FINAL REPORT
2020 FACILITIES MASTER PLAN UPDATE

LTU
CITY OF LINCOLN TRANSPORTATION AND UTILITIES

CITY OF LINCOLN, NE

BLACK & VEATCH PROJECT NO. 401472
FINAL REPORT • JULY 2020

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Executive Summary

ES.1 Purpose and Study Area

This report has been prepared to provide the City of Lincoln with an update to the 2014 Facilities Master Plan. Most significantly this report includes a new 12-year capital improvement program, developed based on revised background data for population growth and demand forecast, with consideration for impacts associated with climate change. The recommended improvements plan presented herein will serve as a basis for the planning, design, construction, and financing of facilities to meet the city's anticipated population growth and commercial development through Year 2032. Figure ES-1 presents the study area and anticipated growth tiers for the plan.

Climate change is of ever-increasing concern to the general public given the volatility of recent weather patterns in the State. This facilities master plan update provided an opportunity for the Lincoln Water System (LWS) to consider the impacts of climate change for the first time in their water supply planning process. The specific climate change impacts considered under this study included reduced supply capacity as a result of higher temperatures, reduced stream flows, increased variability in precipitation, and expected increased summer seasonal peak 90-day demand due to longer periods of dry weather.

The principal elements of this master plan study update include evaluation of the following:

- Update Population Projections - Update the population projections to be consistent with the 2040 Comprehensive Plan (*LPlan 2040*) updated and adopted in December 2016. Design Years will include Year 2020, 2025, 2040, and 2060.
- Revise Demand Projections – Evaluate trends in water use and update demand projections taking into account climate change.
- Update Water Supply Projections – Determine 30-day, 60-day, and 90-day water supply yields utilizing existing groundwater model. Utilize basin-wide groundwater modeling tools, with adjustment for climate impacts, to revise streamflow input into the model.
- Evaluate the Water Treatment Plant (WTP) – Review historical records to confirm compliance with regulations. Perform high level condition assessment to determine necessary improvements for ongoing reliable operations. Evaluate timing and need of plant expansion based upon revised demands, condition assessment, and process considerations.
- Distribution System Analysis - Update the computer model of the Lincoln water distribution system in InfoWater hydraulic analysis software and perform analyses for average day, maximum day, and maximum hour scenarios for Years 2020 and 2032.
- Perform Distribution Water Quality Analyses – Evaluate available historical water quality data, perform distribution water age analyses, and develop protocol for system improvements which enhance water quality in the system.
- Update Transmission Condition Assessment – Develop condition assessment program for the transmission system based upon available technology, inspection cost, pipe material, and main criticality.
- Lead Service Line Review – Review existing records to quantify existing lead service lines and provide summary of regulations and replacement strategies.
- Capital Improvement Program – Prepare an update of recommended water system improvements.

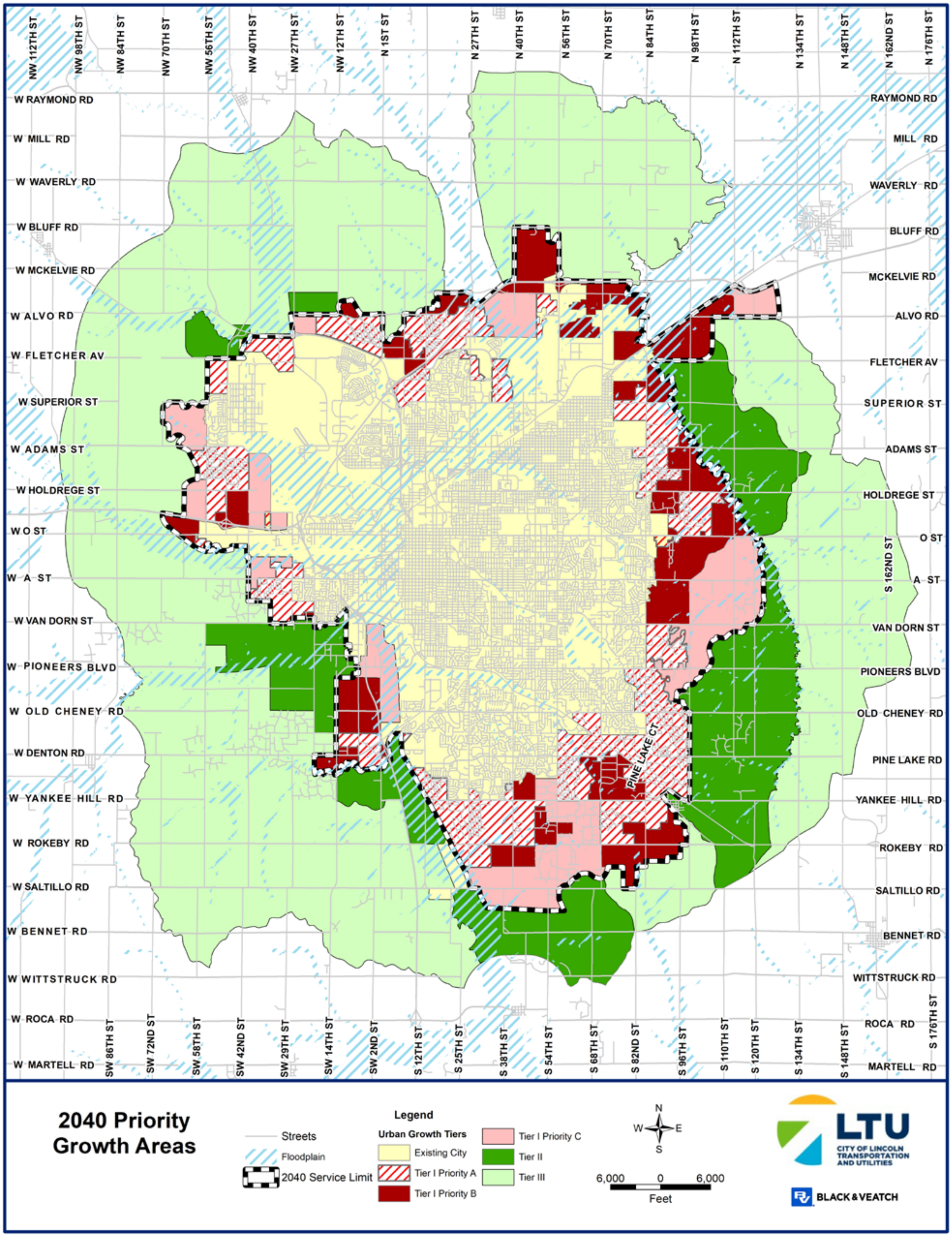

Figure ES-1 Study Area and Priority Growth Areas

ES.2 Population

To accurately predict future water demands, the magnitude, location, and characteristics of future population growth were evaluated. Population projection data for the City of Lincoln was obtained from the current *LPlan 2040*, which delineates the spatial distribution of growth within the growth tiers by Traffic Analysis Zones (TAZ) through Year 2040. Beyond 2040, the *LPlan 2040* uses an extrapolation to develop the 2060 projections. Figure ES-2 presents the historical and projected population through 2060.

The *LPlan 2040* is currently being updated, and the revised version is expected to include a slightly lower rate of population growth. Therefore, the population projections used in this Master Plan update based on the current *LPlan 2040* (adopted in December 2016) may be slightly higher than future projections from the upcoming *LPlan 2040* update.

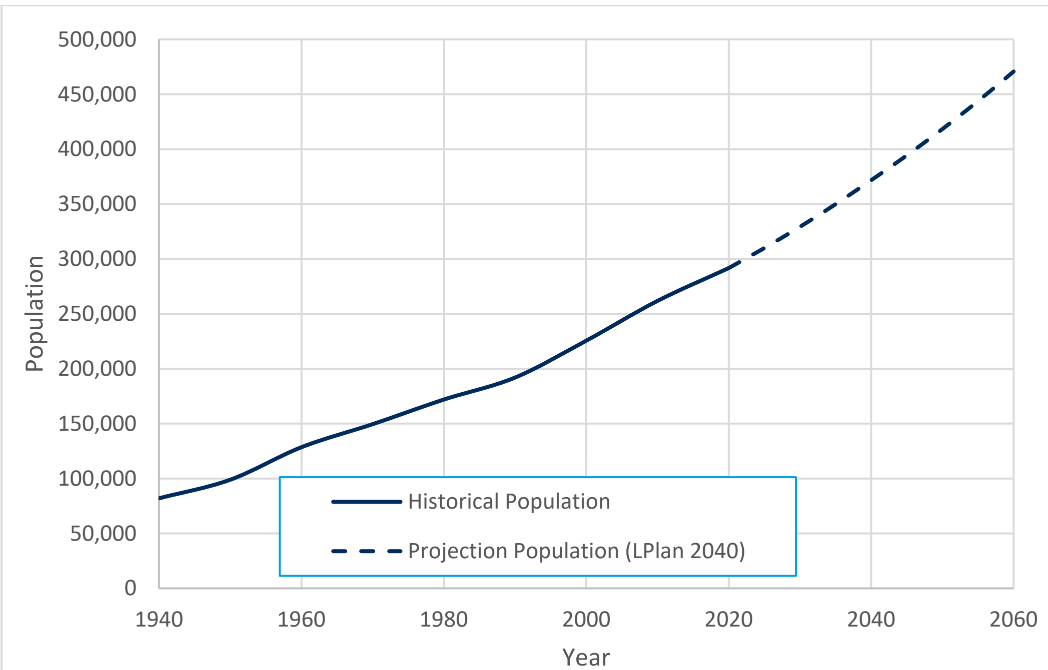

Figure ES-2 City of Lincoln Historical and Projected Population

Census population data was used to update the population by Service Level for Year 2010 and the *LPlan 2040* data was used to develop the population projections by Service Level for the planning period, demonstrated in Figure ES-3.

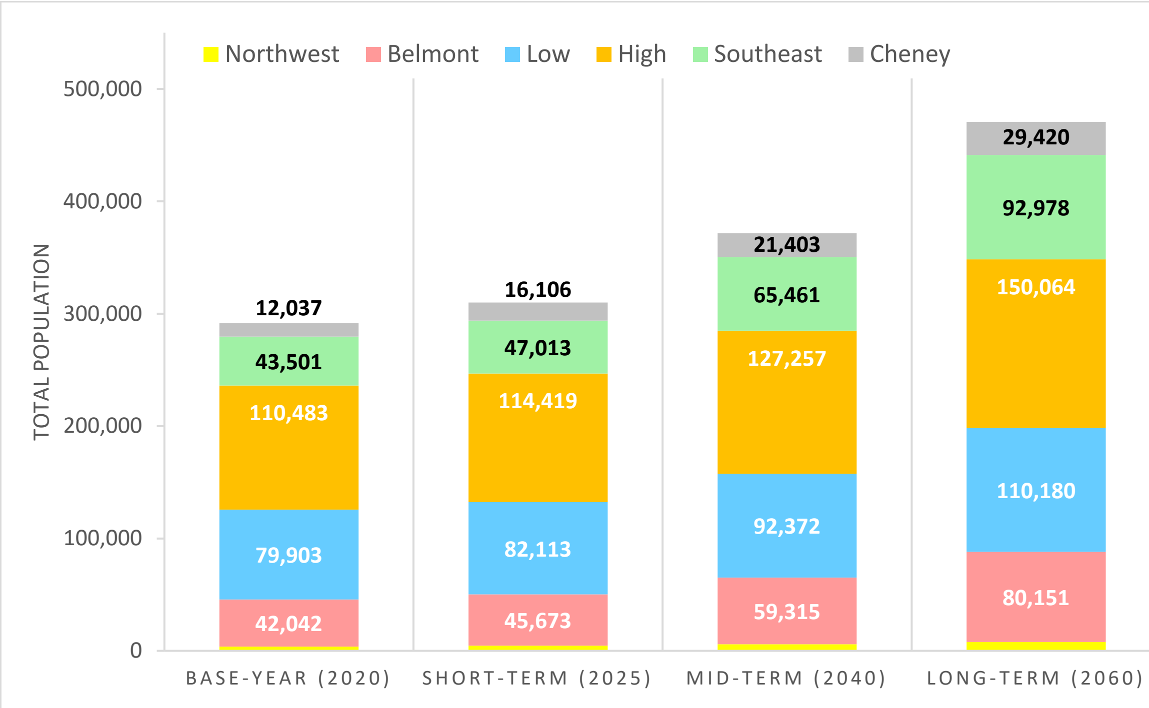

Figure ES-3 Existing and Projected Population by Service Level

ES.3 Water Capacity Requirements

The water capacity assessment focuses on average day demand (AD), seasonal peak demand (SP), maximum day demand (MD), and maximum hour demand (MH), which are typically used for design and operation of WTP and distribution system infrastructure.

- **Average Day (AD)** demand is the total annual water use divided by the number of days in the year.
- **Seasonal Peak (SP)** demand is the average daily use of water over the highest three consecutive months of demand during a given year, generally June through August or July through September.
- **Maximum Day (MD)** demand is the maximum quantity of water used on any one day of the year and is used to size water supply, treatment facilities, and pumping station capacity needs.
- **Maximum Hour (MH)** demand is the peak rate at which water is required during any one hour of the year.

ES.3.1 Historical Water Usage

Historical and current water capacity requirements were updated based on data available from distribution production reports and metered sales reports from 2013 to 2018 and the water treatment plant monthly operating reports from 2013 to 2020. This information was used to characterize historical water usage trends and to establish criteria for future water demand projections, including peaking factors, residential per capita usage, percentage residential usage, and non-revenue water. Figure ES-4 presents a summary of the historical water usage and per capital usage (overall and residential) from Year 2000 to Year 2018.

Historical metered sales data was used to assess the mix of residential and non-residential water use, to determine typical per capita water use rates, and to update non-revenue water characteristics. Analysis of historical meter sales demonstrates that the percentage of residential metered sales has consistently remained around 65 percent of total sales.

Non-revenue water includes water used for flushing, firefighting, water main breaks, leakage and apparent losses (meter inaccuracies). Non-revenue water has varied significantly, ranging from approximately 2 percent (excluding Year 2006) to 15 percent with an average of 9 percent in recent years. The City's non-revenue water was compared with AWWA performance indicators, which characterize non-revenue water as gallons per day of water loss per service connection. The City's 2018 total water loss of 50.5 gal/d per service connection is less than the AWWA median values of 78 gal/d per service connection, meaning that the City is in the lower 50 percent of community water loss based on the audit conducted by AWWA.

A downward trend in per capita usage was observed between Years 2000 and 2018, although the curve appears to start flattening in Year 2014. While the City may continue to see a downward trend in per capita usage, it is anticipated that a limit will be reached over the next decade.

- The following planning criteria were modified based on analysis of historical records:
- Declining trend in residential per capita usage.
- Increased percentage of non-revenue water from 6.7 to 9 percent.
- Reduced MD:AD peaking factor from 2.4 to 2.25.

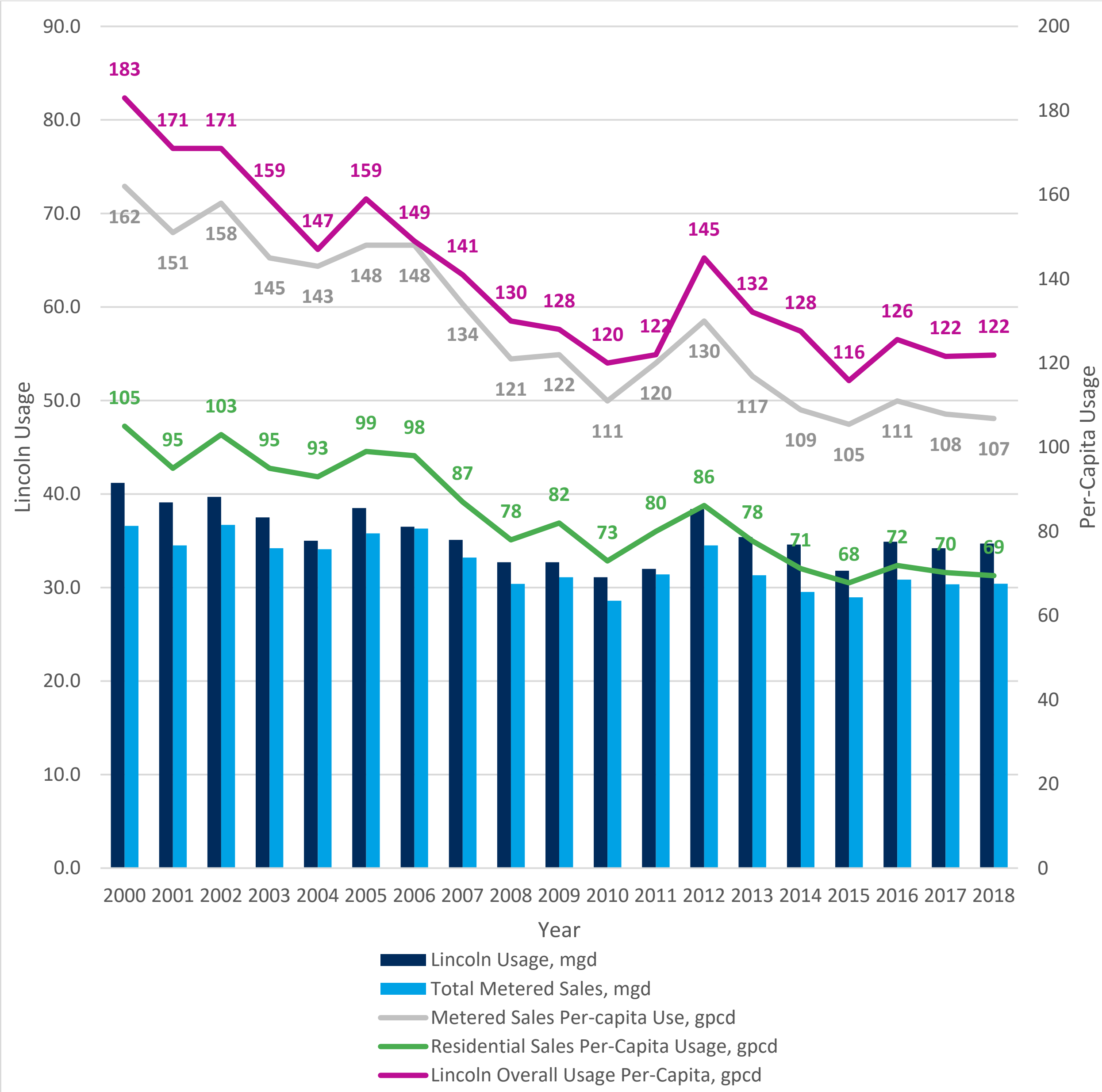

Figure ES-4 Historical Water Use and Per Capita Usage

ES.3.2 Water Demand Projections

Water demands for the planning horizon included Years 2020, 2025, 2040, and 2060 with interpolated demands for interim years (i.e. Year 2032). The water demand projections are based on the population forecasts and historical trends for residential per capita usage, percentage residential usage, non-revenue water, and peaking factors. Figure ES-5 provides a summary of the demand projections based on the planning criteria.

Considerations for climate change were used to evaluate impacts on SP wellfield pumpage. Specifically, water demand projections were adjusted based on the following mid-century (Years 2041 to 2070) climate change projections:

- 4 to 5°F increase in ambient air temperatures year-round.
- 15 to 20 percent increase in precipitation in the winter, spring and fall.
- 15 to 20 percent decrease in precipitation in the summer.

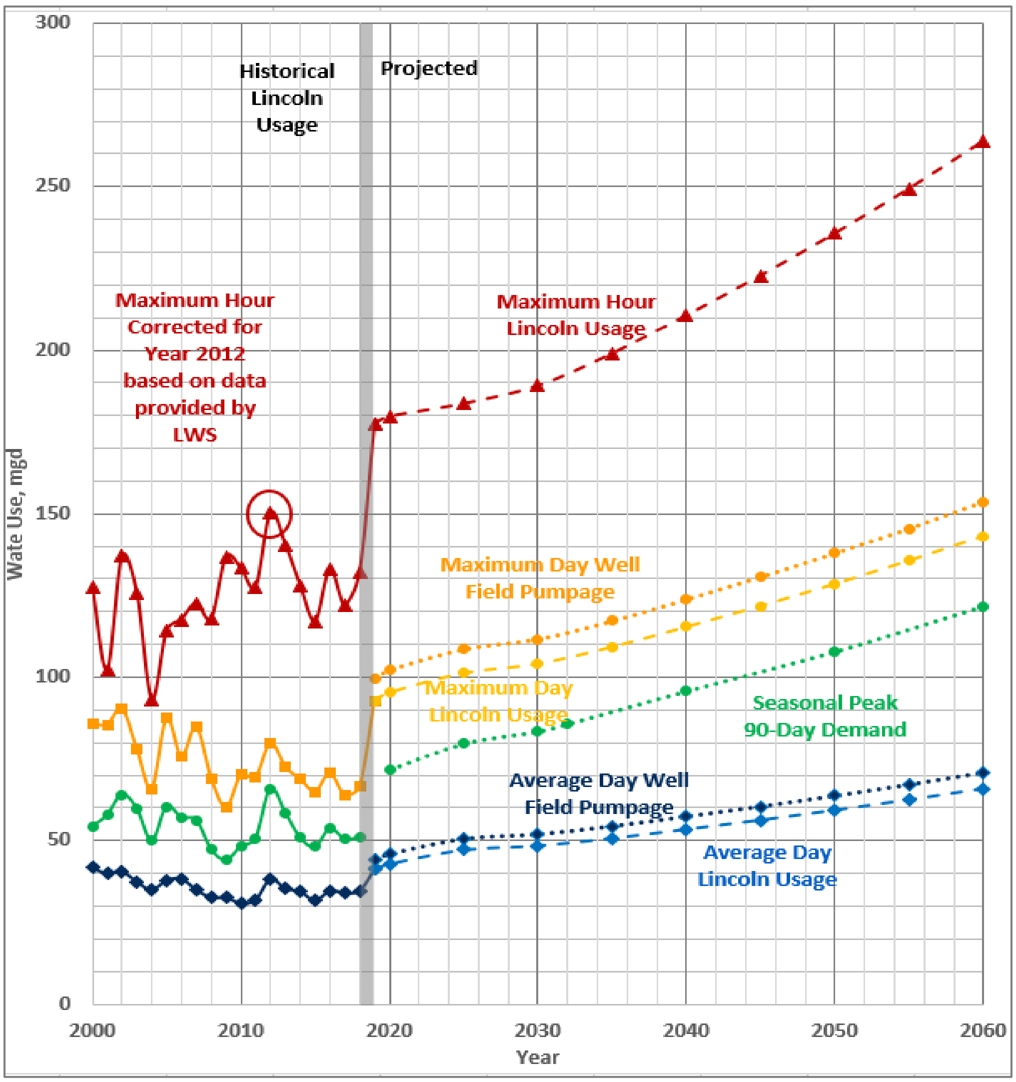

Figure ES-5 Future Demand Projections and Well Field Pumpage Requirements

ES.4 Water Supply

The Lincoln wellfield is heavily dependent on Platte River streamflows that recharge the alluvial aquifer from which water is withdrawn. During periods of normal and high streamflows, the aquifer receives plenty of recharge and the wellfield is easily able to meet demands. However, during periods of lower streamflows, it is possible for withdrawals to begin to exceed the rate at which water is recharged from the stream to the aquifer. The single greatest threat to the wellfield's water supply is extended periods of low river flows, such as those that occurred in early Year 2000 and again in Year 2012. Long-term groundwater flow modeling simulations using regional- scale models were developed to forecast future streamflow conditions and in particular, the impact of streamflows during low-flow conditions.

The groundwater model results demonstrate reductions in streamflow, which primarily occur in the Central Platte River above the confluence with the Loup River. The results of this evaluation for the 90-day low-flow at recurrence intervals between 5 years and 500 years for the 2040 planning horizon are shown in Figure ES-6.

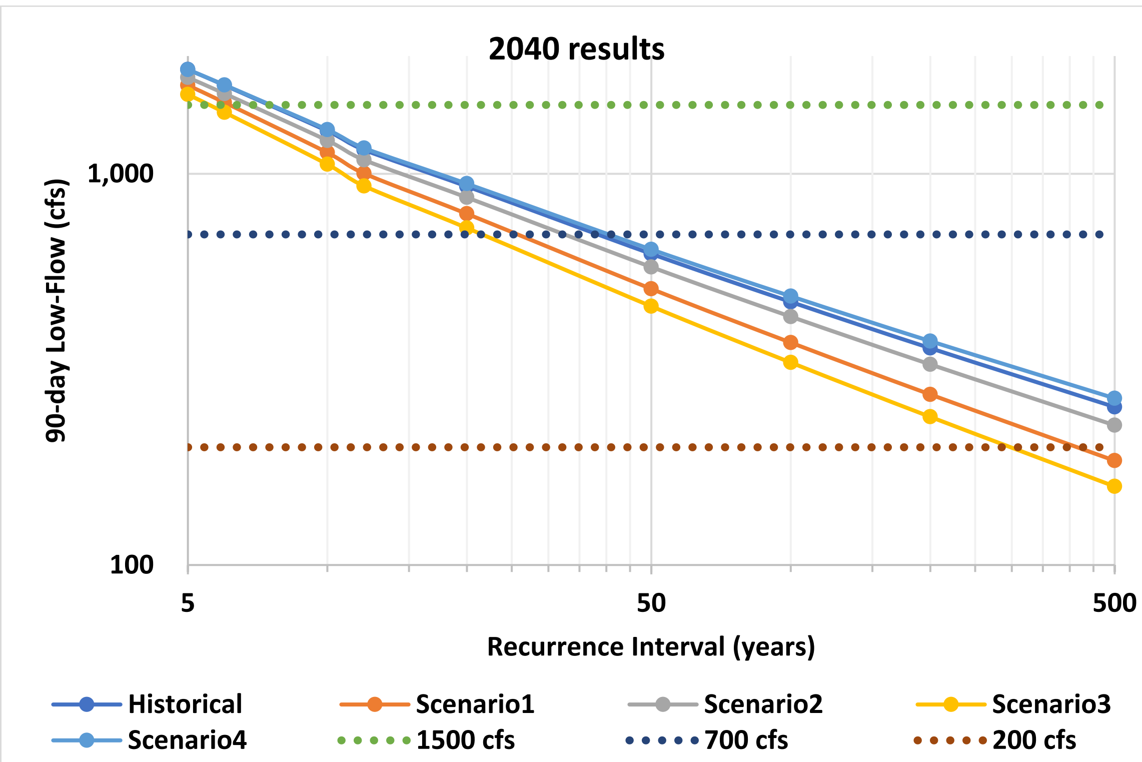

Figure ES-6 90-Day Low-Flow Conditions for the Historical Data and Each Scenario for the 2040 Model Results for Recurrence Intervals Between 5 Years and 500 Years

The benchmark for wellfield expansion is the capability to supply the summer seasonal demands over a 90-day period with the river level at 200 cfs. MODFLOW modeling results determined that the existing system is capable of producing 90 mgd over the 90-day duration. As shown on Figure ES-7, the existing facilities are capable of meeting this hypothetical design condition through Year 2035. Installation of an additional horizontal collector well (HCW-5) by Year 2035 would be considered a "just in time" improvement. It is therefore recommended that the City consider advancing this improvement a few years in the capital improvement plan to be ahead of the demand. MODFLOW modeling was also performed to determine the 90-day system capacity with the implementation of HCW-5 and HCW-6. These analyses indicate that with the two future wells, LWS's projected seasonal capacity would be 105 mgd.

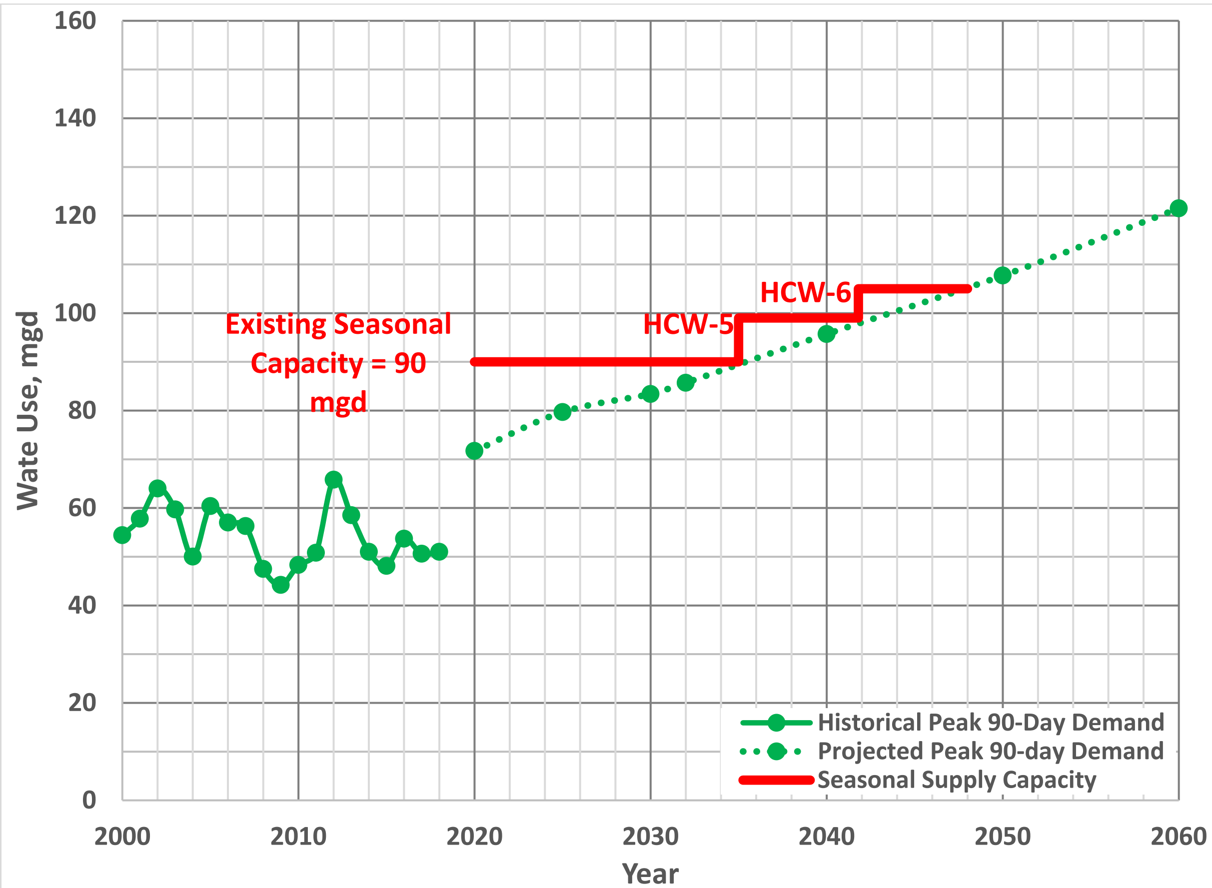

Figure ES-7 Future Supply Expansion

ES.5 Water Treatment

LWS owns and operates two water treatment facilities co-located near Ashland. The East Plant consists of ozone and chlorine for primary disinfection, followed by dual media filtration and chloramines for secondary disinfection. The West Plant consists of aeration, chlorine for primary disinfection, sand filtration and chloramines for secondary disinfection.

ES.5.1 Water Quality Trends

Based on an analysis of the water quality and operating data received, the LWS Ashland plants appear to be in compliance with the applicable rules and regulations. However, as the City continues to expand the use of HCWs, WTP improvements will be required to address more challenging water quality conditions. Since the HCWs are hydraulically connected with the Platte River, water quality from the HCWs is characterized by warmer water temperatures and higher concentrations of atrazine, arsenic, and total organic carbon (TOC). Given these trends in water quality and future installation of HCWs, additional treatment measures will be required to address arsenic and atrazine levels in the future.

Given the relatively high concentrations of atrazine in the Platte River, LWS has undertaken atrazine management practices during the spring and summer when agricultural runoff contributes to elevated atrazine levels. Atrazine management practices include ozonation and limiting the use of the HCWs, which experience higher concentrations of atrazine.

As with atrazine, LWS has had to implement wellfield management practices to maintain compliance with the arsenic MCL. While LWS has maintained regulatory compliance for arsenic, the concentration of arsenic in the raw water supplied from the HCWs appears to be increasing over time, trending towards the MCL of 10 µg/L. Figure ES-8 shows the concentration of arsenic in raw water samples collected from the East and West Plants from January 2017 through August 2019.

LWS will likely need to implement a treatment system in the future to address the relatively high concentrations of arsenic in the HCWs and expected concentrations of arsenic in the future HCWs. Additional bench-scale testing is recommended to further investigate treatment alternatives and identify a cost-effective solution for arsenic treatment.

