System - Miscellaneous Annual Watermain Replacement	Condition/Age/Performance/Lower Power and Maintenance Costs/Reliability	$94,500,000
43	Facilities - Miscellaneous Annual Facilities Improvements	Condition/Age/Performance	$55,500,000
54	Distribution System - Fire Flow Improvements	Meet Fire Flows	$6,970,000

Project Number	Facility/System/Project	Driver/Need	Approximate Capital Cost
36	Ion Exchange Resin Removal and Replacement	Condition/Age/Performance/Lower Maintenance Costs/Reliability	$3,900,000
37	Filter Media Removal and Replacement	Condition/Age/Performance/Lower Maintenance Costs/Reliability	$300,000
57	Sediment Trap Maintenance	Accumulation of Sediment/Reduced Storage Capacity	$24,000,000
58	Lake Decatur Dam Improvements	Condition/Age	$4,800,000
59	Elevated Tank Annual Maintenance Allowance	Condition	$1,500,000
60	Annual Valve Exercising Program	Condition/Age/Maintenance	$1,500,000
		Total Long-Term Improvements	**$239,300,000**

Each improvement project is further detailed in a project form included as **Appendix A** for facilities projects, and **Appendix L** and **Appendix M** for distribution system projects**.** Projects include those developed from the facilities condition assessment, distribution system assessment, and previously developed studies. Figure 8-1 shows the total CIP project costs, separated into Treatment Plant Improvements, Pumping and Storage Facilities Improvements, Source Water Improvements, and Distribution System Improvements.

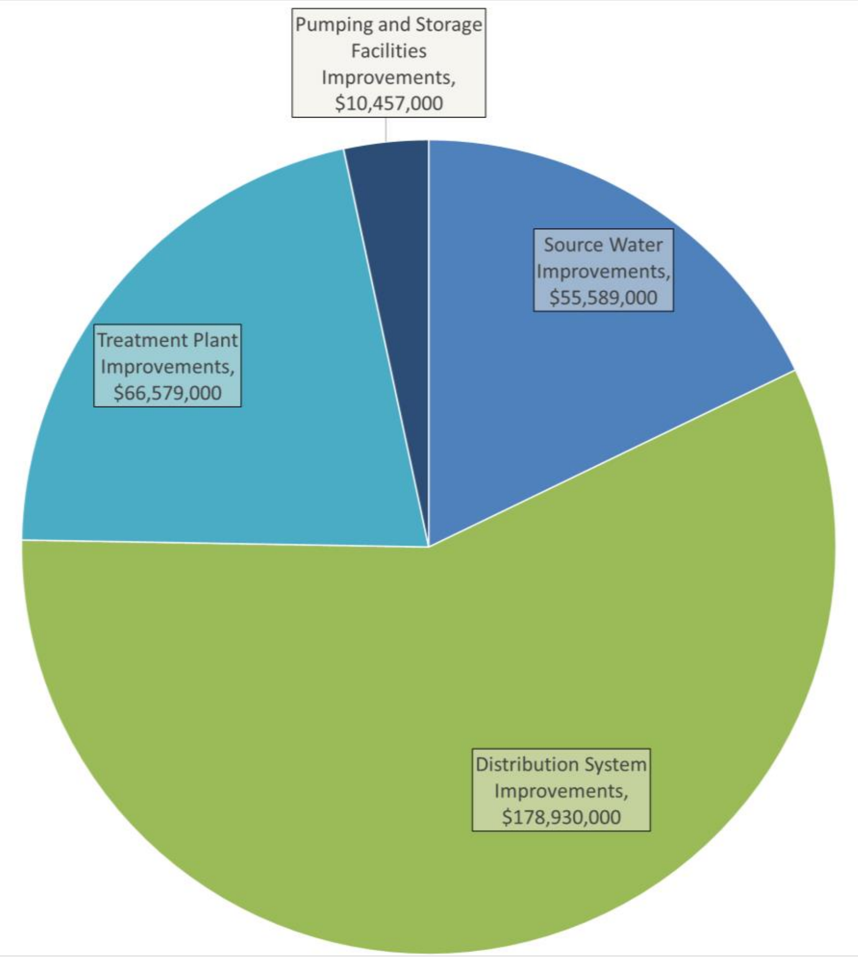

Figure 8-1. Total CIP Costs Separated by Project Type

8.5 Summary of Additional Recommendations

In addition to the CIP projects identified by CDM Smith, the Long-Term Sustainability Plan identified additional recommendations for further studies and considerations. While not currently included as capital projects, the results of these recommendations may lead to future CIP projects, as appropriate. Table 8-5 presents the additional recommendations that were identified for consideration.