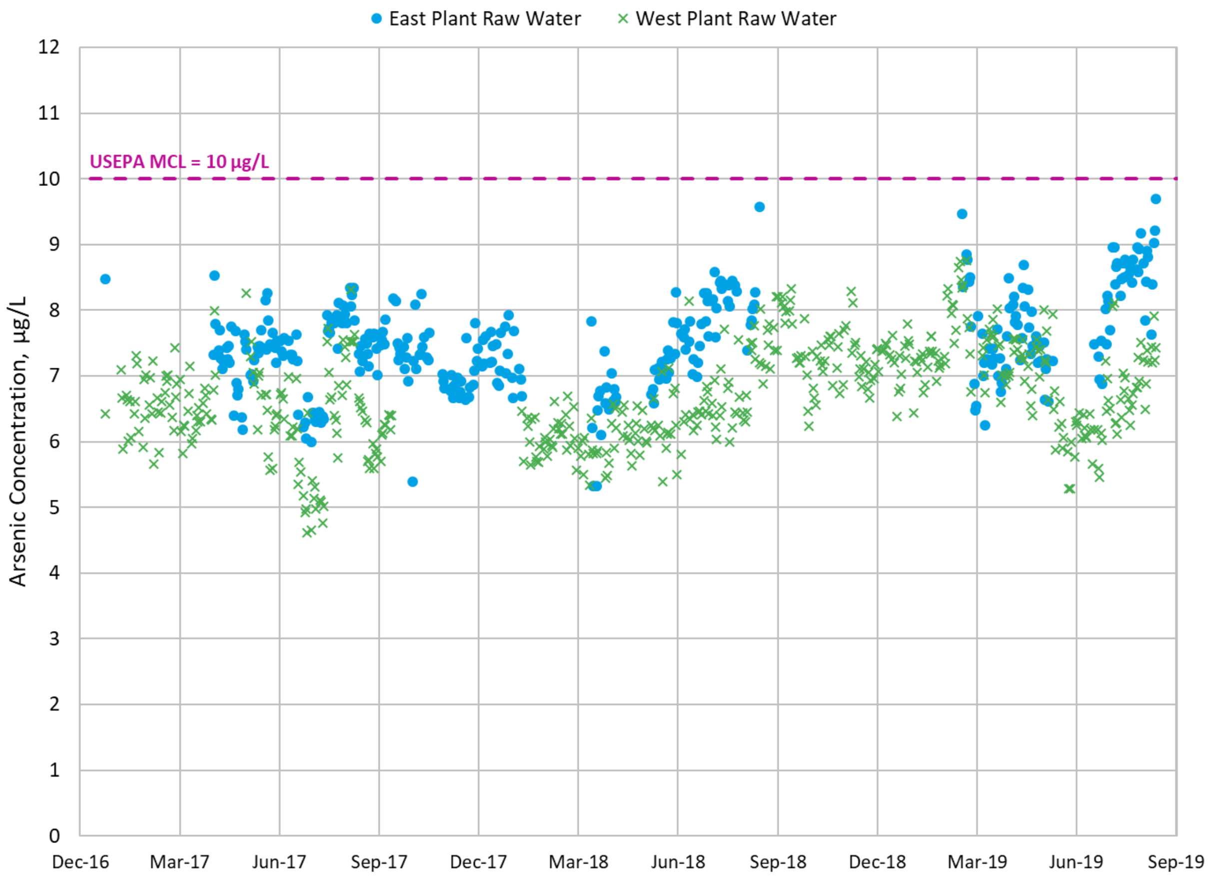

Figure ES-8 Raw Water Arsenic Concentration from January 2017 through August 2019

ES.6 Potential Future Regulations

ES.6.1 Proposed Lead and Copper Rule Revisions

The U.S. Environmental Protection Agency (EPA) announced proposed revisions to the Lead and Copper Rule (LCR) in October 2019 with promulgation of the final rule anticipated in Year 2020. The proposed LCR includes several revisions with a focus on proactive measures to improve finished water quality at the customers' tap. While final revisions to LCR are still being developed, major changes in the proposed LCR revisions include:

- Public water systems (PWSs) must develop a publicly available lead service line (LSL) inventory (including lead goosenecks and downstream galvanized iron service lines on both PWS's side and homeowner's side).
- Retain the current lead AL of 15 µg/L, and add a new lead trigger level of 10 µg/L. If the 90th percentile lead concentration exceeds the new trigger level of 10 µg/L, the PWS would be required to conduct a corrosion control study to optimize or develop a CCT, complete annual LCR monitoring, conduct public outreach and establish an annual goal for LSL replacement.
- If the 90th percentile lead level exceeds the AL, then the PWS must fully replace 3 percent of LSLs annually for consecutive 6-month monitoring periods.

- PWSs must "find-and-fix" sites with lead levels greater than the AL, conduct additional sampling, and work with their Primacy Agency to identify if corrective actions are needed.
- Partial LSL replacements would no longer be allowed except in rare circumstances.
- LCR compliance sampling modifications would include a new Tier structure with LSLs as Tier 1 and copper pipe with lead solder as Tier 3; additionally, pre-flushing and removal of aerators would be prohibited, and the use of wide-mouth sample bottles would be required.
- PWSs must test for lead at 20 percent of schools and 20 percent of childcare facilities.

ES.6.2 PFAS Action Plan and Regulatory Determination

Per- and polyfluoroalkyl substances are a class of thousands of man-made chemicals that are used in the manufacture of many industrial and consumer products. PFAS chemicals are heat stable, non-biodegradable, bioaccumulative, and very persistent in the environment. Due to their widespread application, PFAS are now found in many drinking water sources across the United States. In 2016 the EPA established non-enforceable drinking water health advisory levels for two prevalent PFAS chemicals, perfluorooctanoic acid (PFOA) and perfluorooctane sulfonic acid (PFOS) as a total concentration of 70 ng/L.

In February 2019, the EPA issued a PFAS Action Plan, aimed at comprehensively addressing PFAS in the environment. The EPA has proposed regulating PFAS under the Safe Drinking Water Act (SDWA), the Toxic Substances Control Act (TSCA), the Comprehensive Environmental Response, Compensation and Liability Act (CERCLA, also known as Superfund), and the Clean Air Act. Currently, there are no federal MCLs established for PFAS chemicals under the SDWA. However, in February 2020, the EPA announced that it intends to regulate both PFOA and PFOS under the SDWA.

ES.6.3 Water Plant Expansion

The existing treatment capacity of 120 mgd for the combined East and West Plants is capable of meeting projected demands through the Year 2037. The *2014 Facilities Master Plan* had identified the next plant expansion to occur at the West Treatment Plant by means of filter rehabilitation. The scope of this master plan update included additional focus on condition assessment of the existing treatment plants, along with input from operations, to take a second look at this approach and compare expansion of the two plants.

Concerns have been raised by plant staff about the feasibility of treating over 70 mgd through the West Plant. In order to expand the West WTP, additional modifications beyond filter rehabilitation would be required. Other recommended improvements include replacement of the existing clearwell transfer pumps, addition of a fourth aerator and chlorine contact basin, chemical feed modifications, and an allowance for hydraulic improvements.

Alternatively, the East WTP currently has a capacity of 60 mgd and is expandable in increments of 30 mgd to provide an ultimate capacity of 180 mgd. East Plant expansion alternatives considered the addition of either two filters (15 mgd) or four filters (30 mgd), additional ozone capacity and associated infrastructure. The cost to add only two filters was not deemed to be in the City's best interest as it would be inefficient with respect to building walls, foundations, ozone system expansion, etc. Therefore, expansion of the East Plant by an additional 30 mgd is recommended.

ES.7 Distribution System Facilities

The LWS service area is currently divided into the following service levels: Low, High, Belmont, Southeast, Cheney and Northwest. Service level boundaries are established to maintain acceptable distribution system pressures.

A desktop evaluation was conducted to determine whether the existing distribution system facilities have sufficient pumping capacity and storage capacity to meet future demands for the 2032 planning period. Based on a capacity evaluation, there are no pumping capacity deficits through the 12-year CIP that need to be addressed. The desktop evaluation suggests that there are storage deficiencies under emergency conditions in the High, Northwest and Cheney Service Levels. These storage deficiencies can be addressed through operational practices, “smart watering” programs and planned infrastructure improvement projects.

ES.7.1 Focus Areas

Distribution system modeling was conducted using extended period simulations (EPS) for maximum day demand conditions. Three focus areas were assessed specifically in this Master Plan update.

- North 56th Street and I-80.
- Folsom and Old Cheney.
- 27th and Rokeby.

Hydraulic modeling scenarios were performed for Year 2020 and 2032 to evaluate potential improvements required for localized large user demands near North 56th St and I-80. Recommended improvements include implementation of a booster pump station south of I-80 near Arbor Rd and 56th St and implementation of a 24-inch north loop to provide full redundancy.

The area around Folsom and Old Cheney is expected to grow from a population of 550 to over 4,000 between Year 2026 and Year 2040. This area is served by a long 16-inch main and has no other redundant feed. Modeling results indicate that a pipe improvement along Old Cheney through Wilderness Park and installation of a bi-directional control valve could allow for bi-directional flow between High and Belmont Service Levels, providing the level of redundancy needed.

ES.7.2 Distribution System Modeling

The goal of the EPS modeling was to verify the desktop evaluations for storage and pumping capacities and see how the system responds to a design (extremely hot and dry year) demand condition. The model results support the desktop evaluation in that there is generally excess pumping capacity and storage to meet Year 2020 maximum day demands and the ability to refill storage during replenishment conditions exists. However, the rapid draft rate of Southeast and S. 56th Street tanks during peak hourly demands indicate that hydraulic restrictions do occur when pumping into the High Service Level. The addition of a pump at the Vine East Station, scheduled for Year 2020, will allow for more pumping from the Low Service Level to the Southeast Service Level, reducing flows from High to Southeast and associated hydraulic restrictions. Additionally, the Adams Street Reservoir, scheduled in Year 2030, will provide equalization and emergency storage in the High Service Level.

The results of the 2020 maximum day EPS scenario support the addition of Pump No. 8 at the Vine East Pumping Station East and the addition of the Adams Road Reservoir and pipelines in the 12-year CIP. Pipeline improvements in the Belmont Service Level between "O" Street and Partridge are recommended in the 6-year CIP and will provide support to an area which could experience low pressure.

Several areas in the distribution system with pressures ranging from 30 to 35 psi were identified, whereas only two areas with pressures above 120 psi were identified. The areas along boundaries should be monitored during design years and if it is determined that low pressures are resulting in customer complaints, pressure reducing valves could be added at the boundary locations. The same notable lower-pressure areas in the distribution system also occurred in the 2032 EPS scenario with the exception of the high ground area in the Cheney Service Level which has improved to above 40-psi.

Several of the items in the CIP were evaluated through the 2020 EPS and 2032 EPS base modeling scenarios. Others were individually evaluated to determine their need and usefulness. Several additional scenarios were performed, unique to the improvement being evaluated. Modeling scenarios include construction of new pumps, pumping stations, reservoirs, water mains, and PRVs, as well as other infrastructure improvements, rehabilitation efforts and decommissioning of existing pumping stations.

ES.8 Distribution Water Quality

Based on a review of distribution water quality data, LWS has demonstrated effective management of DBPs and as a result, is on reduced monitoring for bromate, TTHM and HAA5. LWS has maintained a bromate RAA of less than 25 percent of the MCL since 2013. Similarly, the LRAA for TTHMs has consistently been less than 40 μg/L (50 percent of the MCL), and the LRAA for HAA5s has been maintained at less than 20 μg/L (33 percent of the MCL).

LWS is also on reduced monitoring for lead and copper, which requires LCR compliance data to be collected every three years. The 90th percentile values for lead and copper compliance monitoring in 2013, 2016 and 2019 have been below the action levels of 15 μg/L and 1300 μg/L, respectively. The proposed LCR revisions have proposed a new lead trigger level of 10 μg/L to prompt water systems to take proactive actions to reduce lead levels prior to exceeding the lead AL. Since 2004, the 90th percentile value for lead has been less than 5 μg/L, which is well below the proposed trigger level. Additionally, given the LWS's existing LSL inventory and replacement plan, LWS is well-positioned to comply with the potential requirements for implementing a publicly available LSL inventory and proactive, full LSL replacement program.

Between 2014 and 2017, LWS experienced challenges with nitrification between the months of August and December. Nitrification was characterized by rising water temperatures, loss of chlorine residual, increases in nitrite concentration, and in some locations, occurrences of HPCs. In 2018, LWS made significant improvements in distribution system water quality through various nitrification control measures, which resulted in increased chlorine residuals throughout the distribution system and reduced nitrite and nitrate concentrations. The nitrification control measures included increasing the chlorine residual at the POE, taking the East Plant out of service during peak nitrification season, and reducing water age in the distribution system by isolating and reducing operating volumes in reservoirs.

This resulted in considerable improvements to distribution system water quality in the High, Low and Southeast Service Levels. However, the areas surrounding Air Park, Northwest, Cheney, and southern parts of Southeast still had difficulty maintaining chlorine residuals greater than 0.5 mg/L at the distribution system monitoring sites. Additionally, alternative long-term solutions should be investigated, since taking the East Plant out of service limits the overall plant capacity and is not sustainable for future operations. Potential long-term solutions include:

- Chloramine booster systems within the distribution system.
- Improvements to tank mixing in distribution system reservoirs.
- Biological filtration at the East Plant.
- Sodium chlorite feed at the East and West Plant.

Given the continued challenges in Air Park, Northwest, Cheney and southern parts of Southeast Service Levels; a source trace analysis was conducted to identify optimal locations for chloramine booster systems. Source trace analysis is used to identify the percentage of water that comes from a given source, allowing for easier identification of areas that can provide a high impact on water quality. Based on the source trace analysis, it was determined that chloramine booster systems should be implemented at Yankee Hill and Pioneers.

- **Yankee Hill** – Most of the water in the Cheney SL and southern parts of Southeast SL has passed through the Yankee Hill reservoir, making it an ideal location for rechloramination. It is also recommended that a PRV be installed around 84th and South Street to allow rechloraminated water to be transferred to the High SL to address pockets with low chlorine residual.
- **Pioneers** – The source trace analysis found that during winter operations, over 80 percent of the water in Air Park and at least 60 to 80 percent of the water in the Northwest SL has been pumped through Pioneers Pumping Station. With such a high proportion of water from Pioneers being delivered to these areas, there is a meaningful opportunity to improve distribution water quality through rechloramination at Pioneers.

For the time being, it is recommended that LWS continue with their current nitrification control measures, while other in-plant treatment and distribution system management alternatives are evaluated. The following alternatives for distribution system water quality improvements are recommended for further evaluation through pilot testing. Each of the proposed treatment alternatives should be compared with the plant's current operating conditions to establish a baseline and determine the preferred approach for nitrification control.

- **Biological filtration** – This alternative considers implementation of biological filtration in the East Plant to reduce the concentration of AOC, which is increased during the ozonation process. Reducing the AOC in the finished water will improve biological stability in the distribution system, which could allow for continued use of the East Plant during peak nitrification seasons.
- **Sodium chlorite** – This alternative considers feeding 0.3 mg/L of sodium chlorite to the plant finished water. Sodium chlorite is particularly effective at inactivating ammonia oxidizing bacteria and has proven to be effective for nitrification control for other utilities in the Midwest.
- **Improvements to Tank Mixing** – This alternative considers field-testing to evaluate the performance of existing distribution system tank mixing systems to provide guidance on future implementation strategies to reduce potential for stratification.

ES.9 Recommended Improvements

A comprehensive capital improvements program was prepared based on findings from the hydraulic analyses, plant and distribution water quality analyses, WTP condition assessment, transmission main criticality assessments, and projections for overall system growth. The recommended phased improvements summarized in this report represent an update to the *2014 Facilities Master Plan*. Changes to the CIP are a result of updated demand projections, which impact the schedule for implementation. Other changes to the CIP were predicated on additional input from the City, along with alternative analysis by the Black & Veatch.

The phases of the program are summarized below:

- **Phase I – Immediate Improvements:** Phase I improvements have been identified as higher priority as a result of their immediate need or as a result of currently anticipated development and correspond to FY 2019/2020 thru 2025/2026. These improvements are intended to meet the needs of the Comprehensive Plan – Tier 1 (Priority A) growth areas.
- **Phase II – 12-Year Short-Term Improvements:** Phase II improvements are recommended to meet projected water demands from FY 2026/2027 through FY 2031/2032. The Phase II improvements will extend service to the limits of the Tier I –Priority B area.

Table ES-1 summarizes the recommended immediate and short-term improvements included in the 12-Year CIP, as well as the proposed schedule for implementation and opinion of probable construction costs for each activity. Phase I improvements are identified by the code “IM” for immediate improvements. Phase II improvements are identified by the code “ST” for short-term improvements. Other improvements that extend beyond the 12-Year CIP planning period are identified by the code “LT” for long-term improvements.

Table ES-1 Recommended Improvements – Schedule and Cost Summary

Recommended Improvements				
Year	CIP Tag	Description	Improvement Type	Total Capital Cost (FY 2020)
Phase I – Immediate Improvements				
2020	IM-1	Valve Replacement and Automation at 51st Street PS	Facility	$380,000
2020	IM-2	NW 12th Street Pumping Station	Pumping	$4,608,000
2020	IM-3	Vine Street Pumping Station East - Add Pump No. 8 w/ AFD	Pumping	$2,357,000
2020	IM-4	Innovation Campus - Phase 1 - 16-inch Main	Distribution	$1,172,000
2021	IM-5	I-80 & 56th Street Pumping Station - Supply Main and PS	Pumping	$5,760,000
2021	IM-6	I-80 & 56th Street Pumping Station - Belmont Loop	Distribution	$5,607,000
2021	IM-7	Arsenic/Atrazine Study and Preliminary Design	Treatment	$250,000

Recommended Improvements				
Year	**CIP Tag**	**Description**	**Improvement Type**	**Total Capital Cost (FY 2020)**
2022	IM-8	Distribution Water Quality Improvements - Phase 1	Distribution	$3,013,000
2022	IM-9	16-inch Main on NW 56th Street, "O" St. to Partridge Lane	Distribution	$1,439,000
2022	IM-10	Decommission Merrill Street Pumping Station	Pumping	$306,000
2022	IM-11	Rehabilitate Eddy Current Drive - Northeast #6	Pumping	$121,000
2022	IM-12	West Water Treatment Plant Rehabilitation	Treatment	$2,285,000
2023	IM-13	31st and Randolph Valve Vault Relocation to "A" street	Facility	$343,000
2023	IM-14	Add 20.9 mgd WTP South Pumping Station Pump No. 13	Pumping	$1,806,000
2023	IM-15	2023 Master Plan	System	$1,000,000
2024	IM-16	Add AFD's at Pioneers Pumping Station	Pumping	$236,000
2024	IM-17	Pressure Monitoring Stations	Distribution	$165,000
2024	IM-18	East Plant Overall Rehab	Treatment	$669,000
2024	IM-19	Decommission South 56th Street PS	Pumping	$300,000
2024	IM-20	Condition Assessment of 36-inch Cast Iron from 51st to A Street	Condition	$223,000
2024	IM-21	Condition Assessment of 48-inch PCCP from Ashland to NE	Condition	$310,000
2024	IM-22	Condition Assessment of 54-inch PCCP from Northeast to Vine	Condition	$471,000
2025	IM-23	Arsenic Treatment - Adsorber	Treatment	$40,704,000
			Subtotal	**$73,525,000**
Phase II – Short-Term Improvements				
2026	ST-1	Northwest Reservoir (2 MG) and Pipeline	Storage	$5,918,000
2026	ST-2	Belmont to Low PRV Station ("O" Street and N 12th Street)	Distribution	$180,000
2027	ST-3	Decommission NW 12th Street Pumping Station	Pumping	$320,000
2027	ST-4	Decommission Cheney Pumping Station	Pumping	$313,000
2027	ST-5	Yankee Hill Pumping Station - Add 6 mgd Pump	Pumping	$518,000

Recommended Improvements				
Year	CIP Tag	Description	Improvement Type	Total Capital Cost (FY 2020)
2027	ST-6	PRV Southeast SL to High SL - Vault near Southeast PS	Distribution	$180,000
2027	ST-7	Innovation Campus - Phase 2 - 12-inch Main	Distribution	$729,000
2028	ST-8	Distribution Water Quality Improvements - Phase 2 (Pioneers WQ)	Distribution	$1,382,000
2030	ST-9	Adams Street Reservoir and Pipelines for HSL (5 MG)	Storage	$11,985,000
2032	ST-10	54-inch Main from Northeast PS to 88th and Holdrege	Transmission	$26,695,000
			Subtotal	**$48,220,000**
Long-Term Improvements				
2033	LT-1	36-inch Transfer Main from Vine Street Reservoir to A Street Reser	Transmission	$17,518,000
2033	LT-2	Horizontal Collector Well No. 5 - Site 7	Supply	$12,136,000
2034	LT-3	Water Treatment Plant Expansion - Ozone and East Filters	Treatment	$24,804,000
2041	LT-4	Horizontal Collector Well No. 6 - South Site	Supply	$11,922,000
		New Source of Supply Reserve Fund	Supply	$22,000,000
		Lead Service Line Replacement Program	Distribution	$35,280,000
			Subtotal	**$123,660,000**
Total Cost				**$245,405,000**

In addition to these improvements for supply, treatment, transmission capacity, and storage, system growth requires distribution system "main extensions" to serve developing areas. These main extensions, as well as recommended fire flow improvements, required over the next 12 years are summarized in Table ES-2.

Table ES-2 Recommended Fire Flow and Main Extension Improvements

Year	Description	Total Capital Cost (FY 2020)
Immediate	Fire Flow	$1,110,000
Immediate	Main Extensions	$14,263,000
Year 2020-2026	6-Yr Main Extensions	$27,966,000
Year 2027-2032	12-Yr Main Extensions	$41,215,000
Total Cost		**$84,554,000**

1.0 Introduction

1.1 Purpose

This report has been prepared to provide the City of Lincoln with an update to the 2014 Water Facilities Master Plan (*2014 Master Plan*). Most significantly this report has evaluated revised background data for population and demand forecasts, coupled with impacts associated with climate change, to develop a new 12-year capital improvement program. The recommended improvements plan presented herein will serve as a basis for the planning, design, construction, and financing of facilities to meet the city's anticipated population growth and commercial development through Year 2032. The purpose of the recommended improvements is to provide an adequate and dependable supply of water to existing and future customers.

1.2 Scope

The study period for analysis of population and demand projections is from Year 2020 through the Year 2060. These analyses form the framework for understanding timing for expansion of the existing water supply and water treatment systems. Hydraulic analyses of the distribution system were limited to only evaluating design Years 2020 and 2032 since this study is an update to the previous plan.

The study area for this investigation and report is shown on Figure 1-1 located at the end of this chapter. The various components of the Study Area have been delineated by the Lincoln-Lancaster County Planning Department in the updated *2040 Comprehensive Plan* (*LPlan 2040*) as adopted in December, 2016. In general, the areas evaluated as part of this study include Tier I – Priorities A (Developing), Priorities B (Year 2025), and parts of Priority C (Year 2040).

The principal elements of this master plan study update include consideration and evaluation of the following:

- <u>Update Population Projections</u> - Update the population projections to be consistent with the *LPlan 2040*. Design Years will include Year 2020, 2025, 2040, and 2060. Historical water use trends and projections of future water requirements as originally developed for the *2014 Master Plan* were based on recent population projections provided by the Lincoln-Lancaster County Planning Department.
- <u>Revise Demand Projections</u> – Evaluate trends in water use and update demand projections taking into account climate change. Assign base year (2020) and Year 2032 demands to the hydraulic model.
- <u>Update Water Supply Projections</u> – Determine 30-day, 60-day, and 90-day water supply yields utilizing existing groundwater model. Utilize basin wide groundwater modeling tools, with adjustment for climate impacts, to revise streamflow input into the groundwater model. The chapter on water supply also includes an assessment of Lincoln's current water rights.
- <u>Evaluate the Water Treatment Plant</u> – Review historical records to confirm compliance with water treatment regulations. Perform high level condition assessment to determine necessary improvements for ongoing reliable operations. Evaluate timing and need of plant expansion based upon revised demands, condition assessment, and process considerations.

- Distribution System Analysis - Update the computer model of the Lincoln water distribution system in InfoWater hydraulic analysis software and perform analyses for average day, maximum day, and maximum hour scenarios for Years 2020 and 2032. Specific focus areas to be evaluated include 56th and I-80, 27th and Rokeby, and Folsom and Old Cheney.
- Perform Distribution Water Quality Analyses – Evaluate available historical water quality data, perform distribution water age analyses, and develop protocol for system improvements which enhance water quality in the system.
- Update Transmission Condition Assessment – Develop condition assessment program for the transmission system based upon available technology, inspection cost, pipe material, and main criticality.
- Lead Service Line Review – Review existing records to quantify existing lead service lines, provide summary of regulations and replacement strategies, and summarize potential funding options.
- Capital Improvement Program – Prepare an update of recommended water system improvements.

1.3 Climate Change Considerations

Climate change continues to become an ever-increasing concern to the general public given the volatility of recent weather patterns in the State. This master plan update provided an opportunity for the Lincoln Water System to consider the impacts of climate change for the first time in their water supply planning process. The specific climate change impacts considered under this study included reduced supply capacity as a result of higher temperatures, reduced streamflows, and more variability in precipitation, as well as an increased summer seasonal peak 90-day demand expected due to longer periods of dry weather.

1.4 Acronyms and Abbreviations

Acronyms and abbreviations used in this report are as follows:

AD	(Annual) Average Day
AL	Action Level
AM	Average Month
AOB	Ammonia Oxidizing Bacteria
AWWA	American Water Works Association
BG	Billion Gallons
BPS	Booster Pumping Station
CCI	Construction Cost Index
CCL	Contaminant Candidate List
CCT	Corrosion Control Treatment
CDBG	Community Development Block Grant
CFE	Combined Filter Effluent
cfu	Colony Forming Unit
CIP	Capital Improvements Program
Cl_2	Chlorine

CWSRF	Clean Water State Revolving Fund
DBPR	Disinfectant/Disinfection Byproduct Rule
D/DBPR	Disinfection/Disinfectant By-Product Rule
DWSRF	Drinking Water State Revolving Fund
El.	Elevation
ENR	Engineering News Record
EPA	(United States) Environmental Protection Agency
EPS	Extended Period Simulation
ESRI	Environmental Systems Research Institute
ft	Feet
ft^2	Square Feet
gal	Gallons
GFH®	Granular Ferric Hydroxide®
gpcd	Gallons Per Capita per Day
gpm	Gallons Per Minute
gpm/ft^2	Gallons Per Minute per Square Foot
GIS	Geographic Information Systems
GWUDI	Ground Water Under the Direct Influence
HAA5	Five regulated haloacetic acids
HCW	Horizontal Collector Well
HELP	Homeowner’s Emergency Loan Program
HG	Hydraulic Gradient
HGL	Hydraulic Grade Line
hp	Horsepower
HUD	Department of Housing and Urban Development
ICI	Industrial/Commercial/Institutional
IDSE	Initial Distribution System Evaluation
in.	Inch
ISO	Insurance Services Office
LCR	Lead and Copper Rule
LOX	Liquid Oxygen
LSL	Lead Service Line
LT2ESWT	Long-term 2 Enhanced Surface Water Treatment Rule
LWS	Lincoln Water System
MCL	Maximum Contaminant Limit
MCLG	Maximum Contaminant Limit Goal
MD	Maximum Day
MG	Million Gallons

mgd	Million Gallons per Day
mg/L	Milligrams per Liter
MH	Maximum Hour
min	Minutes
mL	Milliliter
MM	Maximum Month
ND	Non-detect
NPDWR	National Primary Drinking Water Regulations
NRW	Non-Revenue Water
NTU	Nephelometric Turbidity Units
ppd	Pound Per Day
ppmv	Parts Per Million by Volume
PRV	Pressure Reducing Valve
PSA	Pressure Swing Adsorption
psi	Pounds per Square Inch
PWS	Public Water System
rpm	Revolutions Per Minute
SCADA	Supervisory Control and Data Acquisition
SL	Service Level
SMP	Standard Monitoring Plan (for Stage 2 D/DBPR)
SP	Seasonal Peak
SSS	System Specific Study (for Stage 2 D/DBPR)
SWTR	Surface Water Treatment Rule
TAZ	Traffic Analysis Zone
TDH	Total Dynamic Head
TOU	Time of Use
TTHM	Total Trihalomethanes
µg/L	Micrograms per Liter
UNF	Unaccounted-for Water
USEPA	United States Environmental Protection Agency
USGS	United States Geological Survey
WIFIA	Water Infrastructure Finance and Innovation Act
WIIN	Water Infrastructure Improvements for the Nation
WQP	Water Quality Parameter
WSE	Water Surface Elevation
WTP	Water Treatment Plant

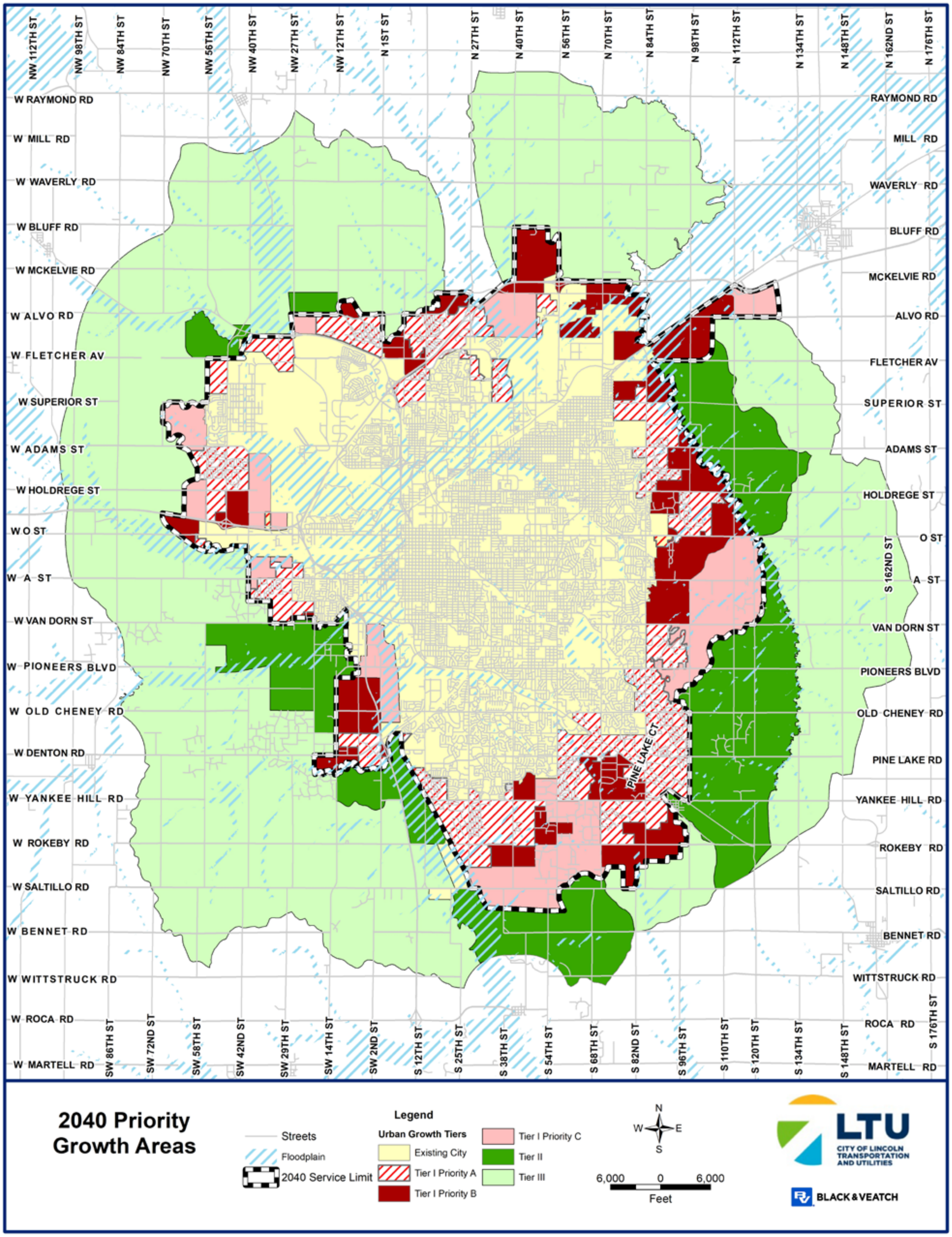

Figure 1-1 2040 Priority Growth Area

2.0 Population

Development of a comprehensive water system master plan begins with an evaluation of the area's historical populational trends and projected growth patterns. To accurately predict future water demands, it is necessary to determine the magnitude, location, and characteristics of future population growth.

2.1 City of Lincoln Population

Historical population data was obtained from the U.S. Census Bureau. Population projection data for the City of Lincoln was also obtained from the current *LPlan 2040* that delineates the spatial distribution of growth within the growth tiers by Traffic Analysis Zones (TAZ) through Year 2040. Beyond 2040, the *LPlan 2040* uses an extrapolation to develop the 2060 projections.

The following key points present a high-level summary of the *LPlan 2040*:

- Lancaster County will continue to grow at a rate of 1.2 percent per year over the next 40 years.
- The City of Lincoln will continue to serve approximately 90 percent of the county population.
- Twenty percent of the future dwelling units are expected to be built within existing developed areas.
- The spatial distribution of population includes a higher percentage of infill and redevelopment than previous plans.
- The number of people per household is expected to trend downward slightly over the next 20 years.
- Population projections for Lancaster County for Year 2040 and Year 2060 are 413,000 and 523,000 respectively.
- City of Lincoln population (service population) for Year 2040 and Year 2060 are 371,700 and 470,700 respectively.
- Year 2040 population projections did not change in the update.
- Year 2060 population projections increased slightly from a previous projection of 461,700 to 470,700 persons.

The City has indicated that the *LPlan 2040* is currently being updated, but the projections will not be final until the Plan is adopted, anticipated to be in 2021. The preliminary projections shared for the upcoming update indicate that population is projected to increase at a slightly lower rate than in the current *LPlan 2040*. This indicates that the population projections used in this Master Plan update and based on the current *LPlan 2040* (adopted in December 2016) may be higher than future projections from the upcoming *LPlan 2040* update. This provides some conservatism to the population projections for this Master Plan update. Table 2-1 presents the historical and projected population through 2060 and Figure 2-1 shows this graphically.