Table 8-5. Additional Recommendations for Consideration

Category/Area	Recommendation
Regulatory – Lead	Continue removing lead and galvanized steel service lines in their entirety, and identify remaining lines made of lead or galvanized stee
Regulatory – Chlorine Gas	Continue to monitor for regulatory changes that could impact the use of chlorine gas as a disinfectant and continues to assess non-cost factors associated with chlorine gas use to determine if change to an alternative form of disinfectant is warranted.
Regulatory – Trihalomethanes	Monitor total trihalomethanes closely and consider the final results of the Strand study to further lower DBP levels to help ensure continued DBP regulatory compliance. Consider the draft recommendation of implementing new tank mixers and investigation into other reduction methods. Continue to achieve good TOC removal through the lime softening process and maintain the chlorine application point until after softening.
Regulatory – PFAS	Monitor for PFAS in the raw and finished water, as requested by the IEPA and in anticipation of potential new regulations related to PFAS.
Regulatory – Miscellaneous	Monitor the regulations for changes in the regulatory atmosphere and make adjustments to the CIP to accommodate these changes.
Water Quality – Well Sources	Collect and review water quality information on potential groundwater sources of supplemental water supply to evaluate any water quality impacts of blending.
Operations	Continue to maintain a highly engaged and knowledgeable staff by retaining institutional knowledge, cross training staff, and perform long term workforce planning to avoid potential reductions in drinking water quality by properly training staff.
Lake Decatur Dam	Have an independent third-party testing firm or utilize Hanson Engineering to oversee the testing and operation of both bascule gates to verify the full range of gate operations and that the gates are fully functioning. A report documenting the results of the testing should be prepared by the testing firm to document the gate testing.
Oakley Sediment Basin	Conduct an analysis using data gathered from lab testing of soil samples. Previous Klinger and Associates analysis was based on assumed values, not lab tested samples. Upstream failure surfaces will be critical and make have lower factors of safety as this section of the embankment is built over previous dredged sediments with very low strength.

Section 9

Financial Assessment

This section provides a financial assessment of the projected financial impact of the capital plan projects included in Section 8 on the Water Utility's revenue requirements over a 30-year planning horizon.

9.1 Overview

Section 8 details the capital spending plan (in 2021 dollars) developed as part of the long-term sustainability plan. This section projects the revenue requirements for the Water Utility over a 30-year planning horizon. This analysis includes a discussion of the impact of financing assumptions, and projected rate increases over the planning horizon.

This analysis is intended to provide the Water Utility with a planning level evaluation of the magnitude of the increases in revenue requirements and water rates over time to fund the capital plan. The CIP included in Section 8 includes projects identified at this time. CDM Smith encourages the Water Utility to assess its critical assets on a periodic basis to determine whether new projects are required to be incorporated into the planning horizon. As part of the periodic review, the Water Utility should consider revisiting the financial projections and required rates to recalibrate the projections to reflect current financial conditions and any revisions to the capital plan. Given the length of the planning horizon for this analysis, a number of assumptions are required which may require updating as regulations change, that current processes become obsolete, and the condition of assets degrade.

9.2 Assumptions

The Water Utility revenue requirements are projected for 30 years. The following assumptions were used to develop the projections:

1. The Water Utility's FY 2021 adopted budget was used as the basis for the analysis.
2. The base year for operating and capital expenses is FY 2021. The City's fiscal year starts on January 1st and ends on December 31st.
3. Operations and maintenance costs are assumed to inflate at an annual rate of 3.0 percent.
4. Capital costs are projected to inflate from FY 2021 at an average annual rate of 4.0 percent.
5. Capital costs are assumed to be financed through a combination of cash and general obligation (GO) bonds. GO bonds are assumed to carry a 4.0% interest rate, with a 20-year term.
    a. No future SRF financing is assumed for these projections.
6. The financing method for each capital project is included in **Appendix O.**

7. For the purpose of this analysis, the assumption is that bonds are issued to meet the capital bond funding requirement each year.

8. Anticipated debt service on bond financed capital is assumed to be structured as level principal, with an interest only payment in the year of issuance.

9. Miscellaneous non-rate revenue generally is assumed to remain constant throughout the projection period, at $1.9M per year.

    a. It is assumed that the Water Utility will expend annually an amount of capital for Lake Improvements that requires ADM to reimburse the maximum $1.0M each year.

10. It is assumed that the Water Utility maintains a balance in its water Operating Fund of 1/6 the total annual operating expenses (i.e., 2-month balance). The beginning balance as of the start of FY 2021 is assumed to be $5.1M.

11. The Water Utility's Capital Fund had a balance as of the start of FY 2021 of $2.8M and based on discussions with the Water Utility there are no restrictions on the use of these funds.

12. The Water Utility maintains a Debt Service Reserve, with a current balance of $12.5M. Based on discussions with the Water Utility it is assumed that there are no restrictions on the use of these funds. As a general policy, the Water Utility attempts to maintain a minimum balance of $1M in this fund.

13. The Water Utility intends to pay for certain portions of debt service through available cash, specifically, debt service related to Lake Decatur Dredging projects.

14. No growth in consumption or accounts is assumed over the projection period.

    a. Consumption is based on Water Utility provided data for 2019

    b. Total consumption is 7,914,723 hundred cubic feet (HCF)

    c. Inside City Residential/Commercial consumption is 2,626,311 HCF

    d. Inside City Industrial consumption is 5,099,810 HCF

    e. Outside City Residential/Commercial consumption is 188,602 HCF

15. ADM consumption is assumed to remain constant at 3,562,220 thousand gallons (4,762,328 HCF) per year.

16. Water rate increases are assumed to occur in May of each year.

17. Water rate increases are set to an annual minimum increase of 2.5%.

18. The Water Utility currently charges residents a fixed meter charge by customer type/meter size, and a decreasing block water quantity charge per HCF by customer type. It is assumed that there is no change to this rate structure.

19. Annual rate increases are assumed to be the same percentage increase across all rate blocks.

20. The outside city water rate is assumed to remain 2 times the inside city rate throughout projections.

21. The annual collection rate on billed water rate revenue is assumed to be 94%. It is assumed that the city recovers 90% of the uncollected revenue in the subsequent year.

The following sections summarize the results of the financial projections.

9.3 Revenue Requirement

A 30-year water revenue requirement projection was developed utilizing the assumptions listed above. Rate revenue requirements represent the amount of water revenue that the Water Utility must generate from its water charges to customers to ensure all expenses can be paid from utility generated revenues. The revenue requirement is calculated by adding operations and maintenance expenses, debt service and capital expenditures, less miscellaneous revenue.

This section summarizes the results of the financial projection, including $586.6M in inflated capital spending assumed through FY 2050. A summary of the annual capital spending by financing source included in the analysis is provided in **Figure 9-1**.