Table 2-1 City of Lincoln Population (Historical and Projected)

Year	Population	Average Annual Growth	
		Persons	Percentage
1940	81,984	-	-
1950	98,884	1,690	1.9%
1960	128,521	2,964	2.7%
1970	149,518	2,100	1.5%
1980	171,932	2,241	1.4%
1990	191,972	2,004	1.1%
2000	225,581	3,361	1.6%
2010	261,796	3,622	1.5%
2020	291,677	2,988	1.1%
2025	309,902	3,645	1.2%
2030	329,266	3,873	1.2%
2035	349,840	4,115	1.2%
2040	371,700	4,372	1.2%
2045	394,303	4,521	1.2%
2050	418,281	4,796	1.2%
2055	443,717	5,087	1.2%
2060	470,700	5,397	1.2%

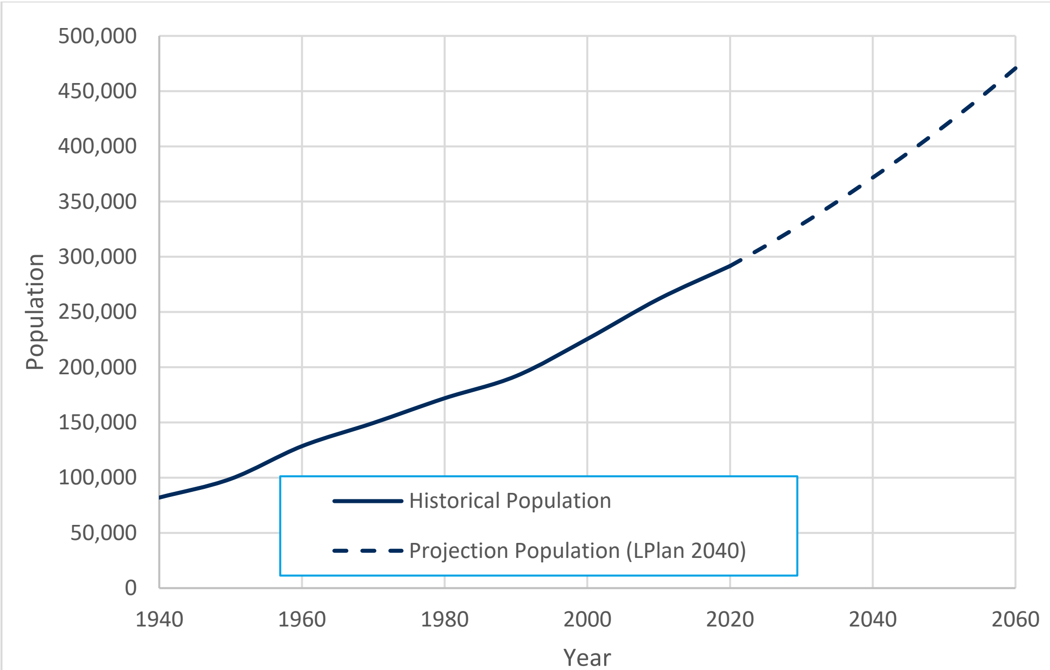

Figure 2-1 City of Lincoln Historical and Projected Population

2.2 Population Distribution

The Lincoln-Laster County Planning Department provided spatial distribution of populations and households within the county for Years 2015, 2026 and 2040. The data included number of households and population by TAZ which covered the entire study area, including the Tier II and Tier III development limits. This spatial data was disaggregated by Service Level and by TAZ for the planning period through Year 2040. No spatial distribution of population was available beyond Year 2040, so the overall growth rates by Service Level from Year 2040 were used to estimate the population by Service Level beyond Year 2040.

2.2.1 Population by Service Level

Census population data was used to update the population by Service Level for Year 2010 and the *LPlan 2040* data was used to develop the population projections by Service Level for the planning period, included Year 2020 benchmark population, short-term, mid-term, and long-term populations. The spatial distribution was benchmarked for the estimated Year 2020 population based on the *LPlan 2040* and Year 2010 Census data. TAZ that were split between Service Levels (overlapping two or more Service Levels) were divided based on equal-area percentage of the population to obtain projections by Service Level.

Table 2-2 provides the historical population by Service Level and Figure 2-2 shows this graphically. Table 2-3 provides the projected population by Service Level and Figure 2-3 shows this graphically.

Table 2-2 Historical Population by Service Level

Service Level	1980	1990	2000	2010
Northwest	-	-	-	2,237
Belmont	14,500	18,890	31,830	35,630
Low	64,800	67,100	71,466	78,992
High	81,600	89,210	94,840	104,238
Southeast	12,350	16,770	27,445	37,277
Cheney	-	-	-	3,422
Total	173,250	191,970	225,581	261,796

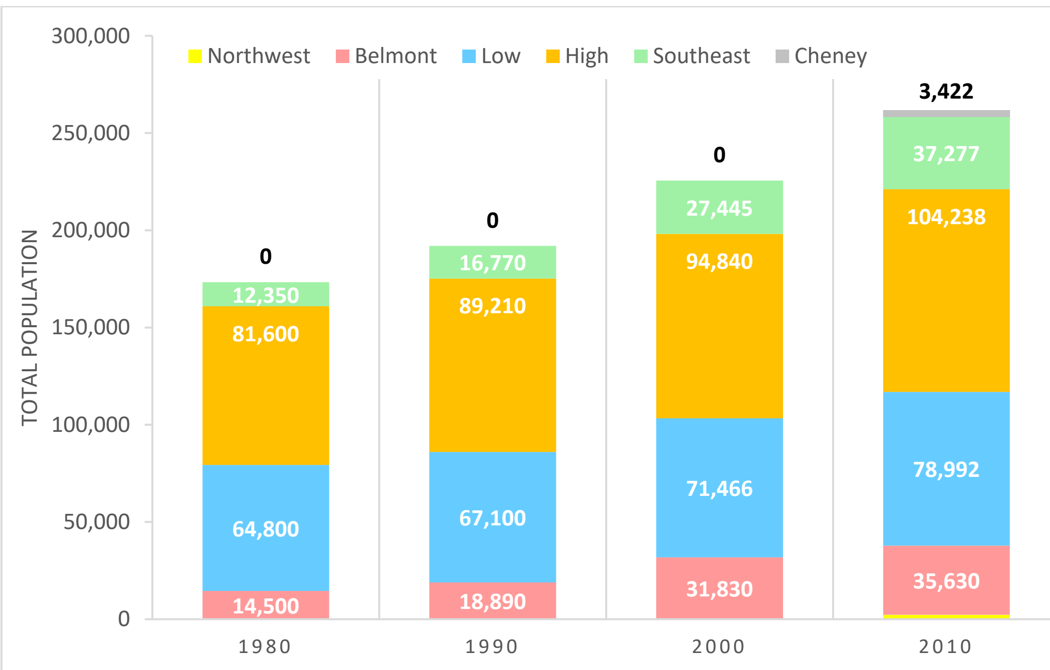

Figure 2-2 Historical Population by Service Level

Table 2-3 Existing and Projected Population by Service Level

Service Level	Base-Year (2020)	Short-term (2025)	Mid-term (2040)	Long-term (2060)
Northwest	3,711	4,578	5,892	7,907
Belmont	42,042	45,673	59,315	80,151
Low	79,903	82,113	92,372	110,180
High	110,483	114,419	127,257	150,064
Southeast	43,501	47,013	65,461	92,978
Cheney	12,037	16,106	21,403	29,420
Total	291,677	309,902	371,700	470,700

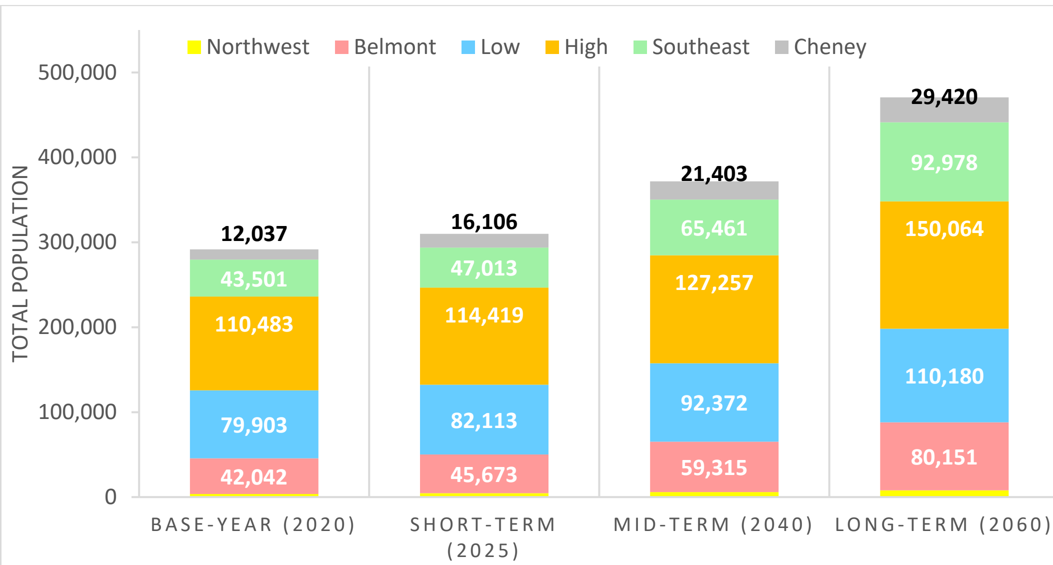

Figure 2-3 Existing and Projected Population by Service Level

In addition to developing the population projections by Service Level, the TAZ data provided from the *LPlan 2040* data for Years 2015, 2026, and 2040 was input into a Power BI dashboard to provide a quick visual reference to the growth hot-spots within the City. This provided a qualitative review of the population breakdown by TAZ and for the City overall. Figure 2-4 shows a few of the characteristics for each of the three years provided. Year 2015 is shown on the left, Year 2026 is illustrated in the center, and Year 2040 on the right. The symbology for the TAZ data shows population density in each of the years with higher populations in dark red (30 persons or greater per acre) and lowest populations in lighter yellows (between 0 and 10 persons per acre). Also shown in the figure are the total populations, categorized by single-family or multi-family, and the ratio between single-family and multi-family for each year. As noted in the *LPlan 2040*, more of the future population will be multi-family and it can be seen that the ratio of multi-family to total population increases slightly over the planning horizon and by Year 2040 will be closer to 24 percent of the total population while currently it is around 22 percent of the total population.

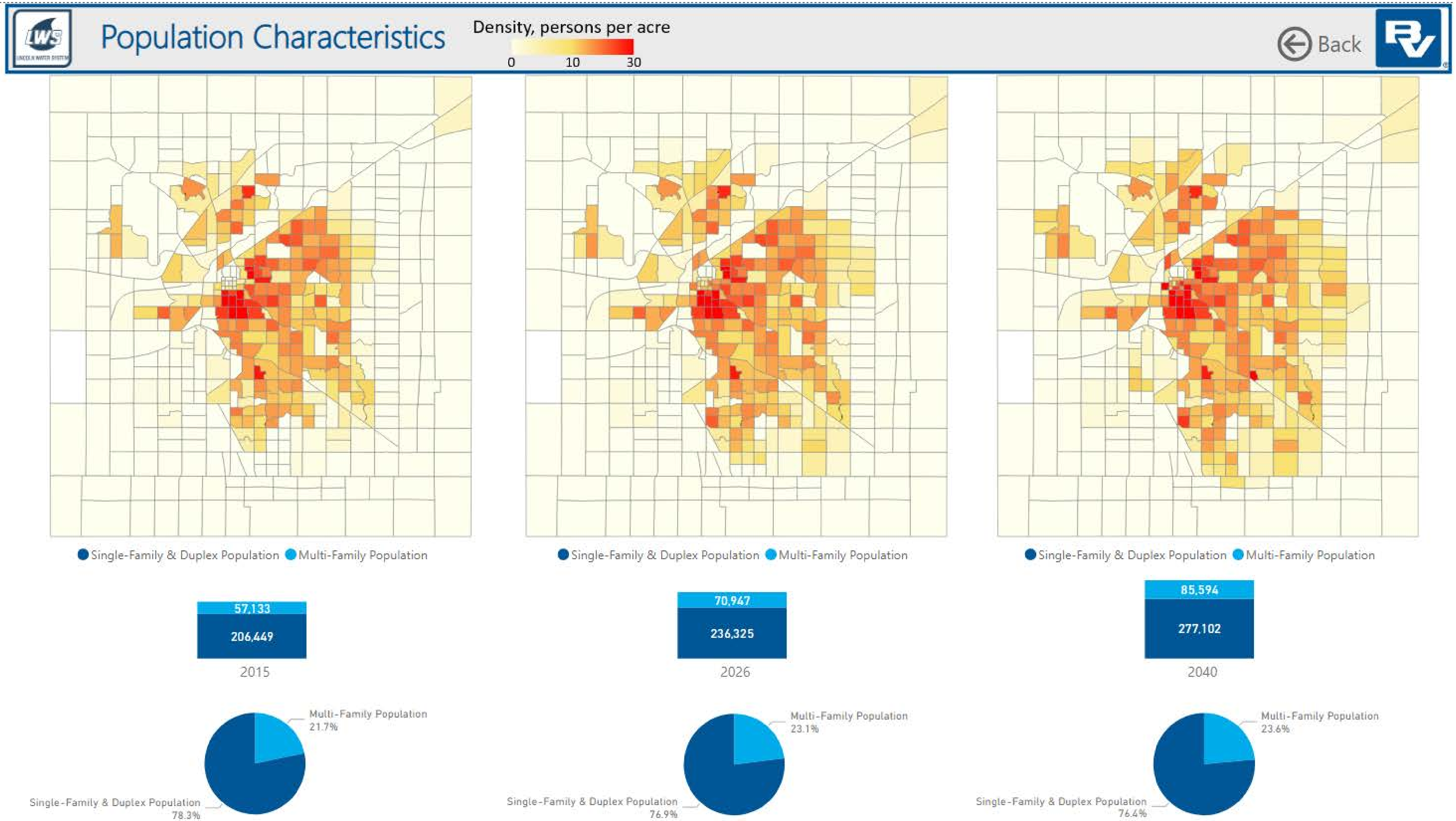

Figure 2-4 Population Characteristics

3.0 Water Capacity Requirements

3.1 General

Development of the water capacity requirements are a critical element of the master plan for consideration in the overall water supply planning as well as the spatial distribution of demands, and peaking factors, for accurate distribution system modeling.

A water utility must be able to supply water at rates that fluctuate over a wide range. Yearly, seasonally, monthly, daily, and hourly variations in water use occur, with higher use during a hot and dry year (i.e. "Design Year"). Water usage also follows a diurnal pattern with peaks in the morning and late afternoon. The rates most important to the hydraulic design and operation of a water treatment plant and distribution system are average day demand (AD), seasonal peak demand (SP), maximum day demand (MD), and maximum hour demand (MH).

- Average Day use is the total annual water use divided by the number of days in the year. The average day rate is used primarily as the basis for estimating maximum day and maximum hour demand for a design year. The average day rate is also used to estimate future revenues and operating costs.
- Seasonal Peak is the average daily use of water over the highest three consecutive months of demand during a given year, generally June through August or July through September. Raw water supply must be evaluated against the seasonal peak design demand to ensure that during a 90-day period of high seasonal demands, the supply capacity is sufficient to meet the water requirements.
- Maximum Day use is the maximum quantity of water used on any one day of the year. The maximum day rate is used to size water supply, treatment facilities, and to determine pumping station capacity needs. The water supply and treatment must be adequate to supply water at the maximum day rate during a design year (hot and dry) which could occur any given year. Pumping capacity must be able to transfer sufficient supply to meet maximum day needs for the system overall and for all individual service levels.
- Maximum hour use is the peak rate at which water is required during any one hour of the year. Since minimum distribution systems are usually experienced during maximum hour, the size and location of storage and pumping facilities are evaluated against this condition. Maximum hour demands are partially met through storage equalization which minimizes the required capacity of transmission mains and permits a more uniform and economical operation of the water supply, treatment and pumping facilities.

3.2 Historical Water Production and Usage

Historical water usage trends and supply characteristics were reviewed and updated to include data from Years 2013 through 2018. Several data sources were used during this update and include:

- Lincoln Distribution Monthly Reports, January 2013 through December 2018
- Production Reports, FY13/14-FY19/20 (FY19/20 does not provide a complete data set until the fiscal year is over at the end of August 2020)
- Total Metered Sales by Customer Class, January 2013 through December 2018
- Metered Sales by Account, All Billing Cycles for 2018

Monthly water treatment plant operating reports (Monthly Reports) were provided from October 2013 through February 2020. Each monthly report contained daily pumpage and usage characteristics for the given month of the report. The most important factors coming from these reports (noted by the same column name from the monthly report) are the following:

- Ashland Pumpage – Well field pumpage is the water delivered to the treatment plant by the wells and is measured by four raw water meters at the head of the treatment plant.
- Treatment and Transmission Usage – In previous master plans, this component which has been referred to as "Treatment and Transmission Usage", is actually a measure of how much water is used directly at the plant, either by process or in-house. This component is the difference between raw water entering the plant (Ashland Pumpage) and the water being pumped into the system in conjunction with storage contribution to meet demands (Lincoln Usage). This is not a discrete column in the pumpage reports and is instead a calculated value. This information is important so that the plant uses are included as a factor in water demand projections. Hereafter in this report, this will be referred to as "Plant Usage" and this term is recommended to use in future Master Plans.
- Lincoln Usage – This data provides the daily calculated usage within the distribution system including non-revenue water. The average of the daily usage over a year equates to AD and the maximum daily value during a given year represents the maximum day of that year. This usage includes the non-revenue water component.
- Lincoln Maximum – This data provides the maximum hour demand for each day. In review of these values, it was noted that the reports can overestimate the maximum hour demand by providing an instantaneous demand from a very discrete time-step (such as a 5-minute period) and the time-step in the calculation could be less than one hour. Corrections were made to some of the maximum hour demands based on discussions with City staff in regard to a few of the over-reported values. City staff provided an updated data set queried from the report to help replace some of the overestimated values in the previous data set and these corrected values were used to review the usage trends.

3.2.1 Ashland Pumpage

The daily Ashland Pumpage was used to develop the average monthly demand and to determine the seasonal peak 90-day pumpage. Figure 3-1 provides the monthly average demands for the last 10 years.

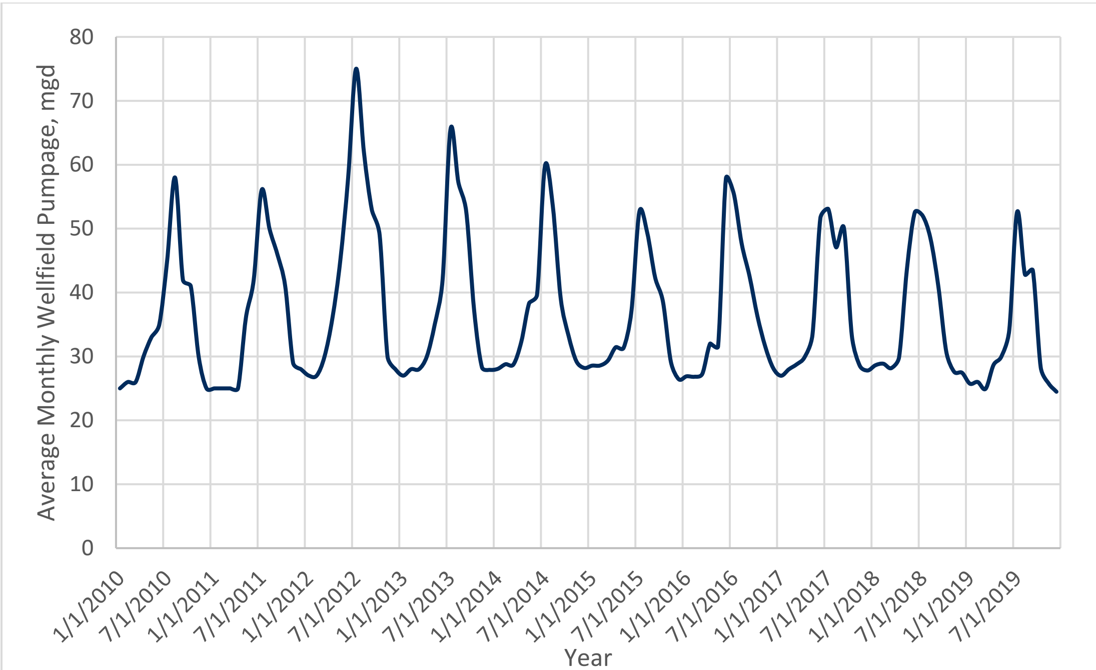

Figure 3-1 Ashland Well Field Pumpage, Monthly Average 2010 - 2019

The SP production is the average daily well field pumpage during the peak three months of water use for each fiscal year. In the recent past, 2012 is considered as a design condition, i.e. hot-and-dry year. The typical SP occurs in June through August or July through September time frame and is used to evaluate future well field production needs. A summary of the Ashland well field pumpage and SP is presented in Table 3-1. The highest seasonal peak/average day (SP:AD) ratio occurred in Year 2012 with a value of 1.56. This value will be used as a design value to obtain the Seasonal Peak 90-day demand based on the average day demand. As noted previously, Year 2019 data is not included in the tables in the following section because the data is incomplete since the fiscal year runs through August 2020.

Table 3-1 Ashland Well Field Pumpage and Seasonal Peak Production

Year	Ashland Well Field Pumpage,		Seasonal Peak Production			
	Total, MG	Average, mgd	Total, MG	Average Day (mgd)	SP Month Time Period	SP/AD
2000	15,041	41.1	5,004	54.4	J, A, S	1.32
2001	14,569	39.9	5,322	57.8	J, A, S	1.45
2002	15,122	41.4	5,884	64	J, J, A	1.55
2003	14,513	39.8	5,491	59.7	J, A, S	1.50
2004	13,885	38.0	4,604	50	J, A, S	1.32
2005	14,775	40.5	5,558	60.4	J, A, S	1.49
2006	14,851	40.7	5,240	57	J, J, A	1.40
2007	13,369	36.6	5,180	56.3	J, A, S	1.54
2008	12,906	35.3	4,371	47.5	J, A, S	1.35
2009	12,512	34.3	4,068	44.2	J, A, S	1.29
2010	12,062	33.0	4,448	48.3	J, A, S	1.46
2011	13,111	35.9	4,675	50.8	J, A, S	1.42
2012	15,747	42.3	6,058	65.8	J, J, A	1.56
2013	14,381	39.4	7,118	58.5	J, A, S	1.48
2014	13,880	38.0	6,205	51	J, J, A	1.34
2015	12,744	34.9	5,852	48.1	J, A, S	1.38
2016	13,491	37.0	6,534	53.7	J, J, A	1.45
2017	13,321	36.4	6,156	50.6	J, J, A	1.39
2018	13,759	37.7	6,205	51	J, J, A	1.35
Planning Criteria						**1.56**
Source: Ashland Well Field, Transmission and Distribution Reports						

3.2.2 Plant Usage

Plant Usage can be calculated by subtracting the Lincoln usage from the Ashland well field pumpage. Table 3-2 shows the Plant Usage for Year 2000 through Year 2018. As noted in the *2014 Master Plan*, additional Plant Usage occurred in Year 2011 through Year 2013 because of increased backwashing due to higher manganese concentrations, but usage has declined since that period. For design values moving forward, it is anticipated to experience Plant Usage similar to those experienced prior to Year 2011 and after Year 2013.

Table 3-2 Plant Usage

Year	Ashland Well Field Pumpage, MG	Total Annual Lincoln Usage, MG	Plant Usage, MG	Total Annual Plant Usage, %
2000	15,041	15,265	-224	-1.5%
2001	14,569	14,603	-34	-0.2%
2002	15,122	14,807	315	2.1%
2003	14,513	13,693	820	6.0%
2004	13,885	12,820	1,065	8.3%
2005	14,775	13,845	930	6.7%
2006	14,851	14,025	826	5.9%
2007	13,369	12,796	573	4.5%
2008(1)	13,006	11,984	1,022	8.5%
2009	12,512	11,941	571	4.8%
2010	12,062	11,338	724	6.4%
2011	13,111	11,686	1,425	12.2%
2012	15,474	14,032	1,442	10.3%
2013	14,381	12,912	1,469	11.4%
2014	13,880	12,646	1,234	9.8%
2015	12,744	11,595	1,149	9.9%
2016	13,491	12,723	768	6.0%
2017	13,321	12,498	823	6.6%
2018	13,759	12,678	1,081	8.5%
Historical Average (Excluding 2000/2001)				**7.5%**
Historical Average (Excluding 2000/2001 and 2011 – 2013)				**6.8%**

(1)Total Well Field Pumpage does not match with prior table as listed in the 2014 Master Plan. It is unclear why, but the higher value was used in the average of Total Annual Plant Usage and the Peaking Factors developed in the next section.

3.2.3 Distribution System Usage

The data from the monthly reports was used to update the historical usage characteristics for Years 2000 through 2018 and is provided in Table 3-3. The Lincoln usage - or, the total water transmitted to the distribution system - is used to assess high service pumping requirements, as well as finished water transmission and distribution system needs. This table also provides the basis to develop design values for projecting system peaking factors (PF) for maximum day and maximum hour demand.

A review of this table shows that the design value of 2.4 in the *2014 Master Plan* has only been experienced once in the last 19 years and is overly conservative. Therefore, the design peaking factor must be updated. The update uses the projections based on a probability of exceedance once every 12 years, to be consistent with the probability of exceedance used in the *2014 Master Plan*. This projection leads to a design peaking factor for MD:AD of 2.25.

The characteristics for maximum hour peaking, even with the addition of data from Years 2013 through 2018, has not changed since the *2014 Master Plan*. A recommended design value of 4.3 remains consistent, representing a probability of exceedance once every 12 years.

Table 3-3 Historic Water Usage and Peaking Factors (PF)

Year	Total Annual Pumpage, BG	Total Annual Lincoln Usage, BG	Average Day Demand, mgd	Maximum Day Demand, mgd	Maximum Hour Demand, mgd	MD:AD PF	MH:AD PF	MH:MD PF
2000	15.0	15.3	41.8	86.0	127.5	2.1	3.0	1.5
2001	14.6	14.6	40.0	85.5	102.1	2.1	2.6	1.2
2002	15.1	14.8	40.6	90.4	136.9	2.2	3.4	1.5
2003	14.5	13.7	37.5	78.0	125.7	2.1	3.4	1.6
2004	13.9	12.8	35.1	65.8	93.3	1.9	2.7	1.4
2005	14.8	13.8	37.9	87.6	114.1	2.3	3.0	1.3
2006	14.9	14.0	38.4	75.7	117.6	2.0	3.1	1.6
2007	13.4	12.8	35.1	84.9	122.6	2.4	3.5	1.4
2008	13.0	12.0	32.8	69.1	117.7	2.1	3.6	1.7
2009	12.5	11.9	32.7	60.1	136.7	1.8	4.2	2.3
2010	12.1	11.3	31.1	70.1	133.3	2.3	4.3	1.9
2011	13.1	11.7	32.0	69.3	127.5	2.2	4.0	1.8
2012(1)	15.5	14.0	38.4	80.0	150.3	2.1	3.9	1.9
2013	14.4	12.9	35.4	72.5	140.5	2.0	4.0	1.9
2014	13.9	12.6	34.6	68.8	128.0	2.0	3.7	1.9
2015	12.7	11.6	31.8	65.0	117.0	2.0	3.7	1.8
2016	13.5	12.7	34.9	70.6	133.0	2.0	3.8	1.9
2017	13.3	12.5	34.2	63.9	122.0	1.9	3.6	1.9
2018	13.8	12.7	34.7	66.5	132.0	1.9	3.8	2.0
Historical Average	13.9	13.0	35.7	74.2	125.1	2.1	3.5	1.7
92nd Percentile All Years (1 in 12-year exceedance probability)						**2.29**	**4.09**	**1.96**
92nd Percentile Last 12-Years (1 in 12-year exceedance probability)						**2.25**	**4.30**	**2.10**

(1) The maximum hour value has changed from what was reported in the 2014 Master Plan for this year. It was found to be over-estimated (instantaneous value rather than hourly maximum). City staff provided an updated maximum hour and verified the number listed in this report.

3.3 Historical Metered Sales

3.3.1 Historical System Metered Sales

Historical metered sales data was provided for Years 2013 through 2018 for this update. This data was used to assess the mix of residential and non-residential water use and to determine typical per-capita water use rates. In addition, this data was used to update the non-revenue water characteristics. Table 3-4 summarizes the historical metered sales, including the distribution between residential and non-residential usage, as well as the resulting non-revenue water and a 10-year running average. The data shows that the percentage of residential metered sales has consistently been around 65 percent of total sales and this ratio was selected as a design value in the demand projections.

Table 3-4 Historical Metered Sales

Year	Historical Metered Sales					Average Day Lincoln Usage, mgd	Non-Revenue Water, % of AD	Non-Revenue Water 10-Year Running Average %
	Residential, mgd	Residential, %	Non-Residential, mgd	Non-Residential, %	Total			
2000	23.7	65	12.9	35	36.6	41.2	11.2	7.5
2001	21.8	63	12.7	37	34.5	39.1	11.8	8.1
2002	23.9	65	12.8	35	36.7	39.7	7.6	7.9
2003	22.3	65	11.9	35	34.2	37.5	8.8	8.0
2004	22.2	65	11.9	35	34.1	35	2.6	7.7
2005	23.9	67	11.9	33	35.8	38.5	7.0	7.8
2006	24.1	66	12.2	34	36.3	36.5	0.5	7.1
2007	21.5	65	11.7	35	33.2	35.1	5.4	7.1
2008	19.6	64	10.8	36	30.4	32.7	7.0	7.1
2009	20.8	67	10.3	33	31.1	32.7	4.9	6.7
2010	18.9	66	9.7	34	28.6	31.1	8.0	6.4
2011	20.9	67	10.5	33	31.4	32	1.9	5.4
2012	22.8	66	11.7	34	34.5	38.4	10.2	5.6
2013	20.8	67	10.5	33	31.3	35.4	11.5	5.9
2014	19.3	65	10.2	35	29.5	34.6	14.7	7.1
2015	18.6	64	10.3	36	29.0	31.8	9.0	7.3
2016	20.0	65	10.9	35	30.8	34.9	11.6	8.4
2017	19.8	65	10.6	35	30.3	34.2	11.3	9.0
2018	19.8	65	10.6	35	30.4	34.7	12.4	9.5
Average 2000-2018	**21.3**	**65**	**11.3**	**35**	**32.6**	**35.5**	**8.3**	**7.3**

3.3.1.1 Non-Revenue Water

Non-revenue water (NRW) is calculated as the difference between the average Lincoln Usage and the average sum of all metered usage over a given period. This component includes water used for flushing, firefighting, water main breaks, leakage and apparent losses (meter inaccuracies). This component was often referred to as unaccounted-for water (UFW) until 2003. In 2003, AWWA abandoned the use of the term UFW, because all volumes of water supplied go towards either beneficial consumption or water loss. All water sent into the distribution system can be accounted for. Today, the term NRW is the term favored by IWA and AWWA [*Best Practice in Water Loss Control: Improved Concepts for 21st Century Water Management, AWWA's Water Loss Control Committee*].

Previous master plans have used a percentage indicator as part of the methodology to project the non-revenue component of water. Although percentage indicators still exist in the industry, especially in projections for the non-revenue water use component, AWWA has discouraged the use of percentage indicators and will soon discontinue support of volumetric percentage performance indicators (VPPI). Upcoming materials which will soon be released by AWWA and will discontinue the use of VPPI include Version 6.0 of the Free Water Audit Software (expected to be released in 2020) and the next edition (5th) of the AWWA *M36, Water Audits and Loss Control* (expected to be released in 2021).

This update followed the same process as the *2014 Master Plan* to develop the non-revenue water component of the projections, by using percentage indicators. In order to shift away from percentage indicators in future planning efforts and transition to the methodology which will be presented in the upcoming AWWA materials, the planning study would need to include the evaluation of additional data and metrics. Additionally, the process to develop demand projections will need to be redefined starting at the population projections process. A few of these changes are listed below.

- Historical data will be needed for the average of number of service connections (both active and inactive) for each fiscal year to be evaluated.
- Authorized consumption must be categorized into billed metered, billed unmetered, unbilled metered, or unbilled unmetered.
- Projections must be developed for the number of service connections per year over for the planning horizon. This methodology differs significantly from a population-based approach to demand projections. However, population projections can still be used in the process to project other components of usage and a combined approach could be taken (i.e. population based to determine consumer usage with a non-revenue component based on projected number of service connections).
- If it is desired to evaluate and apply differing non-revenue characteristics by Service Level rather than a global value, the average number of service connections by Service Level for the historical period will need to be recorded and projections for the number of service connections must also be developed by Service Level.

As seen in Table 3-4, non-revenue water has varied significantly, ranging from approximately 2 percent (excluding Year 2006) to 15 percent. In recent years, the non-revenue running average has increased to over 9 percent. For planning purposes, a non-revenue water percentage of 9 percent was used in the demand projections.

In order to benchmark the City's non-revenue water against research provided by AWWA, it was necessary to obtain a losses per connection value using the number of service connections for the metered sales data from 2018 and the non-revenue water for that same year. This is because AWWA no longer provides benchmarks based on the volumetric percentage indicators. Table 3-5 shows the AWWA typical reported values for the current performance indicators.

Table 3-5 AWWA Performance Indicator Typical Values

INDICATORS	AWWA TYPICAL VALUE (1)
Non-Revenue Water (NRW) (MG)	-
Validation Score	60-70
Apparent Losses (gals/conn/day)	11.01
Real Losses (gals/conn/day)	66.97
Infrastructure Leakage Index (ILI)	3.13

(1) Median values as published by the AWWA Water Audit Data Initiative (WADI) 2016 dataset and The State of Water Loss Control in Drinking Water Utilities: A White Paper from the AWWA.

With the City's non-revenue water of 4.3 mgd during year 2018 and using the number of services of 85,103 from the metered sales data, the City's total non-revenue component was 50.5 gallons per service connection per day for this year. This relates to the values for the apparent losses plus the real losses shown in Table 3-5. When this comparison is made, the City's total loss per service connection of 50.5 gallons per service connection per day during 2018 is less than the sum of the apparent losses and real losses from the AWWA median values of approximately 78 gallons per service connection per day. This indicates that the City is on the lower side of water loss as it is currently quantified by AWWA.

3.3.1.2 Per-Capita Usage

Table 3-6 presents the per-capita usage characteristics (which are also illustrated in Figure 3-2). The difference between the total metered sales and the average day Lincoln usage is the non-revenue component. There has been a noticeable downward trend in per-capita usage over the period of historic data, but the downward trend appears to be flattening. Over the next decade, it is anticipated that a limit will be reached.

Table 3-6 Historical Per-Capita Usage

Year	Population	Residential Sales		Total Metered Sales		Average Day Lincoln Usage	
		Total, mgd	Per-Capita, gpcd	Total, mgd	Per-Capita, gpcd	Total, mgd	Per-Capita, gpcd
2000	225,581	23.7	105	36.6	162	41.2	183
2001	228,861	21.8	95	34.5	151	39.1	171
2002	232,141	23.9	103	36.7	158	39.7	171
2003	235,421	22.3	95	34.2	145	37.5	159
2004	238,701	22.2	93	34.1	143	35.0	147
2005	241,981	23.9	99	35.8	148	38.5	159
2006	245,261	24.1	98	36.3	148	36.5	149
2007	248,541	21.5	87	33.2	134	35.1	141
2008	251,821	19.6	78	30.4	121	32.7	130
2009	255,101	20.8	82	31.1	122	32.7	128
2010	258,379	18.9	73	28.6	111	31.1	120
2011	261,480	20.9	80	31.4	120	32.0	122
2012	264,618	22.8	86	34.5	130	38.4	145
2013	267,948	20.8	78	31.3	117	35.4	132
2014	271,216	19.3	71	29.5	109	34.6	128
2015	274,524	18.6	68	29.0	105	31.8	116
2016	277,872	20.0	72	30.8	111	34.9	126
2017	281,261	19.8	70	30.3	108	34.2	122
2018	284,691	20.0	69	30.4	107	34.7	122
Average 2000-2018	**255,021**	**21.3**	**84**	**32.6**	**129**	**35.5**	**141**

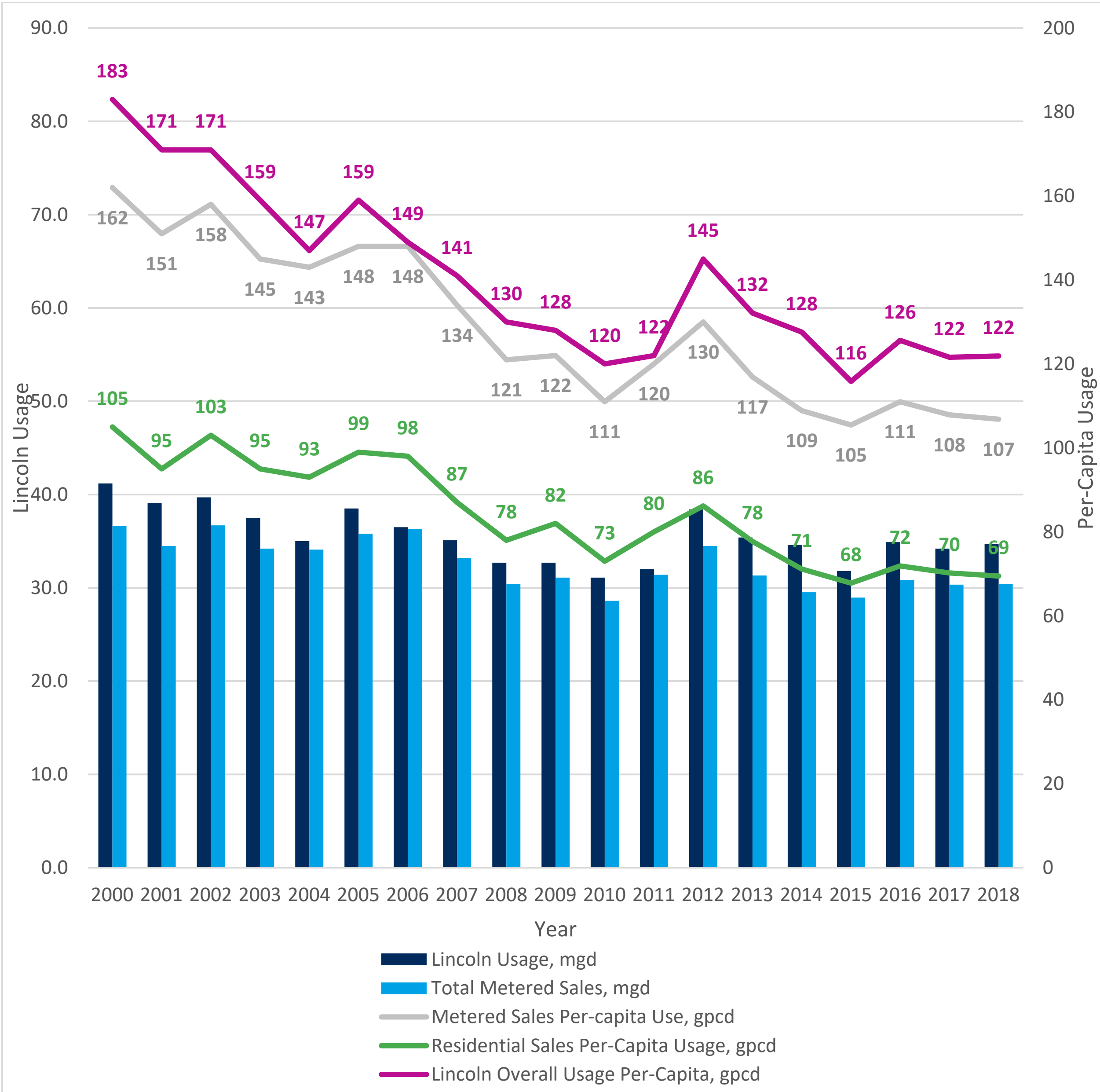

Figure 3-2 Historical Water Use and Per-Capita Usage

3.3.2 Historic Metered Sales by Service Level

The City provided Year 2018 metered data for all billing cycles for all accounts. Each meter was geocoded or matched to its location within the distribution system and assigned within the respective Service Level and TAZ geography. The data was pulled into a Power BI dashboard which allows for rapid filtering, slicing, and review of individual characteristics at the Service Level and even down to the TAZ geographies. In addition, the metered sales data was disaggregated for the entire year to develop the daily average metered usage curve by Service Level by Class. A total of 99 percent of the metered sales by usage were assigned a spatial location match by either geocoding, address matching, or linked to the service account for which there was a spatial location.

Figure 3-3 shows the metered sales breakdown by Service Level by Class for 2018 metered data. As noted, only 1-percent of the metered sales was not assigned to a metered location and this shows up with the "(Blank)" category. This figure presents a variety of information developed from the breakdown of metered sales by TAZ, by TAZ and Service Level, and the overall density of usage in terms of usage per acre for each TAZ which is described below:

- The map and graph on the left side of the dashboard shows the usage over the year within each Service Level with the map identifying the Service Levels.
- The data in the center of the dashboard presents the overall ratio of usage by type (Residential, Non-residential, and HUSER), the overall per-capita usage by Service Level and the percent of that use which is residential, and at the bottom shows the percentage of metered use by each Service Level.
- The map on the right shows the density of usage for each TAZ in gallons per acre per day.

A comparison of the per-capita usage characteristics obtained in the last two Master Plans against the 2018 metered data evaluation for per-capita usage characteristics is shown in Table 3-7. This table shows a declining per-capita usage in all Service Levels when compared against previous year per-capita usage data.

Table 3-7 Comparison of Per-capita usage derived from Metered Sales Breakdown

Service Level	Per-Capita Usage by Service Level		
	2006 Metered Sales	2012 Metered Sales	2018 Metered Sales
Northwest	170	130	106
Belmont	101	78	75
Low	67	60	57
High	92	90	65
Southeast	151	125	88
Cheney	211	175	113
Overall	64	86	70

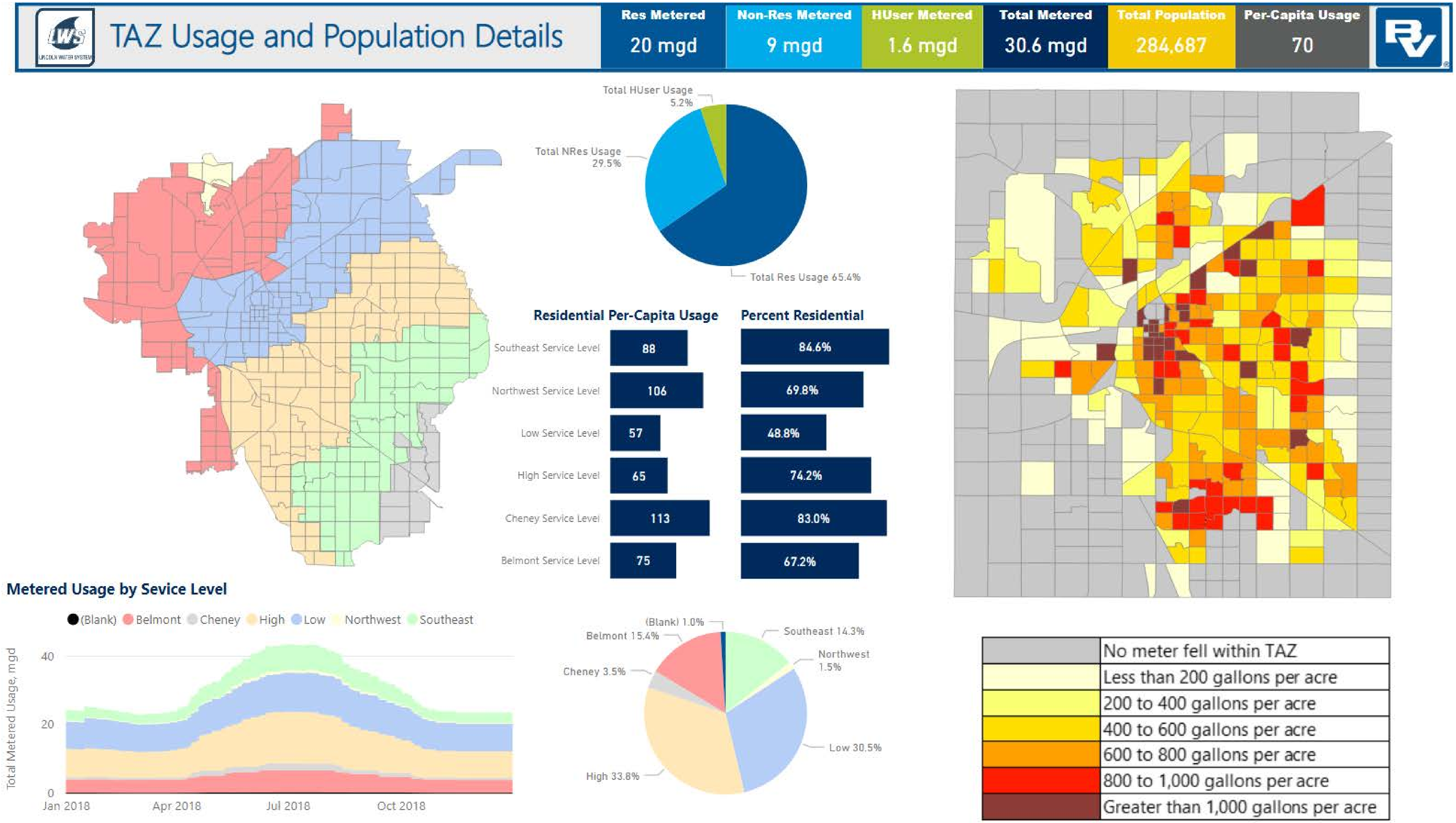

Figure 3-3 2018 Metered Usage Characteristics by Service Level

3.4 Water Demand Projections

Using the information presented in the previous sections, water demands were developed for the planning horizon for Years 2020, 2025, 2040, and 2060. Water demands for interim years (such as Year 2032, the 12-year modeling scenario) are interpolated between these planning years. The demand projections are based on the population forecasts, the residential per-capita usage, percentage residential usage, non-revenue water, and peaking factors.

There are two factors in the approach that are slightly different than the Previous Master Plan and include the evaluation of additional large use point loads north of I-80 that could represent future industrial or large user demands, and the adjustment of the seasonal 90-day peak well field pumpage based on climate data.

3.4.1 Large Use/Industrial Demands

Demands representing potential large use customers were added to the total average day, maximum day, and maximum hour projections. Table 3-8 provides the additional demands that are added to the projections based on the potential for large use customers to develop north of I-80.

Table 3-8 Potential Future Large Use Demands

Demand Condition	2020	2025 and Beyond
Average Day Demand	1.75 mgd or 25% of 7.0 mgd	7.0 mgd
Maximum Day Demand	2.7 mgd or 25% of 10.8	10.8 mgd
Maximum Hour Demand	2.7 mgd or 25% of 10.8	10.8 mgd
Seasonal Peak Demand	2.7 mgd or 25% of 10.8	10.8 mgd

3.4.2 Climate Change Impact

Evaluations for climate change were also performed during this Master Plan update. The full details of the climate change evaluation can be found in *Appendix A – Climate Change Assessment. T*he following is a high-level summary of the details found in this assessment:

- Temperatures are expected to increase for all seasons by 4 to 5 degrees (about 5 to 10 percent) for the mid-century time-frame (Years 2041 to 2070).
- Precipitation will increase in winter, spring, and fall by 15 to 20 percent for the mid-century time-frame.
- Precipitation will decrease for the summer by 15 to 20 percent for the mid-century time-frame.

In addition to the climate change assessment, historic peaking factors for the SP:AD were reviewed against the Palmer Modified Drought Severity Index (PMDSI) for the last 20 years of data to support increasing the seasonal peak design peaking factors based on climate change. The PMDSI is a relative scale that indicates how wet/mild or hot/dry conditions were during a specific time period by using temperature and precipitation. The lower the number (negatives) indicates a hotter-drier year and the high the number (increasing positives) indicates that the year was wetter and milder. Figure 3-4 shows a scatter plot of the SP:AD peaking factors vs. the PMDSI value of the last 18 years, for a month prior to and the 3 months of the seasonal peak 90-day demand during that year. Although more of a qualitative analysis, the general trend shows that seasonal peaking factor does increase with hotter-drier years as would be expected. With climate change, there is a higher probability that any given year will experience PMDSI values on the negative end of the scale, and could even be in the negative four to five range. Extrapolation of the trend line to these values lead to a SP:AD of almost 1.7, which is almost 10-percent higher than the design value of 1.56 shown in a previous section.

Based on the climate change model assessment, and the PMDSI vs. SP:AD evaluation, the SP:AD peaking factors were increased to account for climate change. Adjustments were made to the peaking factor to increase it by 8.5 percent through Year 2040, and then an additional 2.5 percent beyond Year 2040. The SP:AD peaking factors begin at 1.56 for the Base Year (2020), increase to 1.71 by Year 2040, and then increase again to 1.76 by Year 2060.

Maximum day and maximum hour peaking factors were not increased based on the climate data for this update. The reason is that it is uncertain how climate change will impact daily maximum usage. What is certain is that in any given year there will likely be more hot and dry periods leading to more hot and dry days during the 90-day seasonal peak demands.

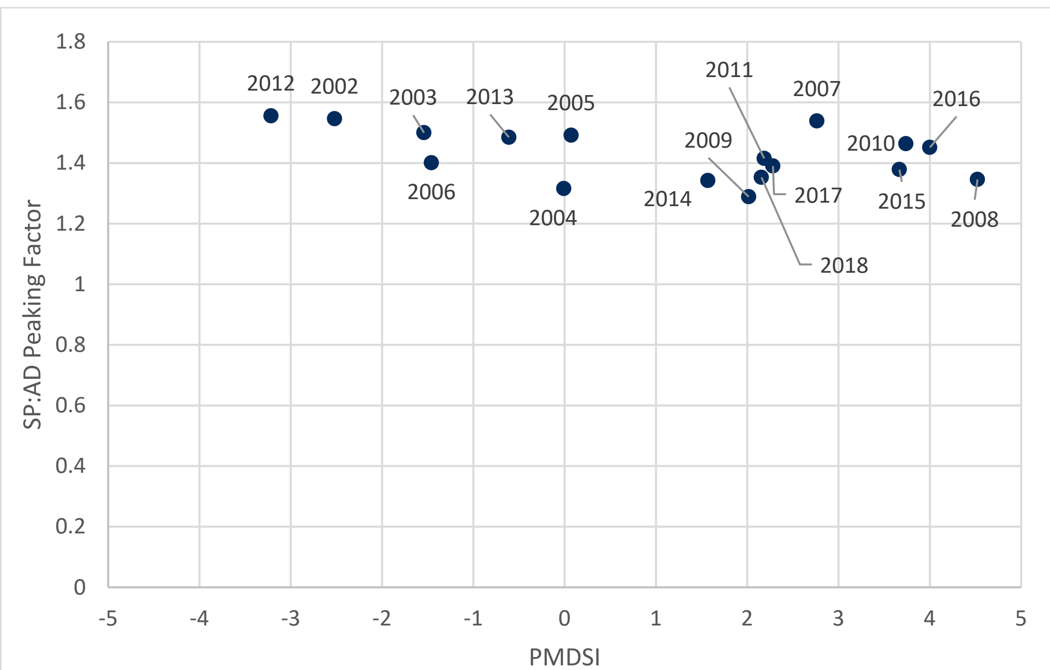

Figure 3-4 SP:AD Peaking Factors vs. PMDSI

3.4.3 Average Day Demand Projections

Design criteria for the average day demand projections is provided in Table 3-9 with a comparison to the previous Master Plans' design criteria. Design peaking factors are provided in Table 3-10.

Table 3-9 Average Day Demand Design Criteria

Description	2007 Master Plan	2014 Master Plan	2020 Master Plan Update
Per-capita Residential Metered Sales (gpcd)	96	90	85 trending down to 75 by 2032
Residential Sales as Percent of Total Metered Sales	65%	65%	65%
Per-capita Total Metered Sales (gpcd)	148	138	130 trending down to 120 by 2032
Non-Revenue Water (percent of Lincoln Usage)	6.25%	6.7%	9%
Total Lincoln Usage as Per-capita Usage (gpcd)	157	148	142 trending down to 125 by 2032
Plant Usage (%)	3%	6.9%	6.8%
Plant Usage (gpcd)	5	10	10
Well field Pumpage (gpcd)	162	158	152 trending down to 135 by 2032

Table 3-10 Maximum Day, Maximum Hour, and Seasonal Peak 90-day Peaking Factors

Description	2007 Master Plan	2014 Master Plan	2020 Master Plan Update
MD:AD Peaking factor	2.7	2.4	2.25
MH:AD Peaking Factor	4.4	4.3	4.3
SP:AD Peaking Factor	-	1.56 consistent through planning horizon	1.56 trending up to 1.76 by 2060
Maximum Day Demand (gpcd) (Lincoln Usage)	437	379	320
Maximum Hour Demand (gpcd) (Lincoln Usage)	693	636	615

Table 3-11 provides a summary of the updated demand projections based on the planning criteria, which are shown graphically in Figure 3-5.

Table 3-11 Future Demand Projections

Year	Estimated Population	Average Day Well Field Pumpage, mgd	Average Day Lincoln Usage, mgd	Maximum Day Well Field Pumpage, mgd	Maximum Day Lincoln Usage, mgd	Maximum Hour Lincoln Usage, mgd	Seasonal Peak 90-Day DemanD, mgd
2020 (Base year)	291,677	45.9	41.0	102.0	95.0	179.2	71.7
2025	309,902	47.1	40.1	108.3	101.0	183.1	79.7
2030	329,266	48.3	41.3	111.4	103.8	188.6	83.4
2032 (12-year CIP)	337,496	49.3	42.3	113.6	105.9	192.5	85.7
2040	371,700	53.4	46.4	123.5	115.1	210.1	95.7
2050	418,281	59.2	52.2	137.5	128.2	235.1	107.7
2060	470,700	65.7	58.7	153.3	142.9	263.2	121.5

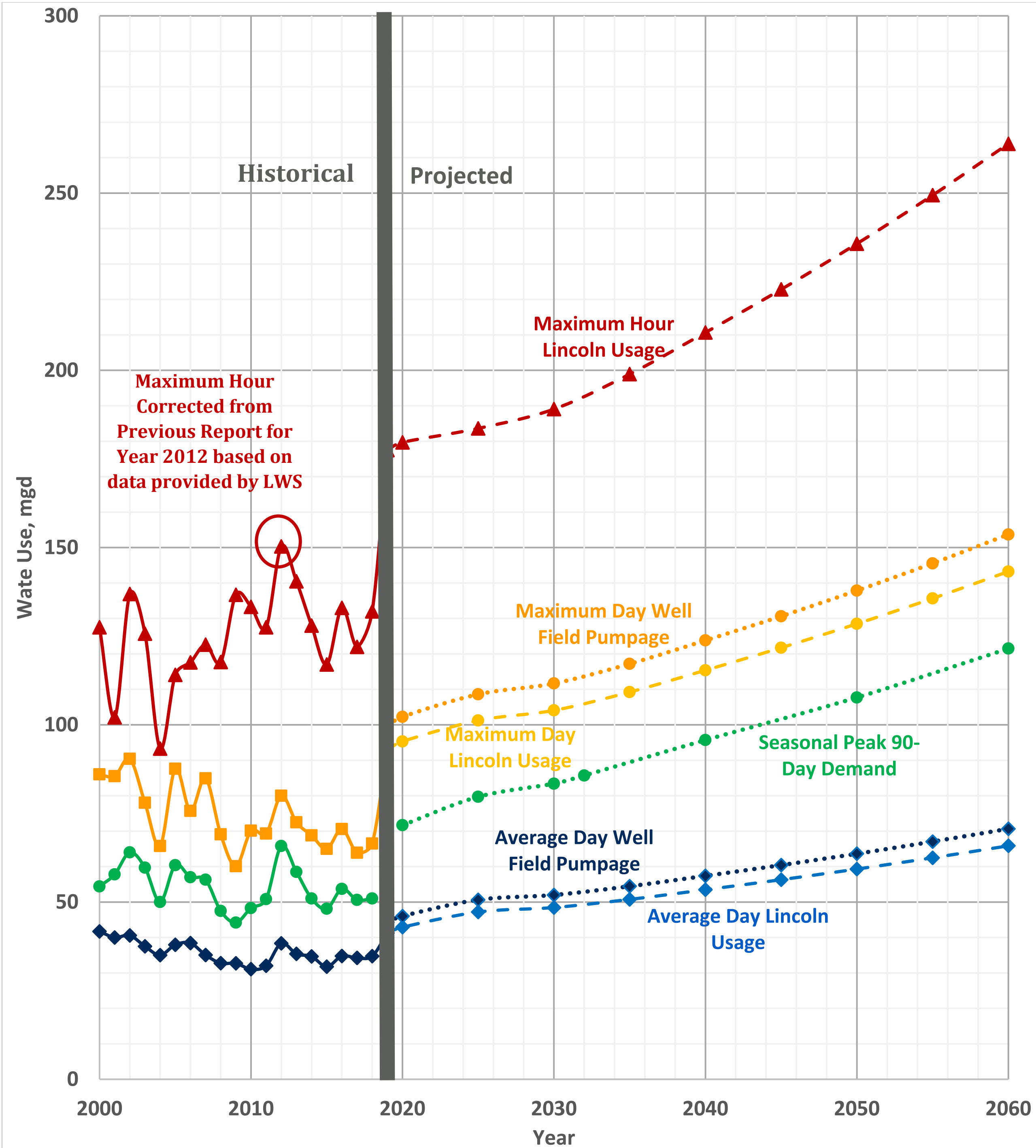

Figure 3-5 Future Demand Projections and Well Field Pumpage Requirements

3.5 Water Usage Projections by Service Level

Based on the total system projections and the data developed by Service Level detailed in a previous section, water demands were allocated to the Service Level basis. For each of the planning years, a balance between the overall design values, and the sum of the components of the Service Level demands was achieved by slight adjustments to individual service level characteristics based on the historical data. Table 3-12 presents the demand characteristics for average day by Service Level by class. Figure 3-6 shows the demands by Service Level graphically.

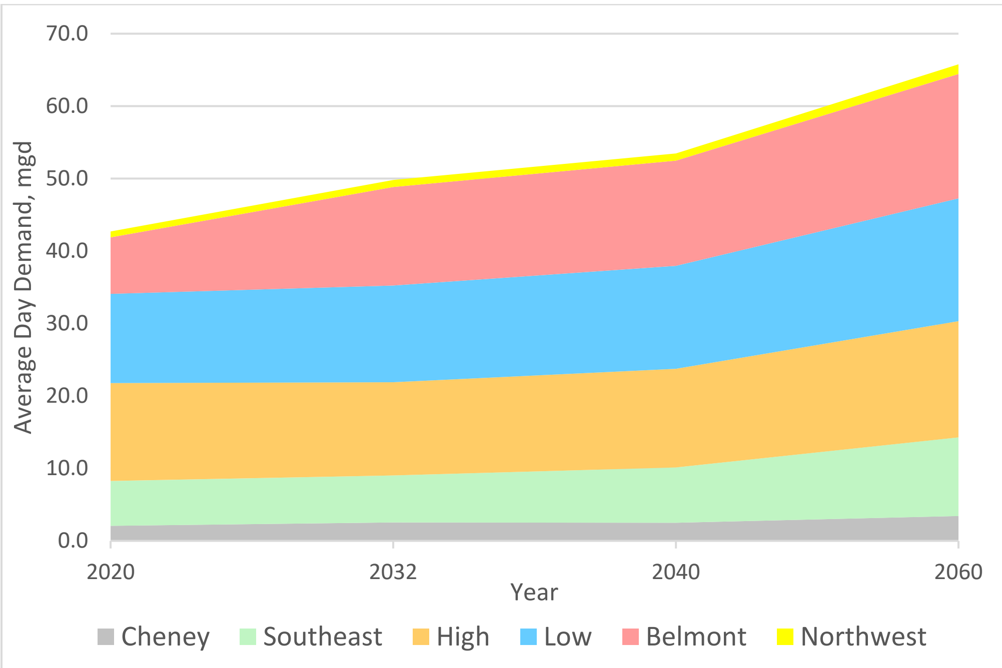

Figure 3-6 Average Day Demand by Service Level

Table 3-12 Average Day Projections by Service Level by Class

Service Level	Residential Per-capita Sales, gpcd	Residential Sales, mgd	Residential/ Total Sales, %	Non-Residential Sales, mgd	Total Sales, mgd	Non-revenue Water, %	Non-revenue Water, mgd	Average day usage, mgd
Base Year (2020)								
Northwest	130	0.5	65%	0.3	0.7	9%	0.07	0.8
Belmont	85	3.6	65%	1.9	5.5	9%	0.54	6.1
Low	70	5.6	50%	5.6	11.2	9%	1.11	12.3
High	80	8.9	72%	3.4	12.3	9%	1.22	13.5
Southeast	110	4.8	85%	0.8	5.6	9%	0.56	6.2
Cheney	125	1.5	80%	0.4	1.9	9%	0.19	2.1
Total	85	24.8	65%	12.5	37.3	9%	3.7	41.0
2032 (12-yr CIP)								
Northwest	110	0.6	65%	0.3	0.9	9%	0.09	1.0
Belmont	75	3.9	65%	2.1	6.0	9%	0.59	6.6
Low	70	6.1	50%	6.1	12.2	9%	1.20	13.4
High	70	8.4	72%	3.3	11.7	9%	1.16	12.9
Southeast	90	5.0	85%	0.9	5.9	9%	0.58	6.5
Cheney	100	1.9	80%	0.5	2.3	9%	0.23	2.6
Total	75	25.8	65%	13.1	39.0	9%	3.9	42.8
2040								
Northwest	100	0.6	65%	0.3	0.9	9%	0.09	1.0
Belmont	75	4.4	65%	2.4	6.8	9%	0.68	7.5
Low	70	6.5	50%	6.5	12.9	9%	1.28	14.2
High	70	8.9	72%	3.5	12.4	9%	1.22	13.6
Southeast	85	5.6	80%	1.4	7.0	9%	0.69	7.6
Cheney	85	1.8	80%	0.5	2.3	9%	0.22	2.5
Total	75	27.8	65%	14.5	42.3	9%	4.2	46.5
2060								
Northwest	100	0.8	65%	0.4	1.2	9%	0.12	1.3
Belmont	75	6.0	65%	3.2	9.2	9%	0.91	10.2
Low	70	7.7	50%	7.7	15.4	9%	1.53	17.0
High	70	10.5	72%	4.1	14.6	9%	1.44	16.0
Southeast	85	7.9	80%	2.0	9.9	9%	0.98	10.9
Cheney	85	2.5	80%	0.6	3.1	9%	0.31	3.4
Total	75	35.4	65%	18.1	53.5	9%	5.3	58.8

Updates were also made to individual Service Level peaking characteristics by class. Years 2014 through 2018 were used to benchmark the typical peaking factors seen by Service Level for MD:AD. Year 2018 SCADA data was used to develop the typical MH:AD peaking factors, specific to service level. Slight adjustments were made to the MD:AD and MH:AD peaking factors by Service Level through the planning horizon to ensure that the sum of the individual Service Levels matches the overall system demands. Table 3-13 provides the individual peaking characteristics by Service Level by Class. Design peaking factors are shown by a range because it was necessary to make slight adjustments in some years on the peaking factors to match the overall MD and MH demands for the system.

Table 3-13 Planning Peaking Factors by Class and Service Level

Service Level	2014-2018 Average MD:AD	Overall Design MD:AD	2018 MH:AD[1]	Overall Design MH:AD
Northwest	3.1	3.0 – 3.1	6.6	6.0 – 6.1
Belmont	1.8	1.8 – 2.1	3.1	2.8 – 3.0
Low	2.1	2.1 – 2.3	3.2	2.9 – 3.1
High	2.5	2.5 – 2.6	5.4	4.9 – 5.2
Southeast	2.7	2.7 – 2.8	7.0	6.5 – 6.6
Cheney	2.5	2.5 – 2.6	7.0	6.5 – 6.6
System Overall	**2**	**2.25**	**3.7**	**4.3**

[1]Obtained from SCADA data provided for 2018

The maximum day and maximum hour demands by Service Level were calculated by applying the MD:AD and MH:AD peaking factors by Service Level to their respective average day demands. Table 3-14 presents the average day, maximum day, and maximum hour demands by Service Level.

Table 3-14 Projected Water Requirements by Service Level

Service Level	Average Day, mgd	Maximum Day, mgd	Maximum Hour, mgd
Base Year (2020)			
Northwest	0.8	2.5	5.0
Belmont	6.1	10.9	16.9
Low	12.3	25.9	35.7
High	13.5	33.8	66.2
Southeast	6.2	16.7	40.3
Cheney	2.1	5.2	13.5
Total	41.0	95.0	177.6
Year 2032 (12-yr CIP)			
Northwest	1.0	3.0	5.9
Belmont	6.6	13.9	19.8
Low	13.4	30.7	40.1
High	12.9	33.4	66.9
Southeast	6.5	18.1	42.7
Cheney	2.6	6.6	16.8
Total	42.8	105.8	192.2
2040			
Northwest	1.1	3.1	6.1
Belmont	7.5	16.1	22.6
Low	14.2	32.7	44.1
High	13.6	35.3	70.7
Southeast	7.6	21.4	50.4
Cheney	2.5	6.5	16.5
Total	46.6	115.1	210.3
2060			
Northwest	1.3	4.0	8.0
Belmont	10.2	22.1	29.5
Low	17.0	37.3	49.2
High	16.0	40.1	81.8
Southeast	10.9	30.4	71.6
Cheney	3.4	8.9	22.7
Total	58.8	142.8	262.7

4.0 Supply

The only significant improvement/change to Lincoln's water supply since the completion of the *2014 Master Plan* is the completion of Horizontal Collector Well 14-2, which increases pumping capacity for the collector well system by 17.5 mgd. Changes to the well field yield are discussed later in this chapter and in *Appendix B – 2019 Lincoln Well Field Groundwater Modeling*. Therefore, the primary points of focus under this master plan update include:

- An updated assessment of Lincoln's Water Rights.
- Updated Platte River flows as impacted by groundwater development and climate change.
- An update to the groundwater model to determine summer pumping rates for existing conditions and expansion of the wellfield to include two future horizontal collector wells.
- Recommended staging for future collector wells based upon projected demands.