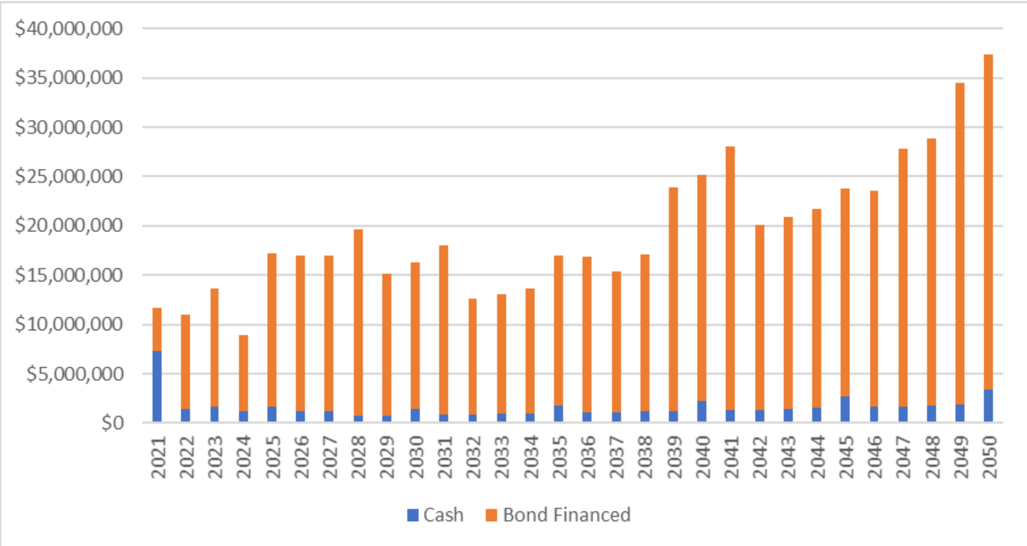

Figure 9-1. Annual Capital Spending (Inflated $)

Over the 30-year forecast period, the analysis includes $49.5M in cash funded capital, and $537.1M in bond financed capital (inflated $).

This section summarizes the projected Water Utility revenue requirement over a 30-year forecast period. Annual revenue requirements consist of operations and maintenance (O&M) expenses, payments of existing debt service, payments of anticipated debt service from bond financing of future capital, cash funded capital, and transfers to reserves. To arrive at the Water Utility rate revenue requirement, miscellaneous non-rate revenue is subtracted from the total.

9.3.1 Operations and Maintenance

Operations and maintenance (O&M) expenses are annual expenses that support the ongoing activities of the Water Utility. O&M expenses have been separated into five departments within the Water Fund, generally consistent with the Water Utility's budget structure, as Lake and Non-Lake as follows:

- Water Lake Services
- Water Production
- Water Administration
- Water Services
- Utility Customer Service

Total O&M expenditures are projected to grow from $15.9 million in FY 2021 to around $37.5 million in FY 2050, summarized in **Table 9-1**. This represents an average annual cost increase of 3.0 percent, reflecting the impact of inflation. For the purposes of this analysis, the budgetary line item "Transfer to Water Capital" has been excluded from the O&M totals below, and instead captured as part of cash funded capital.

Table 9-1. Operations and Maintenance Expenses

Department	2021	2025	2030	2035	2040	2045	2050
Water Production	$5,553,871	$6,250,931	$7,246,542	$8,400,728	$9,738,746	$11,289,876	$13,088,061
Water Lake Services	$1,423,071	$1,601,679	$1,856,785	$2,152,523	$2,495,364	$2,892,810	$3,353,560
Water Administration	$4,271,191	$4,807,263	$5,572,935	$6,460,560	$7,489,559	$8,682,452	$10,065,341
Water Services	$3,576,755	$4,025,669	$4,666,854	$5,410,163	$6,271,862	$7,270,806	$8,428,857
Utility Customer Service	$1,073,140	$1,207,829	$1,400,204	$1,623,221	$1,881,757	$2,181,473	$2,528,925
Total O&M	**$15,898,028**	**$17,893,371**	**$20,743,321**	**$24,047,194**	**$27,877,288**	**$32,317,418**	**$37,464,744**

9.3.2 Debt Service and Capital Expenditures

Capital costs can be financed through bonded debt and paid back as debt service or paid through annual cash payments as cash funded capital. Bonded debt represents bond financed capital projects that are paid for through the issuance of bonds, which is repaid over time as long-term debt service. Cash funded capital represents the annual capital projects that the City would directly fund through current year rate revenue or use of available cash reserves.

Existing debt service represents long-term debt service on water related bond issues that the City has previously issued, as of FY 2021. The expenses are based on the Water Utility's debt service schedules through maturity for each outstanding bond issue.

Anticipated debt service represents projected payments on future bond financed capital projects. As mentioned previously, these results include the impact of $537.1M in inflated bond financed capital spending over the projection period.

Table 9-2 shows the projected capital and debt obligations over the projection period. For the purposes of these projections, the detailed cash flow associated with the capital spending and proceeds from bond issuances are not included. The inherent assumption is that the proceeds from the issuance of future bonds in a given year directly offset the capital costs for the associated projects.