4.1 Water Rights

4.1.1 Nebraska's Water Supply

Nebraska has over 24,000 miles of rivers and streams and nearly 2 billion acre-feet of useable groundwater in the High Plains aquifer beneath the state. Though water is abundant, problems arise due to water availability issues and/or difficult locations. Nebraskans have addressed this by developing nearly 3,000,000 acre-feet of storage in reservoirs (primarily used for irrigation) to retime the surface water supply, and have developed over 8 million irrigated acres for crop production. Most of these irrigated acres rely on groundwater pumping.

Nebraska Department of Natural Resources (NeDNR) developed the Integrated Network of Scientific Information and GeoHydrologic Tools, or INSIGHT, to help water managers across the state understand the dynamic nature of Nebraska's water supply. INSIGHT provides an annual snapshot of the state's water supply, water demands, nature and extent of use, and overall water balance. This information can be analyzed at a statewide level, a basin-wide level, or at a sub-basin level.

Figure 4-1 through Figure 4-3 provide examples of what INSIGHT shows.

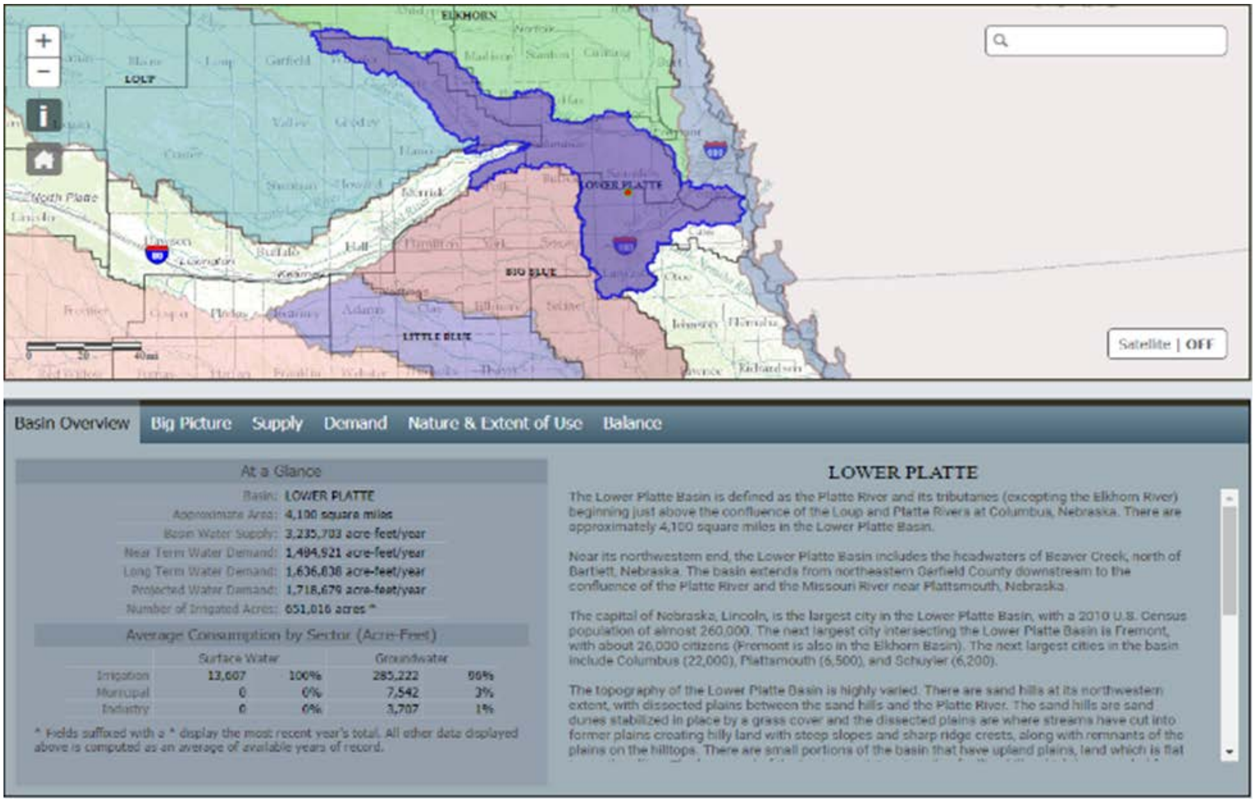

Figure 4-1 **The Lower Platte Basin Overview Screen in INSIGHT Gives A Summary Of The Basin's Characteristics and Projected Water Demands**

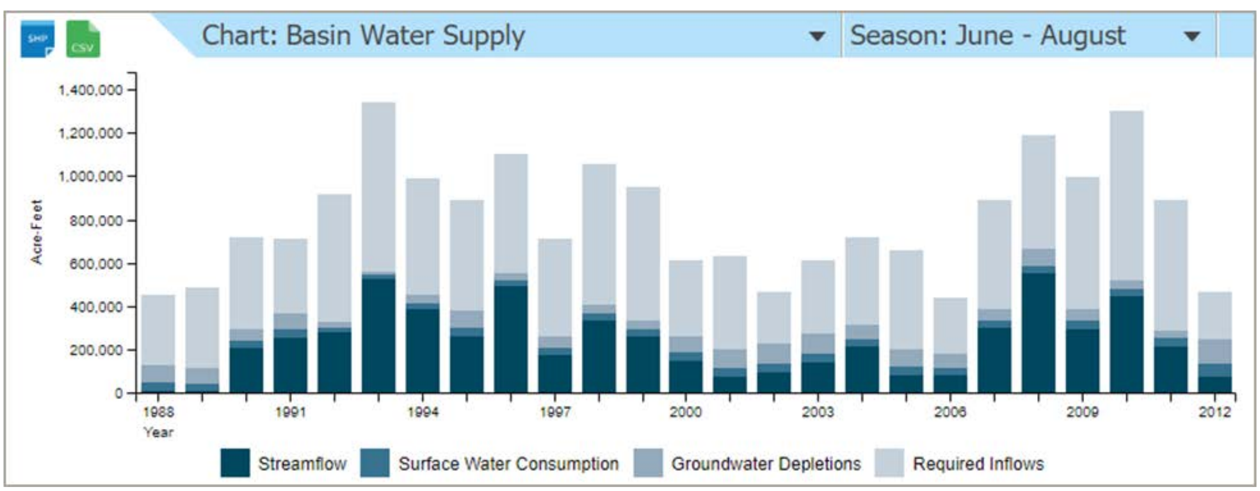

Figure 4-2 **The Lower Platte Basin Annual Water Supply for June-August in INSIGHT**

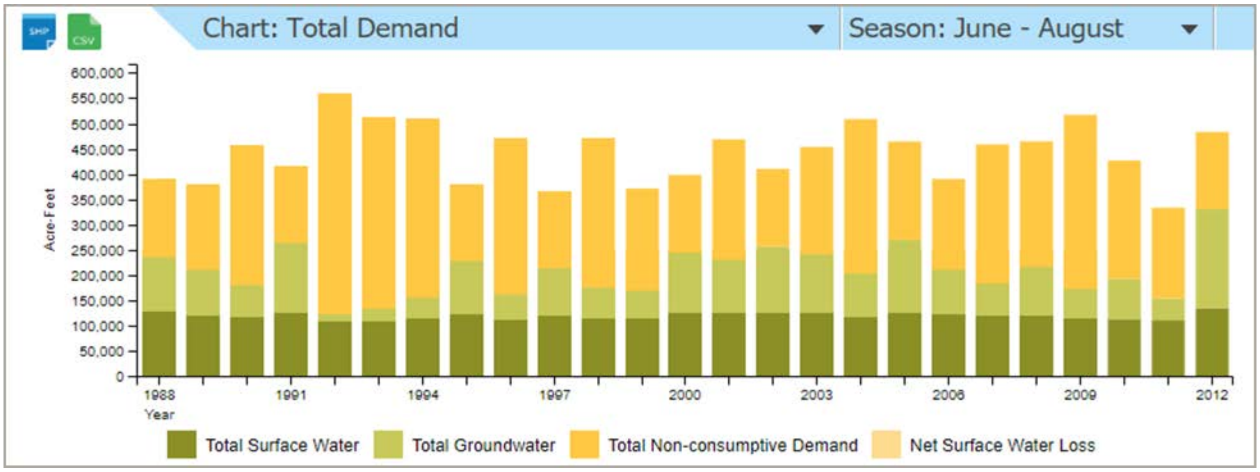

Figure 4-3 **Annual Total Demand Estimates for the Lower Platte Basin for the June- August Season in INSIGHT**

The information and data available from NeDNR and other agencies - including the U.S. Geological Survey, the U.S. Bureau of Reclamation, local natural resources districts, and other water users - can be combined with the state-wide regional groundwater models. This combined information can be used to evaluate the impacts of historic groundwater pumping and future projected groundwater pumping on aquifer levels and surface water flows. These future scenario model runs can also incorporate the effects of a changing climate on water supplies, which will be done as part of the Master Plan update.

4.1.2 Nebraska Water Law

The State of Nebraska has a bifurcated system of water laws that regulate the use of surface water and groundwater differently. The surface water system operates under the prior appropriation doctrine, often referred to as a system of "first in time, first in right." Surface water appropriations are administered at the state level by the NeDNR. Every person that uses surface water must have a valid appropriation. Under this system, appropriators that have senior priority dates (older dates) are entitled to their quantity of water before more junior appropriators (newer dates) get their quantity of water.

Groundwater, however, operates under a modified correlative rights system. This means that in times of shortages, groundwater users will share the remaining groundwater supply. Groundwater in Nebraska is regulated by the 23 natural resources districts.

4.1.3 Existing Surface Water Rights Held by the City of Lincoln for the Ashland Wellfield

Nebraska law allows a public water supplier to make an application to appropriate waters for induced ground water recharge (Neb.Rev.Stat.46-233). The City of Lincoln holds 5 induced groundwater recharge appropriation permits (A-17312A, A-17312B, A-17312C, A-17312D, and A-17312E) for the Ashland wellfield. The amount of the appropriation is limited to 704 cubic feet per second (cfs) in the summer season and 200 cfs in all other seasons. The priority dates associated with streamflow are tied to a particular well series and range in dates from January 21, 1964, to January 1, 1993. The permits are administered by NeDNR in the same manner as other surface water appropriations. When streamflow in the Platte River is reduced to 704 cfs in the summer season or 200 cfs outside of the summer season, the City of Lincoln may request NeDNR to administer all junior surface water appropriations upstream of the Ashland wellfield until Platte

River flows again exceed either 704 cfs or 200 cfs. If requested by the city, NeDNR may also approve the transfer of priority dates among water wells within the wellfield under this permit - including replacement water wells - to improve the wellfield's efficiency of operation with respect to river flow, provided that certain conditions are met.

There are many water rights upstream of the city's wellfield that could be affected by a call for water administration. Figure 4-4 is a map of all storage permits upstream of the city's wellfield that are junior to the city's water right. The total amount of storage currently authorized under these water rights is just under 100,000 acre-feet. The owners of these facilities would not be required to release water stored prior to a call for water administration. However, if any of these facilities are otherwise filling up with water flowing in from upstream, they could be required to discharge those inflows downstream during a period of water administration for the City of Lincoln.

Figure 4-5 and Table 4-1 are a depiction of the permits related to active diversion or withdrawal of streamflow. Table 4-1 presents totals for all permits and for those permits that are senior to the Nebraska Game and Parks (NGPC) in-stream flow permit, as water users junior to the NGPCs in-stream flow permit would likely be shut off before the city could exercise their water right. The NGPC has an instream flow right for 1800 cfs (as measured at the North Bend stream gage) for the reach of the Platte River from the mouth of the Loup Power Canal Return to the mouth of the Elkhorn River, and an instream flow right for 3300 cfs (as measured at the Louisville stream gage) for the reach of the Platte River from the mouth of the Elkhorn River to the confluence with the Missouri River. The NGPC has historically requested administration of its water right on an annual basis.

Table 4-1 presents the diversion point, the number of permits by diversion point, and the total diversion amount for any appropriations that are junior to the city's water right. Pumps refers to multiple locations where water is directly pumped from a stream.

The vast majority of these are for irrigation, but a few are designated for manufacturing use. The Kent Canal is technically classified as an irrigation right, but it does not typically provide irrigation water directly to water users. Instead, it is used in spring and early summer to assist with filling Davis Creek Reservoir[1]. It typically diverts much less water than what is technically allowed under its water right. For the remaining canals, these water rights are not generally representative of typical withdrawals as this table is only documenting the water rights junior to the city's permit. These canals typically have other more senior and more substantial water rights that would not be affected by water administration for the city. However, the listed amounts are junior uses that could be curtailed, meaning these canals would have to cut back on the amount diverted. The diversion category of Pumps represents many water rights that are utilized by pumping water directly out of a stream. This category represents most of the water use that is likely to be occurring during any call for water administration during the summer months.

[1] Personal Communication, 7/17/2019, T. Klanecky, Nebraska Department of Natural Resources

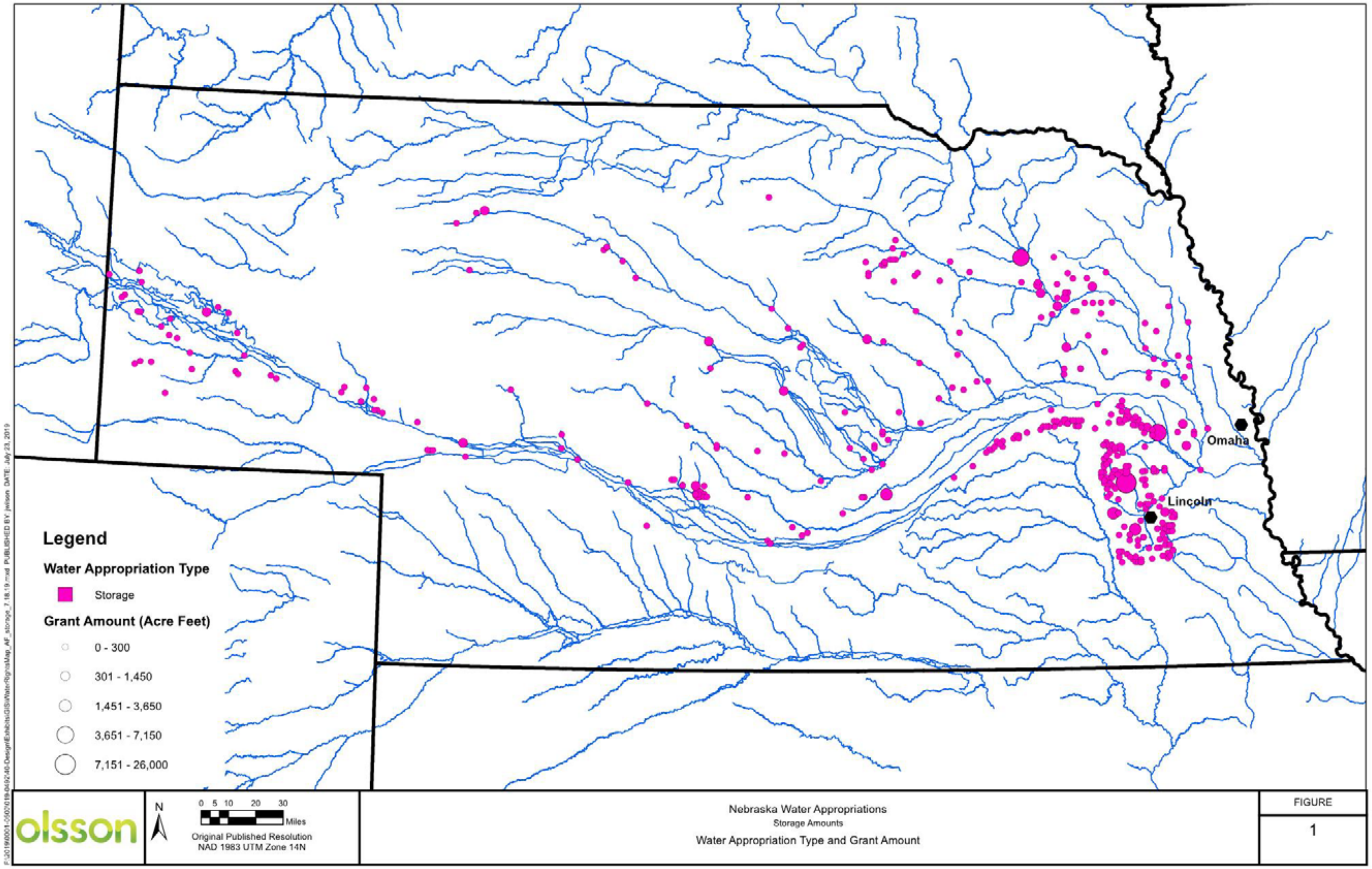

Figure 4-4 Storage Permits Upstream of the City's Wellfield that are Junior to the City's Water Right

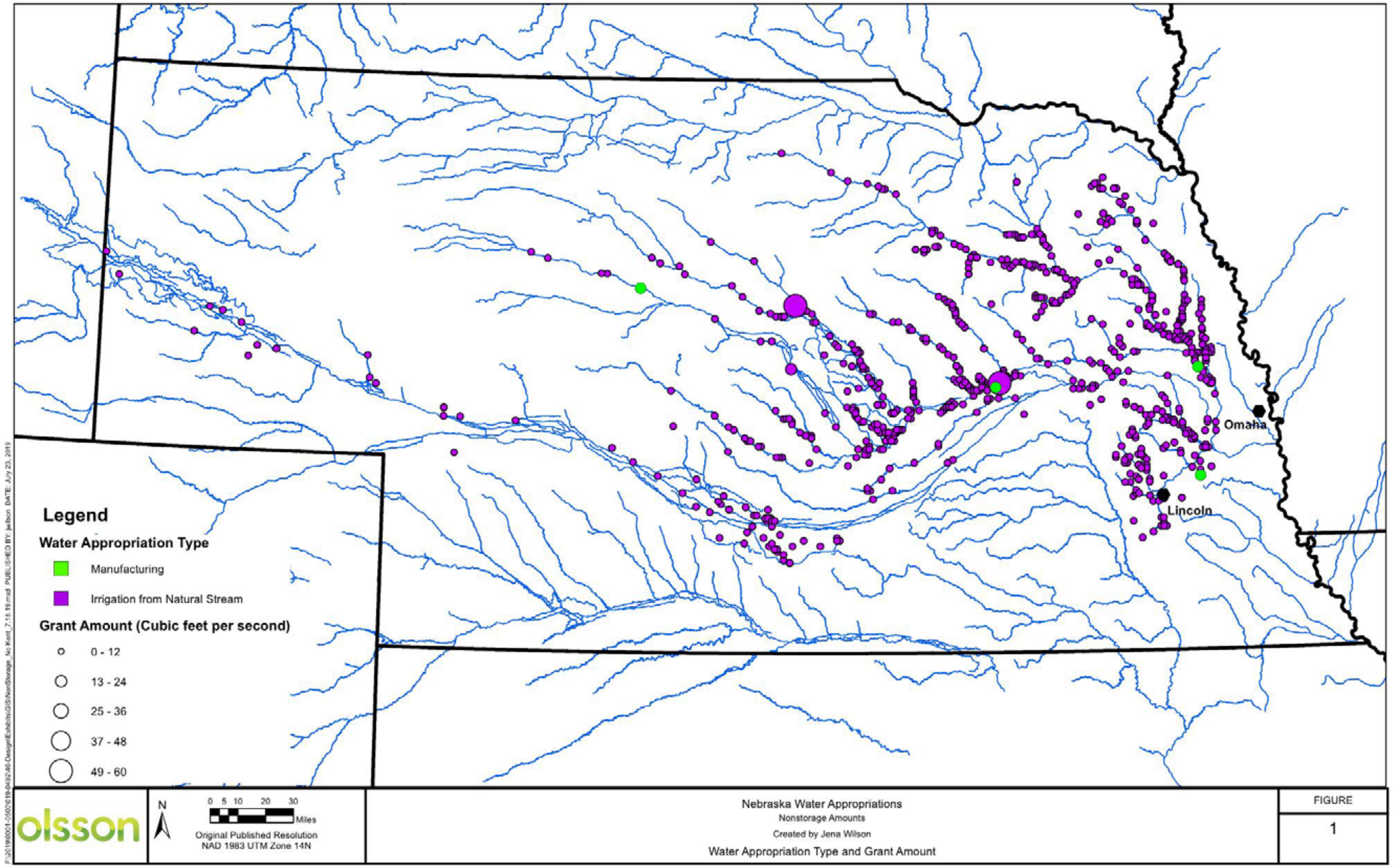

Figure 4-5 Permits to Divert Surface Water Upstream of the City's Wellfield that are Junior to the City's Water Right

Table 4-1 Junior Surface Water Diversion Rights Upstream of the City's Wellfield

Diversion	All Water Rights		Water Rights Senior to the NGPC In-stream Flow Right	
	Number of Permits	Total Diversion Rate (cfs)	Number of Permits	Total Diversion Rate (cfs)
Blue Creek Canal	2	0.24	2	0.24
Burwell-Sumter Canal	15	11.23	14	9.12
Canal No 1	1	0.73		
Canal No. 1 and 2	1	1.09	1	1.09
Canal No. 3	1	0.58		
Canal No. 3 and 4	6	5.29	6	5.29
Columbus-Genoa Canal	73	58.57	47	38.05
Cozad Canal	1	0.50	1	0.5
Dawson County Canal	1	0.91	1	0.91
Farwell Main Canal	5	20.69	4	11.02
Gothenburg Canal	3	3.78	3	3.78
Kearney Canal	1	4.86	1	4.86
Kelly Headgate	1	4.76	1	4.76
Kent Canal(1)	3	787.37	1	783.87
Mirdan Canal	5	50.10	2	45.21
O'Neal Canal	1	2.22	1	2.22
Ord-North Loup Canal	12	6.89	10	6.54
Pumps	937	1,152.42	774	906.53
Sargent Canal	6	10.83	4	4.29
Sutherland Canal	1	2.43	1	2.43
Taylor-Ord Canal	11	4.08	10	3.41
Total	**1,087**	**2,129.57**	**884**	**1,834.12**
Total without Kent Canal	**1,084**	**1,342.20**	**883**	**1,050.25**

(1)The Kent Canal is a feeder canal used to help fill Davis Creek Reservoir in the Spring and early summer. Therefore, it may not be in use during any requested water administration.

These water rights are classified as active by the NeDNR, and they continuously strive to maintain an accurate record of active water rights by investigating and canceling any water rights that have fallen into a state of nonuse. However, it is not possible to state directly from this data exactly how much actual water use would be curtailed during a call for water administration by the city. Periods of shortage of streamflows at the city's wellfield are likely to coincide with a high level of utilization of these junior water rights, and the total amount authorized for use exceeds the city's demand by a significant amount. Therefore, a proactive request for water administration would have a high degree of probability of maintaining the level of streamflow in the Platte that is required for stable operation of water withdrawals for use by the City of Lincoln.

The city should be aware that any non-domestic use of water that occurred in the city during a call for water administration would require compensation be provided to junior users. The Director of NeDNR is required to determine the amount of non-domestic use that does occur during a period of water administration.

A municipal groundwater transfer permit is a permit that a municipality may avail itself of under the Municipal and Rural Domestic Ground Water Transfers Permit Act (Neb.Rev.Stat.46-639), but it is not required. The City of Lincoln holds two municipal groundwater transfer permits for the Ashland Wellfield that total 110 million gallons/day (mgd). These transfer permits are A-10367 (with a priority date of June 15, 1931 for 60 mgd) and A-16917 (with a priority date of January 25, 1990 for 50 mgd). The city also holds a transfer permit for the Antelope Creek wellfield, but water has not been used under that permit for more than five years. The intent of the permit is to recognize continued withdrawals and to protect the source of the water supply for municipalities. If projected water demands for the city exceed 110 mgd, and there are planned expansions to treat and transport this additional demand, the city may apply for another municipal groundwater transfer permit. However, the total of 110 mgd should not be viewed as a limitation on use, it simply represents the amount of use that is protected under the existing permits.

Nebraska law (Neb.Rev.Stat.46-235.03) empowers natural resources districts with the authority to impose restrictions and controls on public water suppliers as specified in the Nebraska Groundwater Management and Protection Act. Such restrictions or controls may limit the withdrawal of groundwater to a greater degree or extent than is otherwise permitted or allowed by a permit issued by NeDNR. If the lower Platte River basin is ever declared fully appropriated under the Act, Nebraska law (Neb.Rev.Stat.46-740) would allow the integrated management plan developed pursuant to this designation to impose controls on the city's water use. Subsection 4 of this provision states in part that:

> *On and after January 1, 2026, the base amount for an annual allocation to a municipality shall be determined as the greater of either (a) the amount of water authorized by a permit issued pursuant to the Municipal and Rural Domestic Ground Water Transfers Permit Act or (b) the greatest annual use prior to January 1, 2026*

In order to avoid being subjected to an alternative annual allocation, the city should ensure that the total amount of water projected to be withdrawn, transported, and used is covered under a municipal groundwater transfer permit if the lower Platte River basin is determined to be fully appropriated in the future.

4.2 Future Streamflow Evaluation

The Lincoln wellfield is heavily dependent on Platte River streamflows that recharge the alluvial aquifer from which water is withdrawn. During periods of normal and high streamflows, the aquifer receives plenty of recharge and the wellfield is easily able to meet demands. However, during periods of lower streamflows, it is possible for withdrawals to begin to exceed the rate at which water is recharged from the stream to the aquifer. The single greatest threat to the wellfield's water supply are extended periods of low river flows, such as those that occurred in early Year 2000 and again in Year 2012.

The availability of streamflows during low-flow (or baseflow) conditions are affected by the amount of aquifer recharge and the resulting streamflows that occur upstream. These streamflows are coming from the High Plains aquifer, which contains an abundant supply of water. These streamflows are impacted over years, decades, or longer by changes in the system, like increased pumping and/or recharge. In other words, low flow conditions in the Lower Platte River during any given year will be affected by what has occurred over the past several decades or more. Therefore, it is not possible to base a prediction of the occurrence and magnitude of low flows on the current season weather conditions alone. To fully understand the impact of streamflows during low-flow conditions, it is necessary to conduct long-term groundwater flow modeling simulations using regional- scale models. These groundwater flow models (described below) represent the best available science to forecast future streamflow conditions.

There now exists regional-scale models that extend over the entire area of the High Plains aquifer in Nebraska. Three models are significant to this evaluation: the Cooperative Hydrology Study (COHYST) Model, the Central Nebraska Model (CENEB), and the Lower Platte – Missouri Tributaries (LPMT) Model. The areal extent of these models is shown in Figure 4-6.

These models have been calibrated to observations of groundwater levels and streamflows over a period that roughly extends from Years 1950-2010. To better understand the potential future streamflows in the Lower Platte River, these models were set up to provide simulations of future conditions based on antecedent conditions and an assumption that water use, and climate patterns would generally mimic those observed from the late 1980's to the early 2010's. By conducting simulations that predict streamflows under these baseline conditions and potential changes to those conditions, the model can be used to provide a likely range of future conditions relative to those experienced in the recent past.

Four future modeling scenarios were completed with each of the three groundwater models, with each groundwater model providing changes in streamflows for a discrete portion of the Platte River and its tributaries upstream of the Lincoln wellfield. With the exception of Scenario 1, which is intended to simulate a future that is very similar to the near past (specifically the climate from 1989-2013), the scenarios are intended to represent the results of an evaluation of potential changes in the future climate, which was conducted by Martha Shulski and is summarized in Appendix A. Two major changes are expected to occur in eastern Nebraska, based on the down-scaled results of a suite of global climate models. The climate is expected to get wetter overall, though these wetter conditions will primarily occur during the winter and spring months, and conditions will be dryer during the summer, with no change expected for the fall. Therefore, it is reasonable to expect that the aquifers that are simulated in these groundwater models may receive a greater amount of recharge during the winter and spring, and a lesser amount in the summer. The second change that is expected will be in temperatures, which are expected to be somewhat warmer during the summer months. The four scenarios are described in greater detail below.

Note: A modeling scenario is a set of conditions represented in the groundwater flow model.

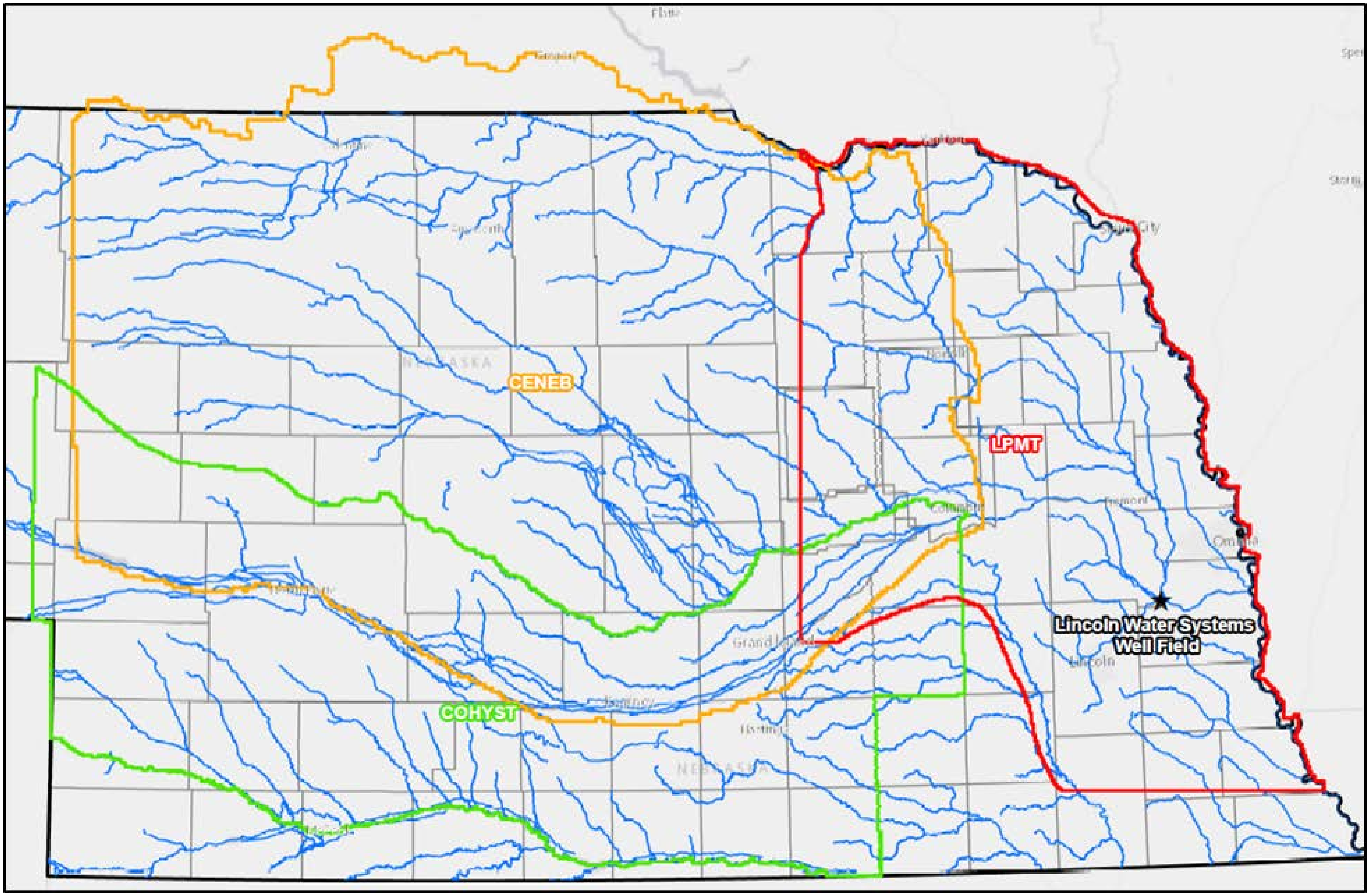

Figure 4-6 **Areal Extents of Regional Groundwater Models**

4.2.1 Modeling Scenarios

Scenario 1: Baseline conditions – this simulation simply repeated recent (late 1980's through early 2010's) climate conditions into the future. The purpose of this scenarios is to simulate how streamflows are predicted to change if climatic conditions do not change.

Scenario 2: Recharge changes – Climate models predict that in eastern Nebraska the precipitation will be somewhat greater in the future due to climate change. However, this increase will not be uniform throughout the year. The precipitation is expected to be greater during the months of December through May, and it is expected to be lower during the months of June through August. The purpose of this scenario is to simulate how streamflows are predicted to change if these changes in the precipitation occur and cause similar changes in the amount of groundwater recharge.

Scenario 3: Groundwater pumping increases – Climate models predict that in addition to being dryer during the summer months, it might also be warmer. This could lead to an increase in consumptive use demands for irrigated agriculture upstream of the Lincoln wellfield. Also, in some portions of the basin upstream of the Lincoln wellfield, groundwater irrigation is continuing to expand somewhat. For both reasons, groundwater pumping may increase in the future, causing a corresponding reduction in streamflow. The purpose of this scenario is to simulate how streamflows are expected to change if these increases in groundwater pumping occur.

Scenario 4: Recharge changes and groundwater pumping increases – The purpose of this scenario is to simulate how streamflows are predicted to change if both changes described in Scenario 2 and Scenario 3 occur.

4.2.2 Modeling Results

In order to evaluate the change in streamflow during future drought conditions, specifically at the Lincoln wellfield, the Years 2000-2006 were selected for evaluation. During these years, there were numerous occurrences of significantly low-flow conditions at the Lincoln wellfield. In the four modeling scenarios, these years recur in the future at roughly the Year 2040 and Year 2060 planning horizon being evaluated in this plan. To estimate the streamflows at these planning horizons under similar climate conditions, the actual modeled streamflows during the Years 2000-2006 were compared with the simulated streamflows at these planning horizons. The average monthly change in modeled streamflow under the Year 2040 planning horizon and the Year 2060 planning horizon are presented in Table 4-2 and Table 4-3, respectively.

These tables present the average monthly change in modeled streamflows during this 7-year period and are rounded to the nearest ten cfs. These results should be considered "order of magnitude" results, and not interpreted as a prediction of the exact quantitative results. Most of the reductions in streamflow are due to reductions that occur in the Central Platte River above the confluence with the Loup River. These reductions are masked to a great extent by increases in both the baseline run and during most months in the remaining scenarios from modeled streamflow increases in the Loup River, the Elkhorn River, and the Lower Platte River above Ashland.

Table 4-2 2040 Planning Horizon Difference in Modeled Streamflows (cfs)

SCENARIO	One	Two	Three	Four
January	-160	-70	-220	-140
February	-150	-70	-210	-130
March	-120	-30	-180	-90
April	-20	110	-80	50
May	130	320	70	260
June	60	180	0	120
July	-120	-40	-190	-110
August	-170	-100	-240	-180
September	-150	-80	-220	-150
October	-130	-60	-200	-120
November	-120	-50	-180	-110
December	-130	-60	-200	-120
Average	-90	0	-150	-60

Table 4-3 2060 Planning Horizon Difference in Modeled Streamflows (cfs)

Scenario	One	Two	Three	Four
January	-210	-90	-300	-170
February	-210	-90	-290	-170
March	-180	-50	-270	-140
April	-50	110	-140	30
May	120	350	40	260
June	40	200	-50	120
July	-170	-50	-270	-150
August	-190	-90	-290	-190
September	-160	-60	-260	-160
October	-120	-20	-220	-110
November	-120	-20	-220	-110
December	-150	-40	-240	-130
Average	-120	10	-210	-80

In order to put these results into the context of historical streamflows, the daily flow record for the stream gauge on the Platte River at Ashland was modified to reflect these potential changes in streamflows by month and scenario. Following this, the recurrence interval for various low flow event intervals were computed for the historical record and the four modified historical records. These low flows were determined using the U.S. Army Corps of Engineers Hydrologic Engineering Center Statistical Software Package Version 2.1.1.137 and the flow rates that were computed represent the maximum flow during a given time interval for a given return interval. For example, based on the historical flow record the 30-day low flow event that is expected to occur every other year on average (i.e., a 50% chance of the flow not being exceeded for 30 consecutive days in a given year) is approximately 2,600 cfs. The results of this evaluation for the 90-day low-flow at recurrence intervals between 5 years and 500 years for the 2040 results and the 2060 results are presented in Figure 4-7 and Figure 4-8, respectively. For additional context, these figures contain a label for 1500 cfs, 700 cfs, and 200 cfs. As can be seen, the worst case is Scenario 3, and a summary of the potential for these flows to not be met for a full 90-day period are summarized for the historical data and for the data adjusted according to the Scenario 3 results.

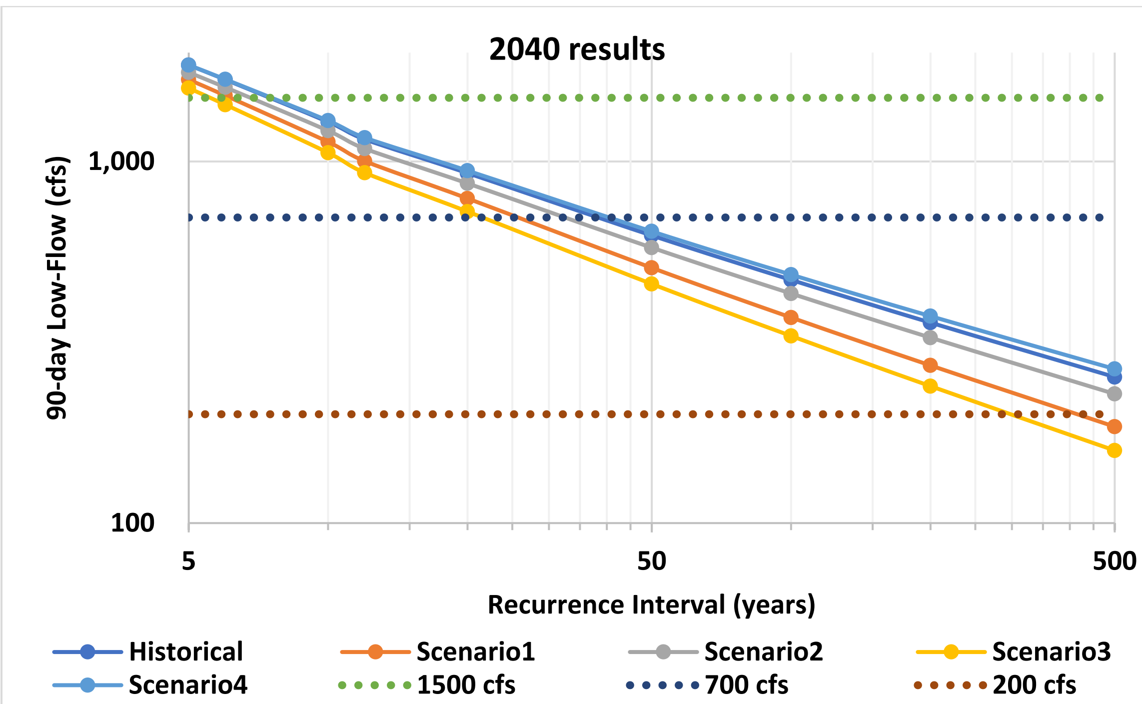

Figure 4-7 **90-Day Low-Flow Conditions for the Historical Data and Each Scenario for the 2040 Model Results for Recurrence Intervals Between 5 Years and 500 Years**

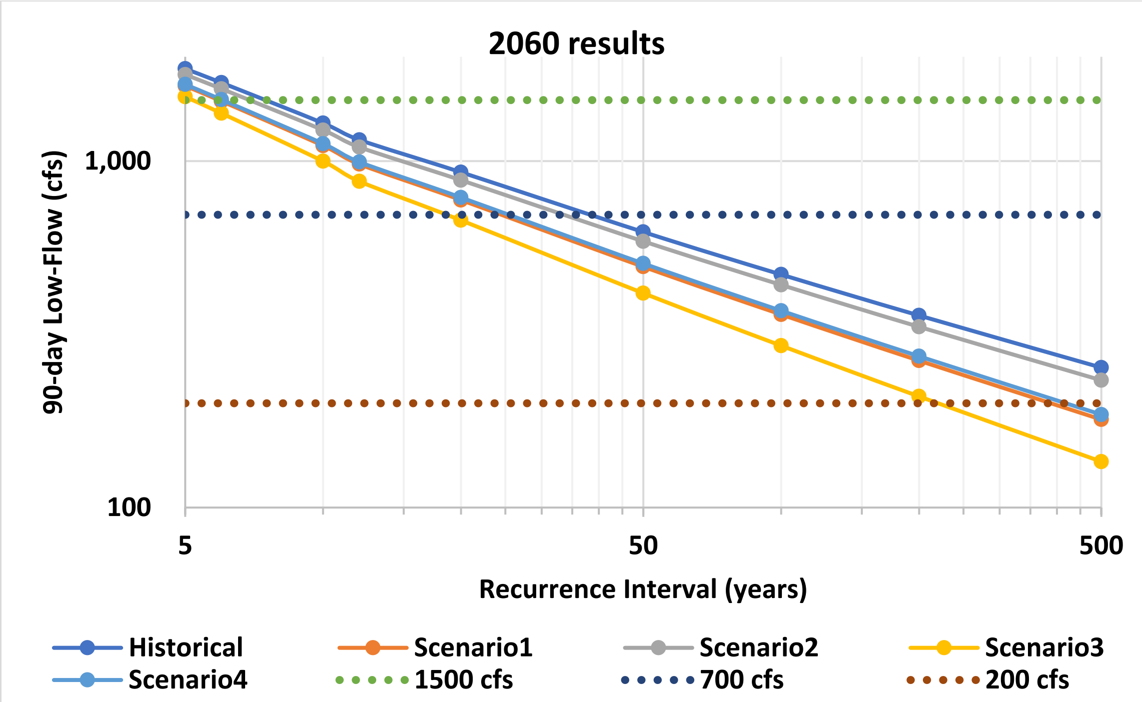

Figure 4-8 90-Day Low-Flow Conditions for the Historical Data and Each Scenario for the 2060 Model Results for Recurrence Intervals Between 5 Years and 500 Years

Table 4-4 2040/2060 Planning Horizon Chance of the Listed Low Flows Occurring At Least Once

	2040		2060	
Flow	Historically	Scenario 3	Historically	Scenario 3
1500	93%	97%	100%	100%
700	40%	64%	64%	87%
200	2%	6%	5%	18%

4.2.3 Climate Change Impact

The impact of climate change was a key consideration in the evaluation of water supply for the City of Lincoln. Results of the Climate Change Assessment (Appendix A) were used as direct inputs to the regional groundwater models, with the results of those modeling efforts summarized in Tables 4-2 and 4-3 for Year 2040 and 2060, respectively. The predicted impact to streamflow as a result of climate change can be derived by comparing Scenario 1 (Baseline Condition) and Scenario 3 (Groundwater Pumping Increases). In general, the anticipated decrease to streamflow in the critical late summer months of July, August, and September is around 70 cfs by Year 2040 and 100 cfs by Year 2060. This decrease is significant in comparison to the historical low flow benchmark for the river of 200 cfs. More specifically, climate change has the potential to reduce streamflow by 50 percent relative to the low flow benchmark by Year 2060.

4.3 Supply Improvements

The capacity of the wellfield is governed by two separate criteria. First, the aquifer must be capable of yielding the volume of water needed and second, the hydraulic capacity of the wells, pumps, and pipelines must be adequate to deliver maximum day demands. Relative to hydraulic capacity, the *2014 Master Plan* indicated that a majority of the flow from the fifth HCW could be conveyed through the existing 54-inch main to the East Plant. Therefore, no additional pipeline was recommended until the construction of the sixth HCW. It was also noted that the total raw water transmission capacity is approximately 145 mgd, which satisfies demands beyond the horizon of this study.

The primary focus of this update was to refine the 90-day seasonal yield, compare those yields to the project 90-day seasonal demand (as defined in Chapter 3), and update the Capital Improvement Program relative to timing of HCW-5 and HCW-6.

4.3.1 Aquifer Yield

The aquifer yield is evaluated using the USGS three-dimensional groundwater flow model, MODFLOW. The model for the LWS wellfield was first developed in the 1980's and has been updated periodically through the years to refine wellfield yield resulting from expansion of the raw water supply system.

Modeling scenarios have focused on river flows ranging from 200 cfs up to 3000 cfs. The low flow of 200 cfs represents a severe drought that may occur for a short duration, while 3000 cfs represents the condition at which point the river is flowing bank-to-bank and experiencing uniform recharge. The MODFLOW analyses were completed by Lamp Rynearson and the results of the analyses are summarized in a technical memorandum, included as Appendix B to this report.

4.3.2 MODFLOW Model Refinements

Two specific model refinements were made to the model used in previous studies. One of these changes was made based on climate modeling conducted by Martha Shulski, the Nebraska State Climatologist. Climate models indicate that on average in the future, fall through spring will tend to be 15 percent wetter, and summers will be 12.5 percent dryer. These results were incorporated into the model by adjusting the precipitation recharge. The 15 percent wetter fall through springs were included by increasing the recharge during the Antecedent period by 15 percent. The dryer summers were included by reducing recharge during the Dry Spring antecedent condition and during each drought scenario by 12.5 percent. The scenario descriptions provide more details on antecedent condition modeling.

The second refinement made was with regard to the location of the Platte River in low flow conditions. Observations made in support of previous modeling indicated that at flow rates less than 3000 cfs, the Platte River is no longer running bank-to-bank, and instead runs in smaller channels in the river bed. In previous studies, the river was modeled as running along the west bank north of Highway 6 and along the east bank south of Highway 6. For this study, an analysis was conducted comparing results where the river was run fully along the west bank.

4.3.3 Sensitivity Analyses

The two refinements described above were analyzed to evaluate to how the model results would be impacted by their incorporation into drought planning scenarios. The changes in precipitation recharge had little effect on well field production. This is attributable to the drought scenarios having a short duration and the prior model assumption that precipitation recharge is only

5 percent of the annual total precipitation during a drought. These combine to add very little water to simulation, and a reduction in that water supply had minimal impact on well field yields.

Placing the river to the west of Ashland Island had significant impact on yields compared to previous modeling. This is attributable to the increased distance between the horizontal wells and their water source. As the river moves further away from the horizontal wells, sustainable production from the wells drops considerably.

4.3.4 Wellfield Expansion

The benchmark for the most recent wellfield expansion is the capability to supply the summer seasonal demands over a 90-day period with the river level at 200 cfs. The MODFLOW modeling results determined that the existing system is capable of producing 90 mgd over the 90-day duration. As shown on Figure 4-9 on the following page, the existing facilities are capable of meeting this hypothetical design condition through Year 2035. Installation of an additional horizontal collector well (HCW-5) by Year 2035 would be considered a “just in time” improvement. It is therefore recommended that the City consider advancing this improvement a few years in the capital improvement plan to be ahead of the demand. MODFLOW modeling was also performed to determine the 90-day system capacity with the implementation of HCW-5 and HCW-6. These analyses indicate that with the two future wells, LWS’s projected seasonal capacity would be 105 mgd.

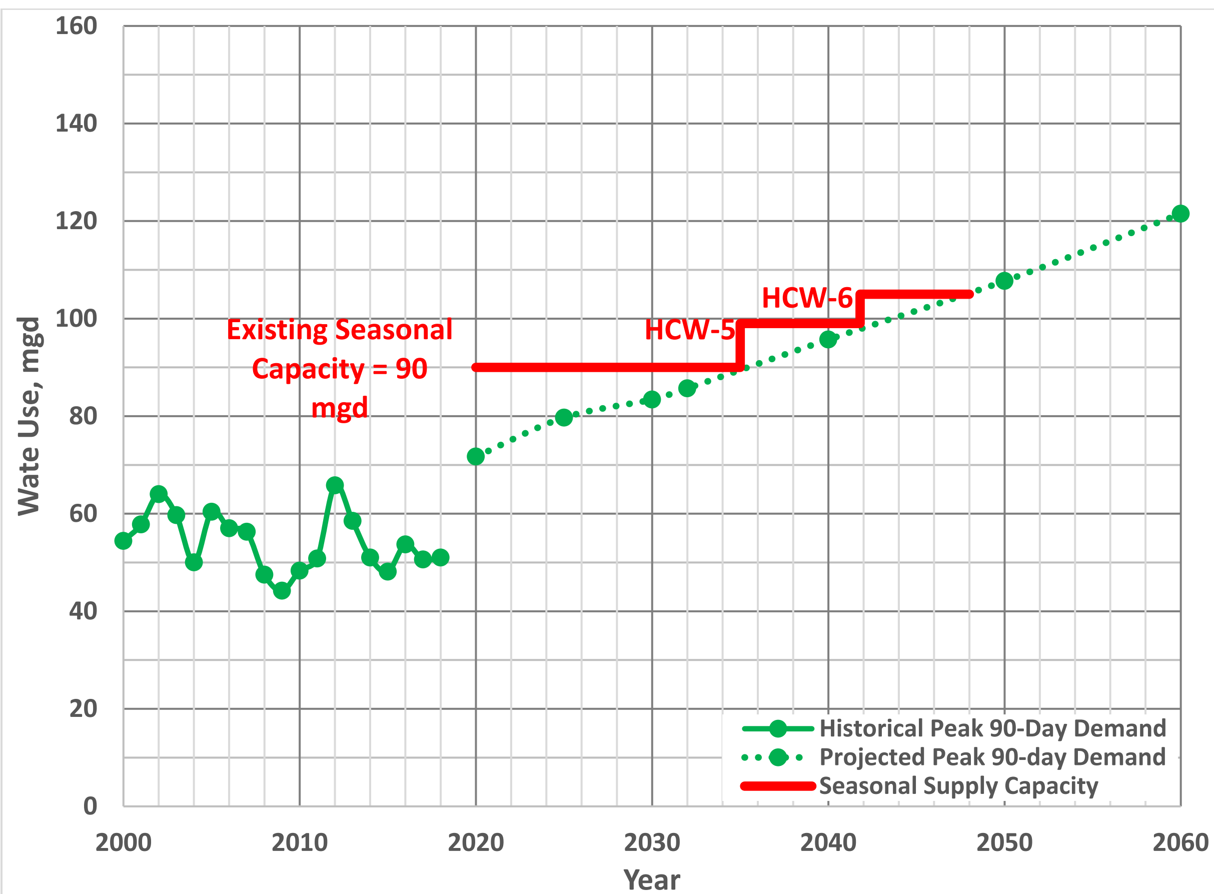

Figure 4-9 Future Supply Expansion

5.0 Water Treatment

5.1 General

Lincoln Water Systems (LWS) owns and operates two water treatment facilities co-located near Ashland. The West Plant was originally constructed in 1935, with major expansions between Years 1948 and 1976 to increase the plant design capacity to 60 million gallons per day (mgd). The East Plant was constructed in 1994 with an initial capacity of 50 mgd and was later increased to a plant capacity of 60 mgd by re-rating of the dual media filters. The agreement to re-rate the East Plant specifies that the filters may be operated at a maximum filter loading rate of 6.0 gpm/sf, under the condition that filter effluent turbidity is less than or equal to 0.1 Nephelometric Turbidity Units (NTU). Therefore, the total treatment capacity of the LWS water treatment facilities is 120 mgd. The East Plant facility was originally designed to be expandable to 150 mgd based upon a filter loading rate of 5.0 gpm/sf. It is anticipated that the East Plant will be expanded in increments of 30 mgd (based upon filter loading rate of 6.0 gpm/sf) to provide an ultimate capacity of 180 mgd in the future.

5.2 East Plant

5.2.1 Water Treatment

The East Plant process flow diagram is shown in Figure 5-1. The East Plant consists of the following treatment processes:

- Ozonation for primary disinfection and oxidation of iron, manganese and atrazine.
- Free chlorine for primary disinfection.
- Filter-aid polymer addition.
- Dual media gravity filtration.
- Fluoride addition.
- Chloramines for secondary disinfection.

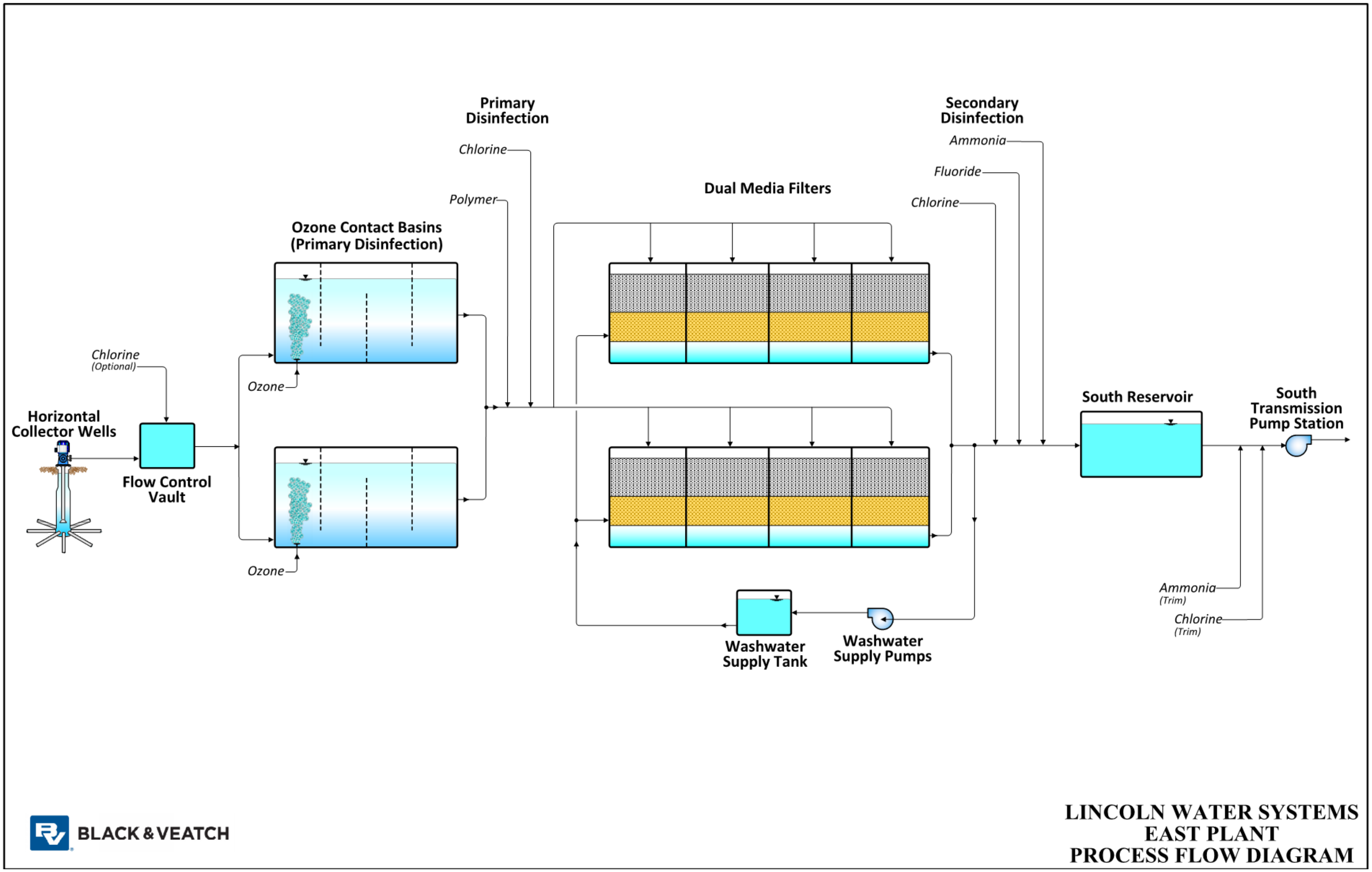

Figure 5-1 **East Plant Process Flow Diagram**

5.2.2 Water Supply

The East Plant receives raw water from the Platte River aquifer by four horizontal collector wells (HCWs). The HCW capacities are summarized in Table 5-1. The total capacity is defined as the design capacity of the well with all pumps running. The firm capacity is defined as the capacity of the well with the largest pump out of service. The hydrogeologic capacity is defined as the maximum capacity of the well as determined through performance testing. The East Plant also has the ability to receive groundwater from the vertical wells and blend supplies for control of atrazine and arsenic.

Table 5-1 East Plant Horizontal Collector Well Rated Capacities

Well Designation	Hydrogeologic Capacity (MGD)	Total Capacity (MGD)	Firm Capacity (MGD)
Horizontal collector well, 90-1	17.5	17.5	17.5
Horizontal collector well, 90-2	17.5	17.5	17.5
Horizontal collector well, 14-1(1)	19.4	19.4	19.4
Horizontal collector well, 14-2(2)	20	17.5(1)	13.5

(1)HCW 14-2 was designed to include smaller pumps for operational considerations. The hydrogeologic capacity of the well is approximately 20 mgd.
(2)HCW 14-2 was rerated by Layne after performance testing was conducted at the conclusion of the wellhouse construction.

5.2.3 Source Water

The source water is classified as ground water under the direct influence (GWUDI) of surface water. Based on this source water classification, the East Plant is required to achieve the 3.0 log removal/inactivation of *Giardia* and 4.0 log removal/inactivation of viruses in accordance with the Surface Water Treatment Rule (SWTR). Table 5-2 provides a summary of the log removal credits received based on water treatment processes. The East Plant receives 2.0 log removal credit of *Giardia* and 1.0 log removal credit of viruses for direct filtration. The ozone and chlorine disinfection processes are designed to provide the remaining 1.0 log inactivation of *Giardia* and 3.0 log inactivation of viruses.

Table 5-2 Log Removal/Inactivation for Filtration and Disinfection Required by SWTR

Process	*GIARDIA* Log Removal	Viruses Log removal
Conventional sedimentation/filtration credit Disinfection inactivation required	2.5 0.5	2.0 2.0
Direct filtration credit Disinfection inactivation required	2.0 1.0	1.0 3.0
Slow sand filtration credit Disinfection inactivation required	2.0 1.0	2.0 2.0
No filtration Disinfection inactivation required	0.0 3.0	0.0 4.0

5.2.4 Ozone Facilities

5.2.4.1 Ozone Contact Basins

The ozone facility at the East Plant includes two ozone contact basins. Each ozone contact basin is sized for 30 mgd treatment capacity. The basins have a shared inlet chamber with slide gates to direct flow to either or both of the ozone contact basins. Each basin was originally designed to include four internal cells with ozone added through fine bubble diffusers in the first two cells. In Year 2013, baffle walls were installed to split the first cell of the basin to accommodate sidestream injection, which in turn improved ozone transfer efficiency and mixing. Following the retrofit, ozone is now delivered via sidestream injection in the first cell. The ozone system utilizes three sidestream injection pumps (2 duty, 1 standby). Flow is directed in a counter-current manner to maximize transfer efficiency. The ozone system improvements also included addition of new sampling locations. Ozone residual is measured in cells 2 and 4 for quantifying disinfection CT (concentration x time) credits. The design parameters for the ozone contact basin are provided in Table 5-3.

Table 5-3 East Plant Ozone Contact Basin Design Parameters

Design Parameter	Units	Value
Number of ozone contact basins	Nos.	2
Unit capacity	mgd	30
Unit volume	gal	281,700
Theoretical detention time at maximum capacity	min	13.5
Baffling factor, T_{10}/T	-	0.58
Effective contact time at maximum capacity	min	7.3

5.2.4.2 Ozone Generation

The East Plant's ozone generation system was upgraded in Year 2013 with two new 1300 pound per day (ppd) generators. The ozone generation system is designed to produce ozone at a concentration of 2 to 12 percent by weight. Table 5-4 summarizes the design parameters for the ozone generation system.

Table 5-4 East Plant Ozone Generation Design Parameters

Design Parameter	Units	Value
Number of units	Nos.	2 (1 duty, 1 standby)
Design unit capacity	ppd	1300
Design ozone concentration	% by weight	2-12
Maximum capacity at low % weight	ppd	1600
Maximum applied dose	mg/L	3.2
Assumed transfer efficiency	%	95%
Maximum transferred dose	mg/L	3.0

5.2.4.3 Liquid Oxygen Storage

The ozone system improvements also included the replacement of the East Plant's air preparation system (refrigerant dryers and desiccant dryers) system with a liquid oxygen (LOX) storage system and supplemental air system. Table 5-5 summarizes the design parameters for the LOX system.

Table 5-5 East Plant LOX Storage Design Parameters

Design Parameter	Units	Value
Number of storage tanks	Nos.	1
Tank volume	gal	13,000
Number of vaporizers	Nos.	3 (1 duty, 1 standby, 1 defrost)

5.2.4.4 Destruct Equipment

Three catalytic destruct systems receive off-gas from the ozone contactor through off-gas demisters, located in each contact basin. Off-gas from the contactors may contain up to 0.25 percent ozone at 95 percent transfer (and 0.6 percent ozone at 12 percent), which exceeds the 0.1 mg/L limit established by OSHA for continuous exposure. Each destruct unit is sized to achieve a maximum ozone concentration of 0.10 parts per million by volume (ppmv) measured in the effluent of the destruct unit. Vent blowers disperse the treated off-gas into the atmosphere. In Year 2013, control valves and pressure transmitters were installed to automate the destruct process.

5.2.5 Filters

The ozonated water is conveyed through a filter influent flume where chlorine and filter-aid polymer are added prior to distribution to the filters. Free chlorine is used to obtain additional CT credits for primary disinfection and it also enhances manganese removal through the filters. From these flumes, the chlorinated water is directed into each filter through a 30-inch filter influent pipe.

The East Plant includes eight dual media filters, which are rated for a maximum filter loading rate of 6.0 gpm/ft^2. Each filter is divided into two 15 feet by 30 feet cells, which provide a total loading area of 900 ft^2 per filter. The filter media is dual media above a 12-inch gravel base layer supported by Leopold "Universal" underdrains. Each filter is equipped with air backwash facilities and fiberglass wash water troughs.

Table 5-6 provides a summary of the design parameters for the filtration system. Loading rate of 6.0 gpm/sf is contingent upon compliance with filter effluent turbidity less than 0.1 Nephelometric turbidity units (NTU).

Table 5-6 East Plant Filter Design Parameters

Design Parameter	Units	Value
Number of filters	Nos.	8
Filter media configuration	-	Dual media (10” sand, 20” anthracite)
Number of cells per filter	Nos.	2
Filter cell dimensions	ft x ft	15 x 30
Filter loading area	ft^2	900
Maximum filter loading rate	gpm/ft^2	6.0
Maximum capacity with all units online (N)	mgd	62.2
Maximum capacity with one unit offline (N-1)	mgd	54.4

Filter backwash is initiated after a filter run-time of 300 hours or when headloss exceeds 7 feet. Table 5-7 provides a summary of the filter backwash system design parameters and typical operating conditions.

Table 5-7 East Plant Filter Backwash Operations

Design Parameter	Units	Value
High rate filter backwash flow rate	mgd	22
High rate filter backwash loading rate	gpm/ft^2	17
Low rate filter backwash flow rate	mgd	6
Low rate filter backwash loading rate	gpm/ft^2	4.6
Filter backwash sequence	-	low rate – 3 min high rate – 8 min low rate – 3 min
Individual backwash volume	gal	147,500
Backwash volume required for 2 backwashes	gal	295,000

Backwash water is fed by gravity from the wash water supply tank, which is filled from two wash water supply pumps. The wash water supply tank is sized for a minimum of two filter backwashes; however, there are some operational challenges associated with conducting two successive backwashes due to insufficient driving pressure from low water levels in the tank. The wash water tank is a steel ground storage reservoir located southeast of the filter building.

Table 5-8 provides a summary of the design parameters for the wash water supply tank and wash water supply pumps.

Table 5-8 East Plant Wash Water Supply Tank Design Parameters

Design Parameter	Units	Value
Wash Water Supply Tank		
Number of backwash volumes	Nos.	2
Tank volume	gal	370,000
Tank diameter	ft	46
Side water depth	ft	30
Wash Water Supply Pumps		
Number of wash water supply pumps	Nos.	2
Pump unit capacity	gpm	940
Pump type	-	Horizontal centrifugal

Backwash return is collected in a pipeline, dechlorinated using Captor® calcium thiosulfate solution and delivered to a mixing chamber, which receives backwash return from both the East and West Plants. From the mixing chamber, the backwash return is discharged to the outfall. The plant's National Pollutant Discharge Elimination System (NPDES) permit requires total chlorine to be at non-detectable levels in the plant effluent discharge.

5.2.6 Clearwells & Reservoirs

Filter effluent is collected in the clearwells located underneath each row of filters. The clearwells have a baffling factor of 0.5. Table 5-9 summarizes the design parameters of the clearwells.

Table 5-9 East Plant Clearwell Design Parameters

Design Parameter	Units	Value
Number of clearwells	Nos.	2
Unit volume	gal	157,000
Clearwell dimensions	ft x ft	144.8 x 25
Clearwell depth	ft	5.8

From the clearwells, filtered water is conveyed by gravity to the South Reservoir. In the pipeline between the clearwells and South Reservoir, fluoride is added to achieve a target finished water concentration ranging from 0.8 to 1.2 mg/L. Additionally, chlorine and ammonia can be added at this location for disinfectant residual trimming.

The South Reservoir has 6 MG of finished water storage and is divided into two cells with baffled compartments. The South Reservoir dimensions are provided in Table 5-10.

Table 5-10 South Reservoir Design Parameters

Design Parameter	Units	Value
Total Volume	MG	6.0
Number of cells	Nos.	2
Reservoir dimensions	ft x ft	289.5 x 160
Reservoir depth	ft	16.5

5.2.7 Primary Disinfection

The East Plant is required to achieve 1.0-log inactivation of *Giardia* and 3.0-log inactivation of viruses for primary disinfection. CT credits for primary disinfection are achieved in the ozone contact basins and through chlorine residual carried through the filter influent flume and clearwell. Additional CT credits are also obtained through monochloramine residual in the reservoir.

5.2.8 Secondary Disinfection

Chloramines are formed in the channel between the Clearwell and South Reservoir to provide a secondary disinfectant residual. The plant has historically maintained a total chlorine residual of 2.5 mg/L at the point of entry. However, since Year 2018, the plant has operated with an elevated total chlorine residual ranging from 3.1 to 3.5 mg/L to inhibit bacterial regrowth and nitrification in the distribution system.

5.3 West Plant

5.3.1 Water Treatment

The West Plant process flow diagram is shown in Figure 5-2. The West Plant consists of the following treatment processes:

- Aeration for oxidation of iron and manganese.
- Free chlorine addition for primary disinfection.
- Filter-aid polymer addition (optional).
- Monomedia sand filtration.
- Fluoride addition.
- Chloramines for secondary disinfection.