Table 9-2. Debt Service and Capital Expenditures

	2021	2025	2030	2035	2040	2045	2050
Existing Debt Service	$13,150,777	$11,684,756	$8,317,166	$4,681,313	$0	$0	$0
Anticipated Debt Service – GO Bonds	$174,400	$3,539,640	$10,038,985	$14,757,053	$20,450,846	$25,345,316	$29,869,127
Cash Funded Capital	$7,333,331	$1,725,776	$1,447,508	$1,785,358	$2,205,871	$2,719,666	$3,358,788
Total Debt Service and Capital Expenditures	**$20,658,508**	**$16,950,172**	**$19,803,659**	**$21,223,724**	**$22,656,717**	**$28,064,982**	**$33,227,915**

Total debt service and capital expenditures are projected to be $20.7M in FY 2021, increasing to $33.2M in FY 2050. As future anticipated debt grows as a result of future capital spending, debt that is currently being paid back (existing debt service) is decreasing over time as that debt is being paid off.

9.3.3 Miscellaneous Revenue

Miscellaneous or non-rate revenue consists of revenue generated by the Water Utility that is not directly related to water sales. To be conservative and consistent with discussions with the Water Utility, the miscellaneous revenues are held constant throughout the projection period.

Table 9-3. Miscellaneous Revenue

Miscellaneous Revenue	2021	2025	2030	2035	2040	2045	2050
Sanitary District	$300,000	$300,000	$300,000	$300,000	$300,000	$300,000	$300,000
Tapping Fees	$15,000	$15,000	$15,000	$15,000	$15,000	$15,000	$15,000
From Other Funds - UCS Billing	$231,192	$231,192	$231,192	$231,192	$231,192	$231,192	$231,192
Boat Licenses	$138,000	$138,000	$138,000	$138,000	$138,000	$138,000	$138,000
Pier Permits	$87,000	$87,000	$87,000	$87,000	$87,000	$87,000	$87,000
Duck Blind Fees	$1,000	$1,000	$1,000	$1,000	$1,000	$1,000	$1,000
Interest Income	$20,000	$20,000	$20,000	$20,000	$20,000	$20,000	$20,000
Miscellaneous Income	$75,000	$75,000	$75,000	$75,000	$75,000	$75,000	$75,000
ADM Cost Share	$1,000,000	$1,000,000	$1,000,000	$1,000,000	$1,000,000	$1,000,000	$1,000,000
Total Miscellaneous Revenue	**$1,867,192**	**$1,867,192**	**$1,867,192**	**$1,867,192**	**$1,867,192**	**$1,867,192**	**$1,867,192**

9.3.4 Revenue Requirement

The revenue requirement is the total revenue that must be generated annually through water rates to fund the Water Utility's expenses, calculated by subtracting non-rate water revenue from total water expenses. **Table 9-4** shows the total projected revenue requirement, which includes the impacts of the capital plan.

Table 9-4. Projected Revenue Requirement

	2021	2025	2030	2035	2040	2045	2050
Operations and Maintenance	$15,898,028	$17,893,371	$20,743,321	$24,047,194	$27,877,288	$32,317,418	$37,464,744
Debt Service and Capital	$20,658,508	$16,950,172	$19,803,659	$21,223,724	$22,656,717	$28,064,982	$33,227,915
Total Expenses	**$36,556,536**	**$34,843,542**	**$40,546,980**	**$45,270,918**	**$50,534,005**	**$60,382,400**	**$70,692,659**
Transfers for Operating Reserve	$0	$0	$0	$0	$0	$0	$0
Less: Miscellaneous Revenue	($1,867,192)	($1,867,192)	($1,867,192)	($1,867,192)	($1,867,192)	($1,867,192)	($1,867,192)
Revenue Requirement	**$34,689,344**	**$32,976,350**	**$38,679,788**	**$43,403,726**	**$48,666,813**	**$58,515,208**	**$68,825,467**

The average annual increase in the projected revenue requirement is 2.4%, through FY 2050. Given the timing of expenses, there is some variability in the amount of increase each year.

9.3.5 Water Revenue Adequacy – FY 2021

From the Water Utility's FY 2021 adopted budget, total expenses for the operating and capital funds are $47.5M. The Water Utility's budget carries $42.3M in revenue in FY 2021, resulting in a revenue shortfall of $5.2M for FY 2021. The Water Utility budget includes $16.4M in capital spending for FY 2021, with $7.9M in GO bond issuance.