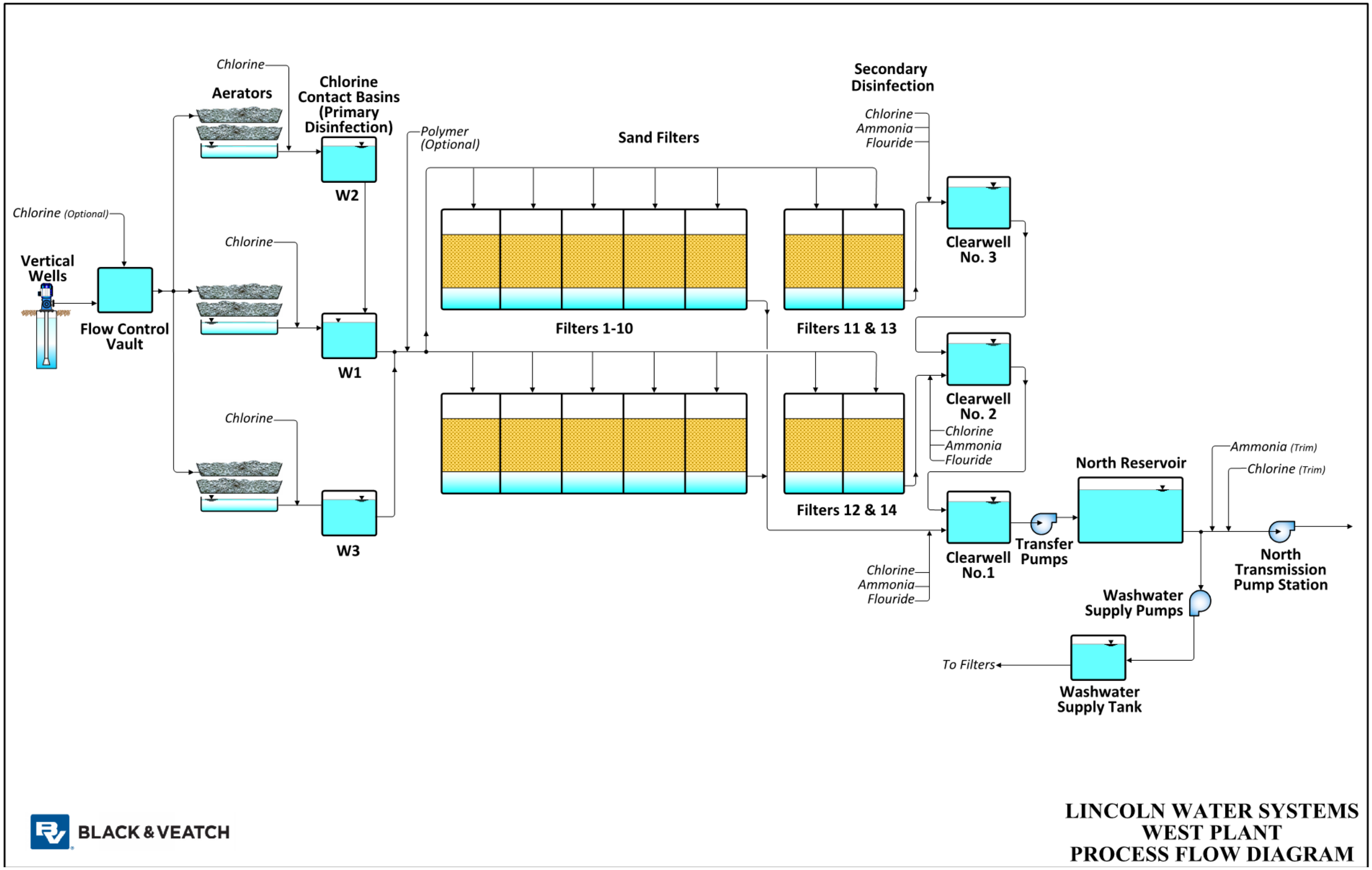

Figure 5-2 West Plant Process Flow Diagram

5.3.2 Water Supply

The West Plant receives ground water supplied by vertical wells and is designated as a ground water source with treatment governed by the Ground Water Rule (GWR). Based on this source water classification, the West Plant is required to achieve 4.0-log removal/inactivation of viruses in accordance with the GWR, which is accomplished by chlorine disinfection.

5.3.3 Aeration

The raw water supplied by the vertical wells contains iron and manganese at concentrations of up to 0.05 mg/L and 0.3 mg/L, respectively. The raw water is delivered to three coke tray aerators to oxidize the iron and manganese. The coke tray aerators cascade the water over a series of trays containing coke coarse media. The media provides increased surface area for air-to-water contact to increase the efficiency of iron and manganese oxidation. The aerated water is collected in a contact basin located below each of the coke tray aerators. Table 5-11 provides a summary of the design parameters for the coke tray aerators.

Table 5-11 West Plant Aerator Design Parameters

Parameter	Aerator #1 (W1)	Aerator #2 (W2)	Aerator #3 (W3)
Flow through each unit, mgd	18.8	17.8	23.4
Tray surface area, ft2	2,175	1,720	2,210
Tray flow rate, gpm/ft2	6.0	7.2	7.4

5.3.4 Chlorine Contact Basins

Chlorine is added to the contact basins which are located downstream of the coke tray aerators. The contact basins are operated in parallel for the most part with an exception being the interconnect between contact basins W1 and W2. The interconnect allows effluent from Contact Basin W2 to be delivered to the midpoint of Contact Basin W1 and blended with water in that basin.

The contact basins provide sufficient contact time for primary disinfection with free chlorine and allow for manganese oxidation reactions to take place. Table 5-12 provides a summary of the design parameters for the coke tray aerators.

Table 5-12 West Plant Chlorine Contact Basin Design Parameters

Parameter	Contact Basin W1		Contact Basin W2	Contact Basin W3
	1ST Half	2nd Half		
Unit volume, MG	0.32	0.32	1.20	2.21
Maximum flow rate through basin, MGD	20	37	17	23
Theoretical detention time at maximum flow rate, min	23	12.5	102	138
Baffling factor, T_{10}/T	0.5	0.5	0.5	0.5
Effective contact time, min	11.5	6.2	50.8	69.2

5.3.5 Filters

The chlorinated water is conveyed through a filter influent flume where flow is distributed into two filter influent channels. Filter-aid polymer may be optionally added in the filter influent channel. The West Plant includes fourteen monomedia sand filters, which are rated for a maximum filter loading rate of 4.5 gpm/ft^2. Each filter is divided into two cells, but the individual dimensions vary since the filters were constructed in three phases. Filters 1 through 6 were constructed in Year 1935, Filters 7 through 10 were constructed in Year 1948, and Filters 11 through 14 were constructed in Year 1956. The filter media consists of 36 inches of sand, which is supported by lateral underdrains on Filters 1 through 6 and clay tile underdrains on Filters 7 through 14. Table 5-13 provides a summary of the design parameters for the filtration system.

Table 5-13 West Plant Filter Design Parameters

Parameter	Phase 1 (1935)	Phase 2 (1948)	Phase 3 (1956)
Number of filters	6 each (Filters 1-6)	4 each (Filters 7-10)	4 each (Filters 11-14)
Number of cells per filter	2	2	2
Filter cell dimensions, ft x ft	20 x 13	20 x 13	20 x 25.5
Filter loading area, ft^2	520	520	1,020
Maximum capacity with all units online, mgd	20	13	27

Filter backwash is initiated after a filter run-time of 300 hours or when headloss exceeds 7 feet. Table 5-14 provides a summary of the filter backwash system design parameters and typical operating conditions. Following the backwash, the filter is typically operated in filter-to-waste mode for approximately 10 to 15 minutes prior to being returned to service.

Table 5-14 West Plant Filter Backwash Operations

Design Parameter	Units	Value (Filters 1-10)	Value (Filters 11-14)
High rate filter backwash flow rate	mgd	11.5	22
High rate filter backwash loading rate	gpm/ft^2	15.4	15
Low rate filter backwash flow rate	mgd	3	6
Low rate filter backwash loading rate	gpm/ft^2	4	4
Filter backwash sequence	-	low rate – 3 min high rate – 6 min low rate – 3 min	
Individual backwash volume	gal	60,500	116,700

Backwash water is fed by gravity from the elevated wash water supply tank, which is filled from wash water supply pumps. The wash water tank was constructed in 1976. The wash water supply tank is sized for a minimum of two filter backwashes. Table 5-15 provides a summary of the design parameters for the wash water supply tank.

Table 5-15 West Plant Wash Water Supply Tank Design Parameters

Design Parameter	Units	Value
Wash Water Supply Tank		
Number of backwash volumes	Nos.	2
Tank volume	gal	300,000
Tank diameter	ft	43
Side water depth	ft	29.6

Backwash return is collected in a pipeline, dechlorinated using Captor® calcium thiosulfate solution and delivered to a mixing chamber, which receives backwash return from both the East and West Plants. From the mixing chamber, the backwash return is discharged to the outfall. The plant's National Pollutant Discharge Elimination System (NPDES) permit requires total chlorine to be at non-detectable levels in the plant effluent discharge.

5.3.6 Filter Clearwells

Filter effluent is collected in three separate clearwells located underneath the filters. Each clearwell includes an influent flume with chemical feed points for fluoride, chlorine and ammonia. Chlorine feed is available for trimming; however, usually only ammonia is fed to form chloramines for secondary disinfection. The clearwells are interconnected, such that filtered water typically flows from Clearwell 3 to Clearwell 2, and then the combined flow from Clearwell 2 feeds into Clearwell 1. Table 5-16 indicates how individual filters feed into the three clearwells and provides a summary of clearwell dimensions and storage volumes.

Table 5-16 Filter Clearwell Design Parameters

Parameter	Clearwell #1	Clearwell #2	Clearwell #3
Filter designation	Filters 1-10	Filters 12 & 14	Filters 11 & 13
Unit volume, gal	413,000	293,000	293,000
Basin dimensions, ft x ft	162 x 20	90.5 x 20	90.5 x 20
Basin depth, ft	17	17	17

5.3.7 Transfer Pumps

There are three 18 mgd transfer pumps that convey water from the clearwells to the North Reservoir. The pumps are fed by 36-inch suction lines that draw from Clearwell 1. The transfer pumps include one adjustable frequency drive (Pump No. 1) to provide variable flow capability, while Pump Nos. 2 and 3 are constant speed. The capacity of these pumps is a limiting factor to operations such that they can only convey 52-54 mgd into the North Reservoir during periods of peak demand.

5.3.8 North Reservoir

From the transfer pumps, water is delivered to the North Reservoir. The North Reservoir is a rectangular cast in place below grade tank with baffle walls and provides 3 MG of finished water storage.

5.3.9 North High Service Pump Station

There are six high service pumps in the North High Service Pump Station. Under normal operations, the plant utilizes three high service pumps (Pumps No. 1-3) to deliver finished water from the West Plant to the distribution system. The City avoids using three of the high service pumps (Pumps No. 4-6) as much as possible due to the electrical demand and associated charges. Additionally, the suction line for Pumps No. 4-6 is connected to the filter clearwells. Therefore, when these pumps are put in service to meet demands greater than 50 mgd, the plant finished water bypasses the North Reservoir.

5.3.10 West Transmission Pump Station

There are two diesel pumps and one electrical pump located in the West Transmission Pump Station. The west transmission pumps are typically only used for peak shaving or when the West Plant is required to operate at flow rates above 50 mgd. These pumps draw water from Clearwells 2 and 3. Therefore, when these pumps are in operation, the plant finished water bypasses the North Reservoir.

5.4 Chemical Systems

The East and West Plants are serviced by a common chemical storage and feed facility, which was constructed in 1992. Based on findings from a facility condition assessment, specific systems within the Chemical Building were identified as needing replacement. The following chemical equipment systems are being replaced in Year 2020:

- Chlorine feed system
- Ammonia feed system
- Polymer storage and feed system
- Fluoride feed system

5.4.1 Chlorine

The chlorine system is located within the first floor of the Chemical Storage Building and is currently being rehabilitated as part of an ongoing chemical system upgrade project. Chlorine is delivered and stored in one-ton containers. The chlorine system is comprised of two banks of four connected one-ton containers, two evaporators, mechanically actuated switchover valves, expansion tanks, vacuum regulators, pressure gauges, rupture disks, fifteen chlorine feeders, eductors, water supply, and chlorine solution feed assemblies. Only one chlorine gas cylinder can be open at a time.

Table 5-17 describes the chemical properties, feed rate, and feed equipment for the chlorine system currently in design and scheduled to be replaced in Year 2020. Chemical usage rates provided in the table represent minimum, average, and maximum daily chlorine usage rates from Years 2014 to 2019. Additionally, the chemical dosages have been calculated based on daily chlorine usage rates and daily plant flow rates.

Table 5-17 Chlorine Storage & Feed System Design Parameters

Parameter	Value
Chemical Information	
Delivered/Fed Chemical	100% Chlorine Gas
Historical Chemical Usage Rates	
Minimum (ppd)	654
Average (ppd)	2,045
Maximum (ppd)	3,283
Historical Chemical Dosages	
Minimum (mg/L)	3.07
Average (mg/L)	7.06
Maximum (mg/L)	9.89
Feed Equipment	
Type	Chlorine Gas Feeder
Quantity	15
Feeder Control	Automatic and manual start/stop. Automatic and manual rate control with local override.
Ancillary Equipment	
Chlorine Evaporator	
Quantity	2
Unit capacity, ppd	10,000
Vacuum Regulator	
Quantity	3 (2 duty, 1 in line spare)
Piping Materials	
Pressurized chlorine gas	Carbon Steel
Vacuum chlorine gas / chlorine solution	PVC

Table 5-18 provides a summary of the chlorine feed points as well as the designated feeders for the future chlorine feed system.

Table 5-18 Chlorine Feed Points

Feeder No.	Application Points
CHFD-201	Contact Basin W1
CHFD-202	Contact Basin W2
CHFD-203	Contact Basin W3
CHFD-204	East Filter Flume
CHFD-205	Standby
CHFD-206	84” Effluent
CHFD-207	North Pump Station Suction
CHFD-208	South Transmission Pump Station
CHFD-209	West Pump Station
CHFD-210	East Plant Clearwell E1
CHFD-211	East Plant Clearwell E2
CHFD-212	Standby
CHFD-213	Clearwell Channel W1
CHFD-214	Clearwell Channel W2
CHFD-215	Clearwell Channel W3

5.4.2 Ammonia

Ammonia is used to provide a chloramine residual. The bulk anhydrous ammonia storage tank, including the tank and two heater-driven vaporizers, is located outdoors adjacent to the Chemical Building parking lot. The Chemical Building is fed by ten ammonia feeders, which are located in an isolated room on the first floor of the building. The ammonia system is currently being rehabilitated as part of the chemical feed upgrade project and is scheduled to be replaced in Year 2020.

Table 5-19 describes the chemical properties, feed rate, and feed equipment for the ammonia system from the proposed design. Ammonia gas will continue to be delivered to the site in bulk and stored in the existing 2,000-gallon carbon steel storage tank. Chemical usage rates provided in the table represent minimum, average, and maximum daily ammonia usage rates from Years 2014 to 2019. Additionally, the ammonia dosages have been calculated based on daily chemical usage rates and daily plant flow rates.

Table 5-19 Ammonia Storage & Feed System Design Parameters

Parameter	Value
Chemical Information	
Delivered/Fed Chemical	100% Ammonia Gas
Historical Chemical Usage Rates	
Minimum (ppd)	139
Average (ppd)	251
Maximum (ppd)	386
Historical Chemical Dosages	
Minimum (mg/L)	0.69
Average (mg/L)	0.85
Maximum (mg/L)	0.97
Feed Equipment	
Type	Ammonia Gas Feeder
Quantity	10
Feeder Control	Automatic and manual start/stop. Automatic and manual rate control with local override.
Piping & Valves	
Pipe material	Carbon Steel

All existing ammonia gas feeders will be replaced along with the existing carbon steel piping around the ammonia feeders connecting to the headers. Table 5-20 presents a summary of the ammonia feed points and designated feeders.

Table 5-20 Ammonia Feed Points

Feeder No.	Application Points
CHFD-101	North Pump Station Suction
CHFD-102	West Plant Clearwell W1
CHFD-103	West Plant Clearwell W2
CHFD-104	West Plant Clearwell W3
CHFD-105	Standby
CHFD-106	Standby
CHFD-107	West Transmission Pump Station Wetwell
CHFD-108	South Transmission Pump Station Influent
CHFD-109	Standby
CHFD-110	South Reservoir Influent Flume

5.4.3 Polymer

LWS utilizes a polyDADMAC cationic polymer (Aqua Hawk 6527) as a filter aid. The polymer storage and feed system is located within the Chemical Storage Building and is comprised of one bulk storage tank, two neat polymer transfer pumps, two polymer mixing/aging tanks with mixers, two one-percent polymer solution transfer pumps, two one-percent polymer solution day tanks, and four one-percent polymer solution metering pumps. Secondary dilution water and static mixers are used to carry the polymer solution from the metering pumps to the points of application. Polymer is fed at the East Plant filter influent and has the ability to be fed to the West Plant filter influent. The polymer system is currently being rehabilitated as part of an ongoing chemical feed upgrade project and is scheduled to be replaced in Year 2020.

Table 5-21 describes the chemical properties, feed rate, and feed equipment for the polymer system for the proposed design. The existing neat polymer storage tank, mixing/aging tanks, and day tanks will be retained. The neat polymer transfer pumps will be replaced with new diaphragm transfer pumps. The 1% polymer solution gear type transfer pumps will be replaced with progressive cavity pumps. The four metering pumps will all be replaced in kind with motorized diaphragm metering pumps. The mixing/aging tanks will be provided new mixers with longer shafts. The PVC piping and valves will only be replaced as necessary around the new pumps.

Table 5-21 Polymer Storage and Feed System Design Parameters

Parameter	Value
Chemical Information	
Delivered Chemical	Neat emulsion polymer
Specific gravity	1.05
Fed chemical	1% polymer solution
Specific gravity	1.05
Historical Chemical Usage Rates (as neat polymer)	
Minimum (ppd)	247
Average (ppd)	1,277
Maximum (ppd)	2,286
Historical Chemical Dosages	
Minimum (mg/L)	0.02
Average (mg/L)	0.20
Maximum (mg/L)	0.24
Drum Pump	
Service	Neat polymer
Type	Diaphragm pump
Quantity	1
Capacity, gpm	10

Parameter	Value
Neat Polymer Transfer Pump	
Service	Neat Polymer
Type	Diaphragm Pump
Quantity	2 (1 duty, 1 standby)
Unit capacity, gph	13
Tag numbers	CHMP-501 CHMP-502
Mixing / Aging Tank Mixers	
Service	1% polymer solution
Quantity	2
Polymer Solution Transfer Pumps	
Service	1% polymer solution
Type	Progressive Cavity Pump
Quantity	2 (1 duty, 1 standby)
Capacity, gpm	20
Tag numbers	CHTP-501 CHTP-502
Polymer Solution Feed Equipment	
Type	Mechanical Diaphragm Metering Pumps
Quantity	4 (3 duty, 1 standby)
Unit capacity, gph	1.16 to 47.6
Pump control	Automatic and manual start/stop. Automatic and manual stroke length and stroke speed control with local override.
Piping & Valves	
Pipe material	PVC

Table 5-22 presents a summary of the designated feed points for each polymer metering pump included in the future polymer feed system.

Table 5-22 Polymer Feed Points

Metering Pump No.	Application Points
CHMP-503	East Plant filter influent
CHMP-504	East Plant filter influent
CHMP-505	East Plant filter influent
CHMP-506	East Plant filter influent

5.4.4 Fluoride

The fluoride system is located within the Chemical Building with equipment split between the first floor and the basement. The system consists of two bulk storage tanks, two transfer pumps, two days tanks, and five metering pumps. Fluoride is fed into the West Plant filter clearwells and into the East Plant 84-inch finished water supply. The fluoride system is currently being rehabilitated as part of an ongoing chemical feed upgrade project and is scheduled to be replaced in Year 2020.

Table 5-23 describes the chemical properties, feed rate, and feed equipment for the fluoride system as currently designed. Chemical usage rates provided in the table represent minimum, average, and maximum daily fluoride usage rates from Years 2014 to 2019. Additionally, the chemical dosages have been calculated based on daily fluoride usage rates and daily plant flow rates.

Table 5-23 Fluoride Storage & Feed System Design Parameters

Parameter	Value
Chemical Information	
Delivered Chemical	23-25% Hydrofluorosilicic acid (18-21% Fluoride)
Specific gravity	1.21
Historical Chemical Usage Rates	
Minimum (ppd)	422
Average (ppd)	972
Maximum (ppd)	1,646
Historical Chemical Dosages	
Minimum (mg/L as F)	0.41
Average (mg/L as F)	0.57
Maximum (mg/L as F)	0.88

Parameter	Value
Feed Equipment	
Type	Mechanical Diaphragm Metering Pumps
Quantity	5 (4 duty, 1 standby)
Pump Control	Automatic and manual start/stop. Automatic and manual stroke length and stroke speed control with local override.
Piping & Valves	
Pipe material	PVC

Table 5-24 presents a summary of the new fluoride metering pumps and designated feed points.

Table 5-24 Fluoride Feed Points

Metering Pump No.	Application Points
CHMP-301	West Plant Clearwell 2
CHMP-302	West Plant Clearwell 1
CHMP-303	West Plant Clearwell 3
CHMP-304	Standby
CHMP-305	East Plant 84” pipeline

5.5 Raw Water Quality

The raw water quality assessment is based on data provided for the following constituents and timeframes:

- Herbicide concentrations for individual samples collected between Years 2014 and 2019.
- Microbiological contaminant measures for individual samples collected between Years 2014 and 2018.
- General water quality parameters for individual samples collected between Years 2014 and 2018.
- Nitrogen species concentrations for individual samples collected between Years 2014 and 2018.
- Inorganics and metals concentrations for individual samples collected between Years 2017 and 2019.

5.5.1 East Plant Raw Water Quality Data

The East Plant primarily receives raw water from horizontal collector wells that are classified as ground water under the direct influence of surface water. A summary of the East Plant raw water quality is provided in Table 5-25.

Table 5-25 East Plant Raw Water Quality Summary

PARAMETER	UNITS	MIN(1)	AVG(1)	MAX
General Parameters				
Alkalinity, Total	mg/L as CaCO3	165	198	223
Ammonia, Free (NH3-N)	mg/L	0	0.04	0.21
Ammonia, Total (NH3-N)	mg/L	0	0.05	0.41
Dissolved Oxygen (DO)	mg/L	0.56	2.74	5.28
Hardness, Total	mg/L as CaCO3	174	250	306
Oxidation Reduction Potential (ORP)	mV	30	304	408
pH	s.u.	7.42	7.82	8.10
Temperature	°C	8.0	18.5	29.2
Total Organic Carbon	mg/L	1.74	2.94	5.12
Inorganic Chemicals				
Calcium	mg/L as CaCO3	131	173	211
Chloride	mg/L	11.9	14.2	17.2
Fluoride	µg/L	186	329	417
Magnesium	mg/L as CaCO3	51	69	90
Nitrate (as N)	mg/L	0.035	2.315	4.195
Nitrite (as N)	mg/L	0.00	0.03	0.45
Phosphate (PO4)	mg/L	0.65	0.82	1.02
Potassium	mg/L	5.88	8.64	11.7
Sodium	mg/L	21.5	27.3	35.1
Sulfate	µg/L	60.8	76.0	90.6
Metals				
Aluminum	µg/L	ND	4.16	73.3
Antimony	µg/L	0.16	0.37	0.58
Arsenic, Total	µg/L	5.33	7.50	9.69
Barium	µg/L	99	141	173
Beryllium	µg/L	ND	ND	0.011
Cadmium	µg/L	ND	0.05	0.26
Chromium, total	µg/L	ND	0.10	0.32

PARAMETER	UNITS	MIN[1]	AVG[1]	MAX
Cobalt	µg/L	0.03	0.11	0.21
Copper	µg/L	0.09	3.33	14.7
Germanium	µg/L	88	94	113
Germanium-1	µg/L	88	95	109
Iron, total	µg/L	ND	2.8	73.5
Lead	µg/L	ND	0.04	1.09
Manganese, Total	µg/L	1.60	43.4	199
Molybdenum	µg/L	2.20	3.52	4.90
Nickel	µg/L	0.73	1.72	27.7
Scandium-1	µg/L	91	98.5	118
Selenium	µg/L	2.93	7.93	19.3
Silver	µg/L	ND	0.002	0.276
Terbium	µg/L	87	98	121
Thallium	µg/L	ND	0.016	0.039
Zinc	µg/L	ND	2.59	102
Radionuclides				
Thorium	µg/L	ND	ND	0.50
Uranium	µg/L	6.99	10.3	13.0
Vanadium	µg/L	4.45	7.16	12.2
Herbicides				
Acetochlor	µg/L	0.15	0.30	0.49
Atrazine	µg/L	0.13	0.51	1.95
Desethylatrazine	µg/L	0.10	0.18	0.32
Metolachlor	µg/L	0.12	0.38	1.69
Simazine	µg/L	0.01	0.18	0.64
Microbiological Contaminants				
Coliform, total (P/A) [2]	A=0, P=1	0	0	1
E coli	A=0, P=1	0	0	0
Heterotrophic Plate Count	cfu/100 mL	0	131	999

Notes:
[1]"ND" indicates that the concentration was non-detect or below the method detection limit.
[2]For total coliform measurements, "P" indicates the presence of coliforms and "A" indicates an absence of coliforms in the sample collected.

5.5.2 West Plant Raw Water Quality Data

The West Plant primarily receives raw water from vertical ground water wells. A summary of the West Plant raw water quality is provided in Table 5-26.

Table 5-26 West Plant Raw Water Quality Summary

PARAMETER	UNITS	MIN(1)	AVG	MAX
General Parameters				
Alkalinity, Total	mg/L as CaCO3	18.8	165	224
Ammonia, Free (NH3-N)	mg/L	0	0.04	0.25
Ammonia, Total (NH3-N)	mg/L	0	0.04	0.46
Dissolved Oxygen (DO)	mg/L	1.42	2.89	6.71
Hardness, Total	mg/L as CaCO3	144	204	269
Oxidation Reduction Potential (ORP)	mV	2.29	337	485
pH	s.u.	7.11	7.50	7.93
Temperature	°C	3.1	16.6	23.6
Total Organic Carbon	mg/L	1.51	2.17	4.00
Inorganic Chemicals				
Calcium	mg/L as CaCO3	109	139	194
Chloride	mg/L	15.5	17.5	18.8
Fluoride	µg/L	200	366	459
Magnesium	mg/L as CaCO3	45.2	54.8	79.1
Nitrate (as N)	mg/L	0.037	0.55	1.67
Nitrite (as N)	mg/L	0.00	0.02	0.14
Phosphate (PO4)	mg/L	0.52	0.71	0.86
Potassium	mg/L	6.53	8.81	13.1
Sodium	mg/L	24.2	34.1	50.4
Sulfate	µg/L	72.4	86.3	101
Metals				
Aluminum	µg/L	ND	0.77	36.8
Antimony	µg/L	0.11	0.27	0.66
Arsenic, Total	µg/L	4.62	6.72	8.76
Barium	µg/L	88	106	163
Beryllium	µg/L	ND	0.00	0.10
Cadmium	µg/L	ND	0.05	0.46
Chromium, total	µg/L	ND	0.06	2.53
Cobalt	µg/L	0.02	0.11	0.51

PARAMETER	UNITS	MIN(1)	AVG	MAX
Copper	µg/L	0.27	6.37	14.1
Germanium	µg/L	77.1	88.1	95.1
Germanium-1	µg/L	77.0	90.4	104
Iron, total	µg/L	0.03	5.63	42
Lead	µg/L	ND	0.09	1.32
Manganese, Total	µg/L	0.57	51.2	258
Molybdenum	µg/L	2.31	3.37	4.61
Nickel	µg/L	0.63	1.49	4.95
Scandium-1	µg/L	77.5	92.1	98.9
Selenium	µg/L	0.38	1.87	9.16
Silver	µg/L	ND	0.01	0.12
Terbium	µg/L	84.4	97.9	119
Thallium	µg/L	ND	0.02	0.05
Zinc	µg/L	ND	4.73	35.5
Radionuclides				
Thorium	µg/L	ND	0.04	0.78
Uranium	µg/L	5.32	7.72	12.1
Vanadium	µg/L	3.89	5.51	7.27
Herbicides				
Atrazine	µg/L	0.09	0.27	0.46
Desethylatrazine	µg/L	0.10	0.14	0.28
Metolachlor	µg/L	0.11	0.22	0.43
Simazine	µg/L	0.02	0.03	0.03
Microbiological Contaminants				
Coliform, total (P/A) (2)	A=0, P=1	0	0.02	1
E coli	A=0, P=1	0	0	0
Heterotrophic Plate Count	cfu/100 mL	0	45	999

Notes:
(1)"ND" indicates that the concentration was non-detect or below the method detection limit.
(2)For total coliform measurements, "P" indicates the presence of coliforms and "A" indicates an absence of coliforms in the sample collected.

5.6 Finished Water Quality

The finished water quality analysis is based on data provided for the following constituents and timeframes:

- General water quality parameters for individual samples collected between Years 2014 and 2018.
- Nitrogen species concentrations for individual samples collected between Years 2014 and 2018.
- Inorganics and metals concentrations for individual samples collected between Years 2017 and 2019.
- Herbicide concentrations at the East and West Plant compliance monitoring sites between Years 2014 and 2019.
- Assimilable organic carbon (AOC) concentrations collected between Years 2001 and 2009.
- Microbiological contaminant measures for individual samples collected between Years 2014 and 2018.
- Disinfectant residual monitoring data between Years 2013 and 2018.

5.6.1 East Plant Finished Water Quality Data

A summary of the East Plant finished water quality is provided in Table 5-27.

Table 5-27 East Plant Finished Water Quality Summary

PARAMETER	UNITS	MIN(1)	AVG(1)	MAX(1)	Primary MCL	Secondary MCL
General Parameters						
Alkalinity, Total	mg/L as $CaCO_3$	159	193	219	-	-
Dissolved Oxygen (DO)	mg/L	8.13	11.6	14.5	-	-
Hardness, Total	mg/L as $CaCO_3$	180	254	317	-	-
ORP - Oxidation Reduction Potential	mV	471	505	533	-	-
pH	s.u.	7.20	7.65	7.96	-	6.5-8.5
Temperature	°C	6.70	17.5	23.5	-	-
Total Organic Carbon	mg/L	0.63	2.72	4.68	-	-
Assimilable Organic Carbon	µg acetate C/L	80	154	350	-	-
Inorganic Chemicals						
Calcium	mg/L as $CaCO_3$	130	173	212	-	-
Chloride	mg/L	14.7	19.6	34.8	-	250
Fluoride	mg/L	0.60	0.81	1.05	4.0	2.0
Magnesium	mg/L as $CaCO_3$	51	65	83	-	-
Nitrate (as N)	mg/L	1.19	2.04	3.41	10	-
Nitrite (as N)	mg/L	0.001	0.003	0.02	1	-

PARAMETER	UNITS	MIN(1)	AVG(1)	MAX(1)	Primary MCL	Secondary MCL
Phosphate (PO4)	mg/L	0.59	0.80	1.07	-	-
Potassium	mg/L	7.0	8.8	11.7	-	-
Sodium	mg/L	21.9	27.3	34.6	-	-
Sulfate	mg/L	0.00	66.1	93.4	-	250
Metals						
Aluminum	µg/L	ND	0.34	1.75	-	50-200
Antimony	µg/L	0.16	0.37	0.58	6	-
Arsenic, Total	µg/L	6.17	7.79	9.35	10	-
Barium	µg/L	74.3	135	164	2,000	-
Beryllium	µg/L	ND	0.00	0.007	4	-
Cadmium	µg/L	ND	0.017	0.148	5	-
Chromium, Total	µg/L	ND	0.06	0.32	100	-
Cobalt	µg/L	0.009	0.064	0.091	-	-
Copper	µg/L	ND	2.87	4.83	TT (AL=1,300) (2)	1,000
Iron, total	µg/L	ND	0.72	3.86	-	300
Lead	µg/L	ND	0.07	1.27	TT (AL=15) (2)	-
Manganese, Total	µg/L	1.22	3.97	11.5	-	50
Molybdenum	µg/L	2.46	3.69	4.66	-	-
Nickel	µg/L	0.29	0.83	2.65	-	-
Selenium	µg/L	2.72	7.14	17.4	0.05	-
Silver	µg/L	ND	0.0	0.03	-	100
Thallium	µg/L	ND	0.0	0.04	0.002	-
Zinc	µg/L	ND	1.89	23.3	-	5
Radionuclides						
Thorium	µg/L	ND	0	0.62	-	-
Uranium	µg/L	6.94	9.99	12.4	30	-
Vanadium	µg/L	3.42	7.08	10.3	-	-
Herbicides						
Alachlor	µg/L	ND	ND	ND	2	-
Aldrin	µg/L	ND	ND	ND	-	-
Atrazine	µg/L	ND	0.10	0.43	3	-
Benzo [a]pyrene	µg/L	ND	ND	ND	0.2	-
Butachlor	µg/L	ND	ND	ND	-	-
Butylate	µg/L	ND	ND	ND	-	-

PARAMETER	UNITS	MIN(1)	AVG(1)	MAX(1)	Primary MCL	Secondary MCL
Chlordane	μg/L	ND	ND	ND	2	-
Chlorpyrifos	μg/L	ND	ND	ND	-	-
Cyanazine	μg/L	ND	ND	ND	-	-
Di (2-ethylhexyl) adipate	μg/L	ND	ND	ND	400	-
Di (2-ethylhexyl) phthalate	μg/L	ND	ND	ND	6	-
Dieldrin	μg/L	ND	ND	ND	-	-
Endrin	μg/L	ND	ND	ND	2	-
Fonofos	μg/L	ND	ND	ND	-	-
Heptachlor	μg/L	ND	ND	ND	0.4	-
Heptachlor epoxide	μg/L	ND	ND	ND	0.2	-
Hexachlorobenzene	μg/L	ND	ND	ND	1	-
Hexachlorocylcopentadiene	μg/L	ND	ND	ND	50	-
Lindane	μg/L	ND	ND	ND	0.2	-
Methoxychlor	μg/L	ND	ND	ND	40	-
Metolachlor	μg/L	ND	0.05	0.29	-	-
Metribuzin	μg/L	ND	ND	ND	-	-
Propachlor	μg/L	ND	ND	ND	-	-
Simazine	μg/L	ND	ND	ND	4	-
Trifluralin	μg/L	ND	ND	ND	-	-
Disinfectant Residual						
Ammonia, Free (NH3-N)	mg/L	0	0.08	0.23	-	-
Ammonia, Total (NH3-N)	mg/L	0.41	0.67	1.09	-	-
Chlorine Free	mg/L	0	0.01	0.16	4.0	-
Chlorine Total	mg/L	1.42	2.61	3.91	4.0	-
Dichloramine	mg/L	0	0.18	0.76	-	-
Monochloramine	mg/L	0.25	2.30	3.43	-	-
Microbiological Contaminants						
Coliform, total (3)	A=0, P=1	0	0	0	< 5% P	-
E coli	A=0, P=1	0	0	0	< 5% P	-
Heterotrophic Plate Count	cfu/100 mL	0	0.58	6.00	-	-

Notes:
(1)"ND" indicates that the concentration was non-detect or below the method detection limit.
(2)AL = Action Level. ALs for lead and copper are monitored in the distribution system. Finished water quality data presented in this table is not for Lead and Copper Rule compliance monitoring.
(3)For total coliform measurements, "P" indicates the presence of coliforms and "A" indicates an absence of coliforms in the sample collected.