This analysis includes $11.7M in capital spending for FY 2021, with $4.4M in GO bond issuance. Based on the current capital plan and the assumptions with respect to the funding of projects (cash or debt), there would be a projected year end deficit of $4,263,838 for FY 2021.

Table 9-5 summarizes the differences between the Water Utility budget and the projections in this analysis for FY 2021.

Table 9-5. Projected Water Rate Revenue Adequacy – FY 2021; Debt Finance $4.4M in FY 2021

	Recommended Plan – FY 2021	City Budget – FY 2021	Comment
O&M	$15,898,028	$15,898,028	For consistent comparison, carrying the $2,100,000 in transfer to Capital Fund as cash funded capital.
Existing Debt Service	$13,150,777	$13,183,784	Difference may be impact of 2020 refunding. Have not received updated DS schedules since early 2020.
Anticipated Debt Service	$174,400	$0	CDM assumes $4.4M in bond financed capital in FY 2021, interest only payment.
Cash Funded Capital	$7,333,331	$8,462,331	For City Budget, assumed that the entire expense carried in the capital fund is included here, less the $7,900,000 identified for the clarifier bond.
Less: Miscellaneous Revenue	($1,867,192)	($1,867,192)	
Revenue Requirement	**$34,689,344**	**$35,676,951**	

	Recommended Plan – FY 2021	City Budget – FY 2021	Comment
Rate Revenue	$30,425,507	$30,425,400	CDM Calculated, City from FY 2021 Budget
Revenue Requirement	$34,689,344	$35,676,951	
FY 2021 Projected Surplus/(Deficit)	**($4,263,838)**	**($5,251,551)**	

The Water Utility has approximately $20M in total unrestricted reserves, through its Operating Fund, Capital Fund, and Debt Service Reserve Fund. The Water Utility could utilize these reserve balances to address the estimated shortfall in FY 2021.

Another option to addressing the projected FY 2021 shortfall is debt financing a larger portion of the FY 2021 capital spending. If the Water Utility were to debt fund a more significant portion of the capital spending for FY 2021, the Water Utility would be projected to generate sufficient rate revenue to cover expenses. **Table 9-6** summarizes the projections for FY 2021, if the Water Utility were to debt finance $9.7M in FY 2021 (which would include the Annual Water Main Replacement project).

Table 9-6**. Projected Water Rate Revenue Adequacy – FY 2021; Debt Finance $9.6M in FY 2021**

	Recommended Plan – FY 2021
O&M	$15,898,028
Existing Debt Service	$13,150,777
Anticipated Debt Service	$389,480
Cash Funded Capital	$1,956,331
Less: Miscellaneous Revenue	($1,867,192)
Revenue Requirement	**$29,527,424**
Rate Revenue	$30,425,507
Revenue Requirement	$29,527,424
FY 2021 Projected Surplus/(Deficit)	**$898,082**

With the additional debt financing, the Water Utility would be projected to run a FY 2021 surplus of $898,082.

9.4 Projected Rate Increases

The purpose of this section is to summarize the projected water rate increases. The projections assume that the City increases water rates annually by 2.5 percent, as stipulated in the ordinance.

9.4.1 Water Rate Projections

Over the projection period the Water Utility is projected to issue $537.1M in GO bonds, and cash fund $49.5M in capital spending.

Based on the projected expenses, the recommended capital plan, and capital funding assumptions, the Water Utility is projected to experience an operating shortfall of $4,263,838 in the water fund in FY 2021. The Water Utility would need to offset the projected deficit in the water fund through increased water rates (through a mid-year rate increase), reduced spending, transfers from reserves, transfers through the General Fund, or a combination of those methods. As noted, the Water Utility's fund has approximately $20M in total unrestricted reserves, through its Operating Fund, Capital Fund, and Debt Service Reserve Fund. The projections assume the Water Utility utilizes available cash to cover the estimated shortfall in FY 2021.

Figure 9-2 shows a comparison between projected annual revenue requirements, and projected water rate revenue assuming a 2.5 percent annual increase in water rates, as defined in the ordinance. Each year the rate revenue exceeds the rate revenue requirement, an annual surplus is projected. Each year the rate revenue requirement exceeds the rate revenue, a rate increase or use of available reserves would be required.

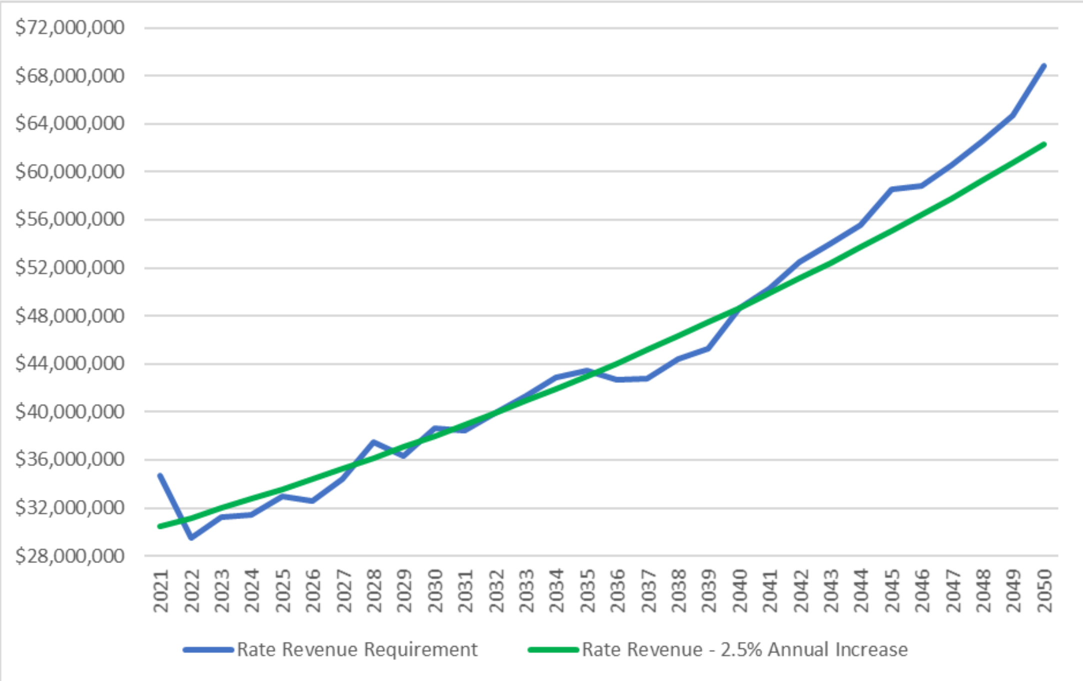

Figure 9-2. Revenue Requirement and Rate Revenue with 2.5% Annual Rate Increase

After FY 2021, the 2.5 percent annual increase established by the ordinance is projected to be sufficient annually until FY 2028. In FY 2028, after the 2.5 percent increase, there is a projected revenue shortfall of approximately $1.3M. To cover the projected deficit in FY 2028 through an additional rate increase, the Water Utility is projected to need a total annual rate increase in FY 2028 of 8.3 percent.

After an 8.3 percent increase in FY 2028, the 2.5 percent annual rate increase is projected to be sufficient until FY 2045. In FY 2045, after the 2.5 percent increase, there is a projected revenue shortfall of approximately $353,000. To cover the projected deficit in FY 2045 through an additional rate increase, the Water Utility is projected to need a total annual rate increase in FY 2028 of 3.5 percent.

After a 3.5 percent increase in FY 2045, the 2.5 percent annual rate increase is projected to be sufficient until FY 2050. In FY 2050, after the 2.5 percent increase, there is a projected revenue shortfall of approximately $2.4M. To cover the projected deficit in FY 2050 through an additional rate increase, the Water Utility is projected to need a total annual rate increase in FY 2050 of 8.3 percent.

As mentioned, the Water Utility has approximately $20M in available cash balances that may be utilized to mitigate projected future rate increases. If the Water Utility were to commit to using the maximum number of available reserves to reduce rate increases, the 2.5 percent annual rate increases are projected to be sufficient into FY 2049, at which point the Water Utility would expend all available reserves. The projected rate increase required in FY 2049 is 2.9 percent, and the projected rate increase required in FY 2050 is 18.8 percent.

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