5.6.2 West Plant Finished Water Quality Data

A summary of the West Plant finished water quality is provided in Table 5-28.

Table 5-28 West Plant Finished Water Quality Summary

PARAMETER	UNITS	MIN(1)	AVG(1)	MAX(1)	Primary MCL	Secondary MCL
General Parameters						
Alkalinity, Total	mg/L as $CaCO_3$	143	162	216	-	-
Dissolved Oxygen (DO)	mg/L	8.77	9.78	11.96	-	-
Hardness, Total	mg/L as $CaCO_3$	180	207	283	-	-
ORP - Oxidation Reduction Potential	mV	431	501	548	-	-
pH	s.u.	7.32	7.63	7.96	-	6.5-8.5
Temperature	°C	12.50	17.9	22.7	-	-
Total Organic Carbon (TOC)	mg/L	1.70	2.16	3.70	-	-
Assimilable Organic Carbon (AOC)	µg acetate C/L	0	80	180	-	-
Inorganic Chemicals						
Calcium	mg/L	43.3	55.2	78.0	-	-
Chloride	mg/L	18.2	23.7	47.8	-	250
Fluoride	mg/L	0.62	0.87	1.11	4.0	2.0
Magnesium	mg/L	11.5	13.0	15.6	-	-
Nitrate (as N)	mg/L	0.42	0.90	1.71	10	-
Nitrite (as N)	mg/L	0.00	0.002	0.007	1	-
Phosphate (as PO4)	mg/L	0.50	0.71	0.90	-	-
Potassium	mg/L	7.06	8.66	11.32	-	-
Sodium	mg/L	29.2	32.6	37.3	-	-
Sulfate	mg/L	69	87	114	-	250
Metals						
Aluminum	µg/L	ND	0.52	3.50	-	50-200
Antimony	µg/L	0.17	0.27	0.39	6	-
Arsenic, Total	µg/L	5.45	7.00	8.72	10	-
Barium	µg/L	90.8	106	124	2,000	-
Beryllium	µg/L	ND	0.0002	0.075	4	-
Cadmium	µg/L	ND	0.024	0.082	5	-
Chromium, Total	µg/L	ND	0.04	0.63	100	-
Cobalt	µg/L	0.03	0.08	0.12	-	-

PARAMETER	UNITS	MIN(1)	AVG(1)	MAX(1)	Primary MCL	Secondary MCL
Copper	µg/L	ND	1.68	3.42	TT (AL=1,300) (2)	1,000
Iron, total	µg/L	0.17	1.38	5.67	-	300
Lead	µg/L	ND	0.07	1.28	TT (AL=15) (2)	-
Manganese, Total	µg/L	0.51	2.72	30.4	-	50
Molybdenum	µg/L	2.79	3.43	4.06	-	-
Nickel	µg/L	0.57	1.16	3.56	-	-
Selenium	µg/L	0.52	2.26	27.5	0.05	-
Silver	µg/L	ND	0.06	0.32	-	100
Thallium	µg/L	ND	0.0	0.05	0.002	-
Zinc	µg/L	ND	0.45	11.6	-	5
Radionuclides						
Thorium	µg/L	ND	0.31	0.97	-	-
Uranium	µg/L	6.02	7.80	10.3	30	-
Vanadium	µg/L	4.22	5.27	6.78	-	-
Herbicides						
Alachlor	µg/L	ND	ND	ND	2	-
Aldrin	µg/L	ND	ND	ND	-	-
Atrazine	µg/L	ND	0.103	0.168	3	-
Benzo [a]pyrene	µg/L	ND	ND	ND	0.2	-
Butachlor	µg/L	ND	ND	ND	-	-
Butylate	µg/L	ND	ND	ND	-	-
Chlordane	µg/L	ND	ND	ND	2	-
Chlorpyrifos	µg/L	ND	ND	ND	-	-
Cyanazine	µg/L	ND	ND	ND	-	-
Di (2-ethylhexyl) adipate	µg/L	ND	ND	ND	400	-
Di (2-ethylhexyl) phthalate	µg/L	ND	ND	ND	6	-
Dieldrin	µg/L	ND	ND	ND	-	-
Endrin	µg/L	ND	ND	ND	2	-
Fonofos	µg/L	ND	ND	ND	-	-
Heptachlor	µg/L	ND	ND	ND	0.4	-
Heptachlor epoxide	µg/L	ND	ND	ND	0.2	-
Hexachlorobenzene	µg/L	ND	ND	ND	1	-
Hexachlorocyclopentadiene	µg/L	ND	ND	ND	50	-

PARAMETER	UNITS	MIN(1)	AVG(1)	MAX(1)	Primary MCL	Secondary MCL
Lindane	µg/L	ND	ND	ND	0.2	-
Methoxychlor	µg/L	ND	ND	ND	40	-
Metolachlor	µg/L	ND	ND	ND	-	-
Metribuzin	µg/L	ND	ND	ND	-	-
Propachlor	µg/L	ND	ND	ND	-	-
Simazine	µg/L	ND	ND	ND	4	-
Trifluralin	µg/L	ND	ND	ND	-	-
Disinfectant Residual						
Ammonia, Free (NH_3-N)	mg/L	0.00	0.04	0.28	-	-
Ammonia, Total (NH_3-N)	mg/L	0.05	0.56	0.95	-	-
Chlorine, Free	mg/L	0.00	0.01	2.46	4.0	-
Chlorine, Total	mg/L	0.92	2.48	3.96	4.0	-
Dichloramine	mg/L	0.00	0.15	2.54	-	-
Monochloramine	mg/L	0.09	2.21	3.65	-	-
Microbiological Contaminants						
Coliform, total (3)	A=0, P=1	0	0	1	< 5% P	-
E coli	A=0, P=1	0	0	0	< 5% P	-
Heterotrophic Plate Count	cfu/100 mL	0.00	0.65	7.00	-	-

Notes:
(1)"ND" indicates that the concentration was non-detect or below the method detection limit.
(2)AL = Action Level. ALs for lead and copper are monitored in the distribution system. Finished water quality data presented in this table is not for Lead and Copper Rule compliance monitoring.
(3)For total coliform measurements, "P" indicates the presence of coliforms and "A" indicates an absence of coliforms in the sample collected.

5.7 Water Quality Trends

5.7.1 Temperature and pH

The temperature of raw water supplied to the East Plant varies seasonally, typically ranging from 8 degrees Celsius (°C) to 14°C in winter months and 20°C to 25°C in late summer to early fall. The temperature of raw water supplied to the West Plant typically ranges from 12°C to 16°C in the winter and 18°C to 23°C in late summer to early fall and is generally less impacted by seasonal variations. Between July and October, elevated water temperatures can contribute to conditions that promote biological regrowth in the distribution system. The temperature of raw water supplied by the vertical wells is typically 2-3°C lower than the temperature of water supplied by the HCWs. In recent years, Lincoln Water Systems has augmented the amount of water supplied from the vertical wells and treated through the West Plant to reduce the water temperature in the distribution system. Figure 5-3 demonstrates seasonal variations in raw water temperature supplied to the East and West Plant.

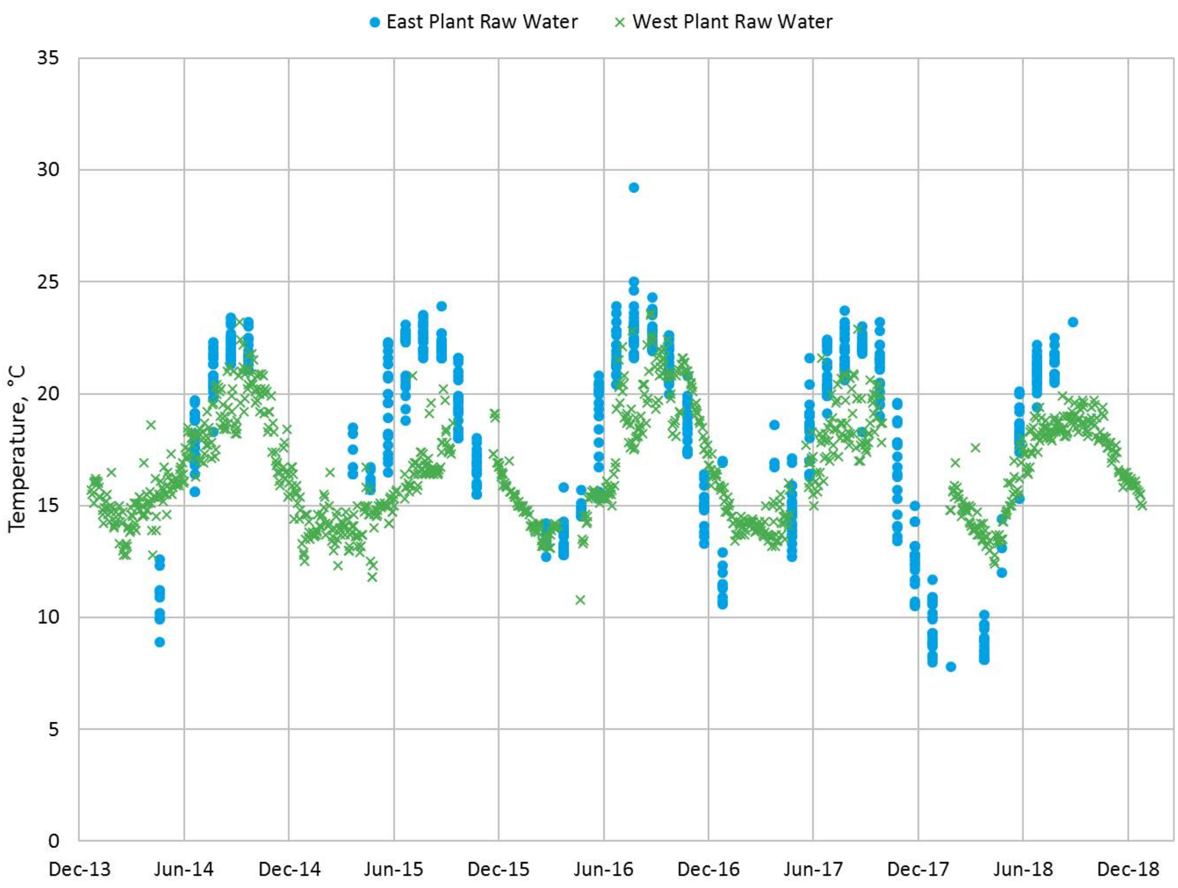

Figure 5-3 Raw Water Temperature for Samples Collected from the East and West Plant from January 2014 to December 2018

The pH of raw water supplied to the East Plant typically ranges from 7.6 to 8.0, while the West Plant raw water pH typically ranges from 7.4 to 7.6. Figure 5-4 provides a summary of the East and West Plant raw water pH.

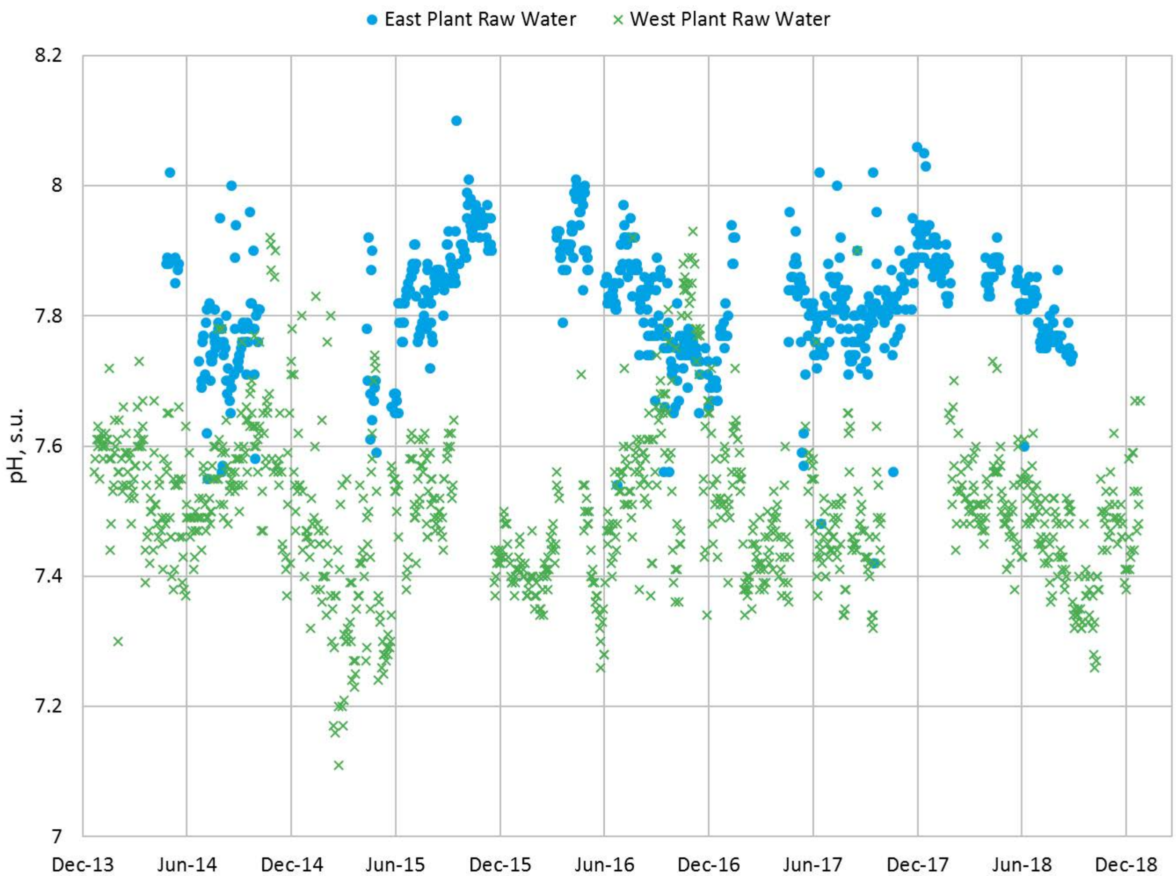

Figure 5-4 Raw Water pH for Samples Collected from the East and West Plant from January 2014 to December 2018

5.7.2 Nitrate and Nitrite

Nitrate and nitrite are naturally occurring in ground water supplies. Under the National Primary Drinking Water Regulations (NPDWR), nitrate and nitrite have a maximum contaminant limit (MCL) of 10 mg/L as N and 1 mg/L as N, respectively. Compliance with the MCL is monitored at the point of entry to the distribution system. The raw water supplied to the East Plant has significantly higher nitrate concentrations than the raw water supplied to the West Plant. The concentration of nitrate in the East Plant raw water typically ranges from 1 to 4 mg/L. Based on data collected from Years 2014 to 2019, the maximum concentration recorded at the East Plant was below 50 percent of the MCL. Figure 5-5 demonstrates the concentration of nitrate measured in the East and West Plant raw water.

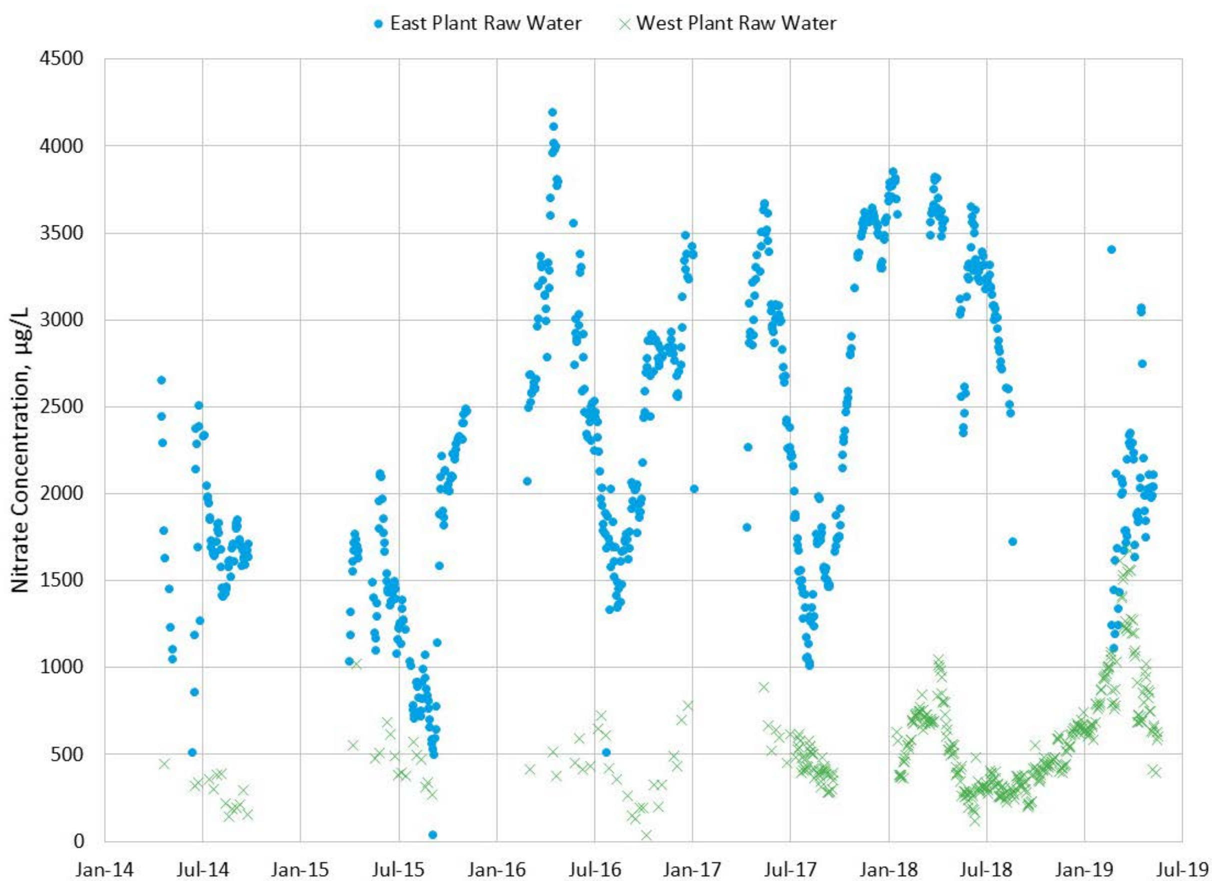

Figure 5-5 Raw water nitrate concentration for samples collected from the East and West Plant from January 2014 through July 2019.

Figure 5-6 demonstrates the concentration of nitrite in the raw water supplied to the East and West Plants. Both water supplies experience significant variations in nitrite concentration with spikes typically occurring between July and August. Based on data collected from Years 2014 to 2019, the maximum concentration recorded at the East Plant was approximately 25 percent of the MCL. In some parts of the distribution system, the concentration of nitrite tends to increase between the months of August and October due to nitrification. Discussion of nitrification impacts on distribution system water quality is provided in Chapter 7.

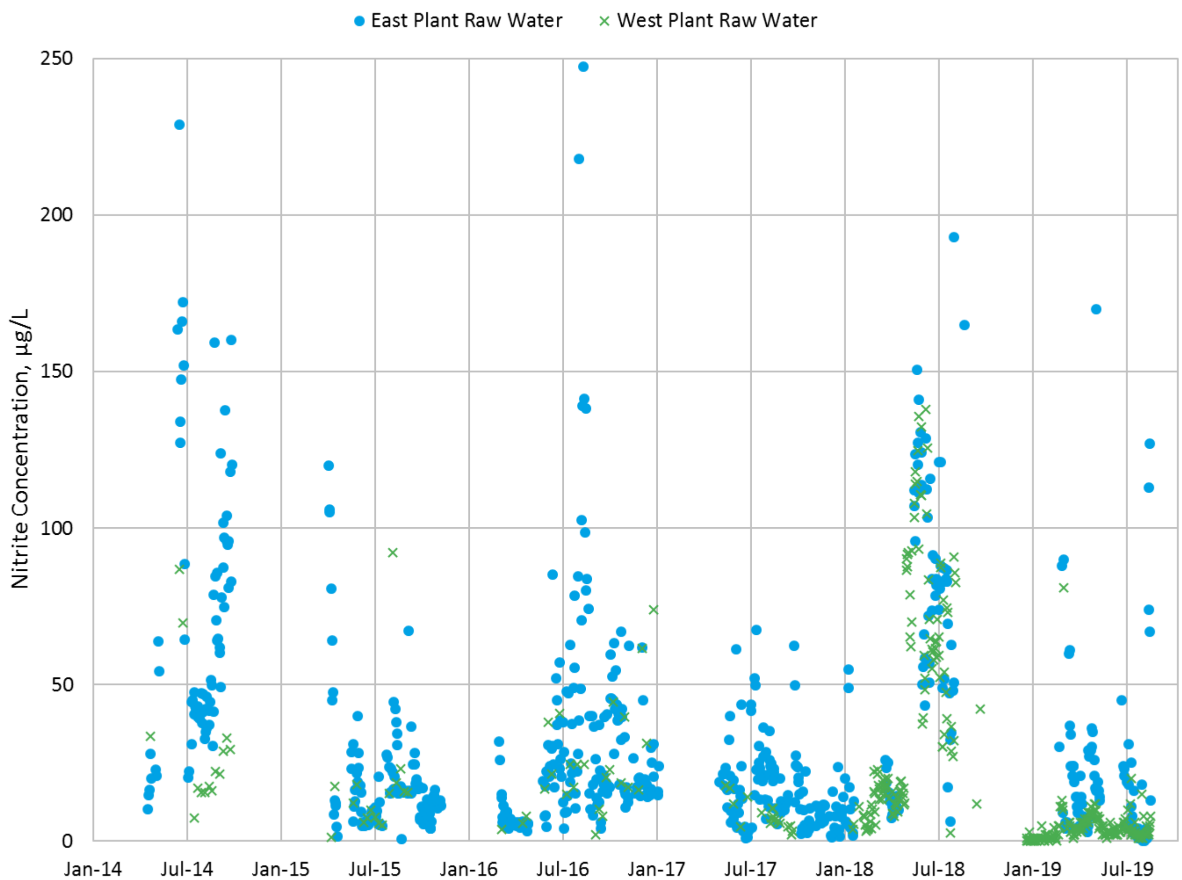

Figure 5-6 Raw water nitrite concentration from samples collected from the East and West Plant from January 2014 through August 2019

5.7.3 Manganese

Figure 5-7 demonstrates the concentration of manganese in the raw water supplied to the East and West Plants based on samples collected between January 2017 and August 2019. The concentration of manganese in the raw water has typically remained below 150 µg/L. Higher concentrations of manganese have been observed in the vertical wells that supply water to the West Plant than in the HCWs that supply water to the East Plant. Manganese concentrations observed in 2019 demonstrated an average raw water concentration of 43 µg/L and 51 µg/L in the East and West Plant, respectively. These concentrations are consistent with historical water quality data observed from 2005 to 2011.

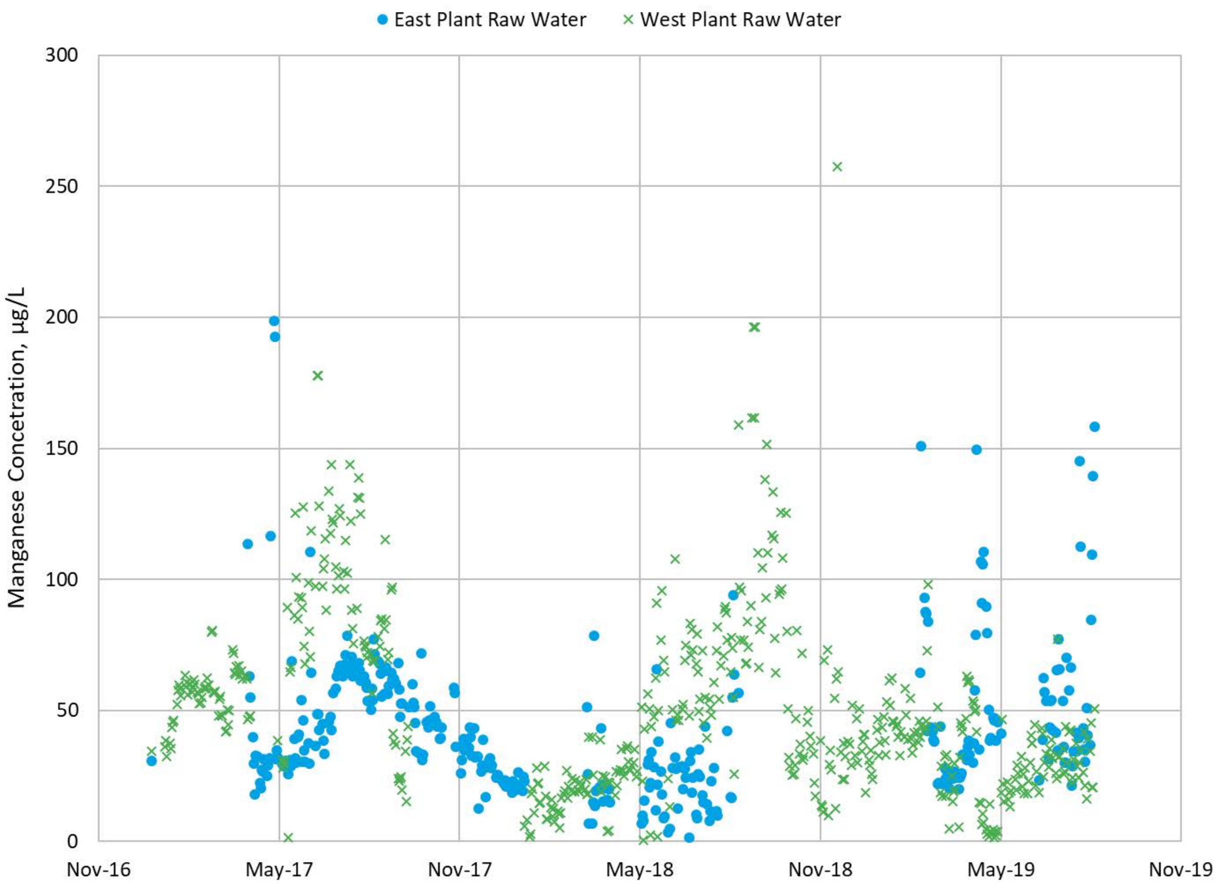

Figure 5-7 **Raw Water Manganese Concentration from the East and West Plant from January 2017 through August 2019**

Manganese is removed through oxidation and filtration, where the East Plant utilizes ozone for oxidation and the West Plant utilizes chlorine. The USEPA has a non-enforceable secondary MCL of 50 µg/L for manganese. LWS has a treatment goal of less than 10 µg/L of manganese in the finished water. Figure 5-8 provides a summary of the concentration of manganese in the East and West Plant finished water from January to August 2019. The West Plant was able to meet this goal 100 percent of the time, and the East Plant was able to meet this goal 96 percent of the time.

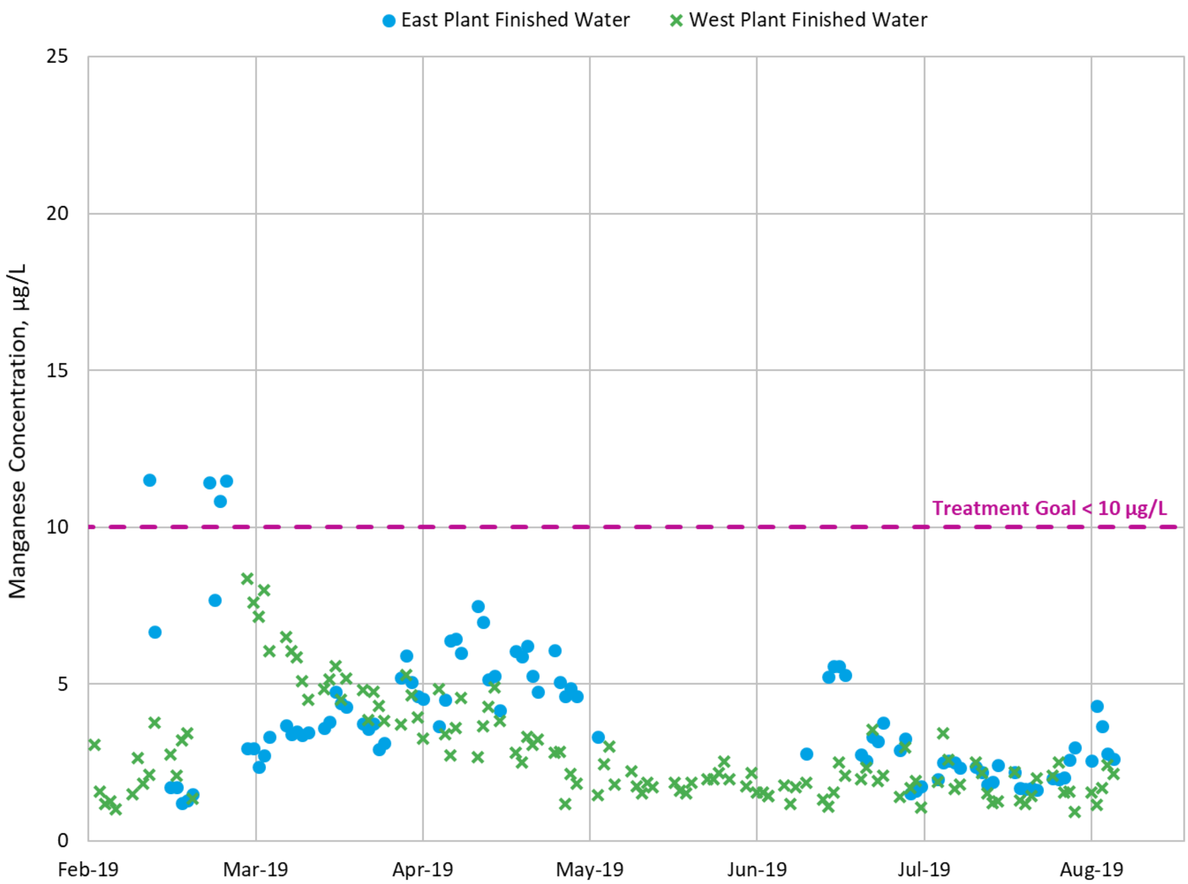

Figure 5-8 Finished Water Manganese Concentration from the East and West Plant from January to August 2019

5.7.4 Arsenic

Arsenic is a naturally occurring inorganic chemical, which is regulated under the NPDWR with an MCL of 10 µg/L. The Arsenic Rule requires monitoring at the point of entry based on the running annual average (RAA) of quarterly samples with provisions for reduced monitoring on an annual basis.

Figure 5-9 provides the concentration of arsenic in raw water samples collected from the East and West Plants from January 2017 through August 2019. The water quality data presented in this figure was collected from the plant's laboratory sampling program, which goes beyond the annual sampling requirements for regulatory compliance.

The concentration of arsenic has historically been higher in the HCWs servicing the East Plant, which have an average and maximum concentration of 7.5 and 9.7 µg/L, respectively. In the West Plant raw water, the average and maximum arsenic concentrations are 6.72 µg/L and 8.76 µg/L, respectively. As with atrazine, LWS has had to implement wellfield management practices to maintain compliance with the MCL, which limits the use of HCWs and the East Plant. While LWS has maintained regulatory compliance for arsenic, the concentration of arsenic in the raw water supplied from the HCWs appears to be increasing over time, trending upward towards the MCL of 10 µg/L.

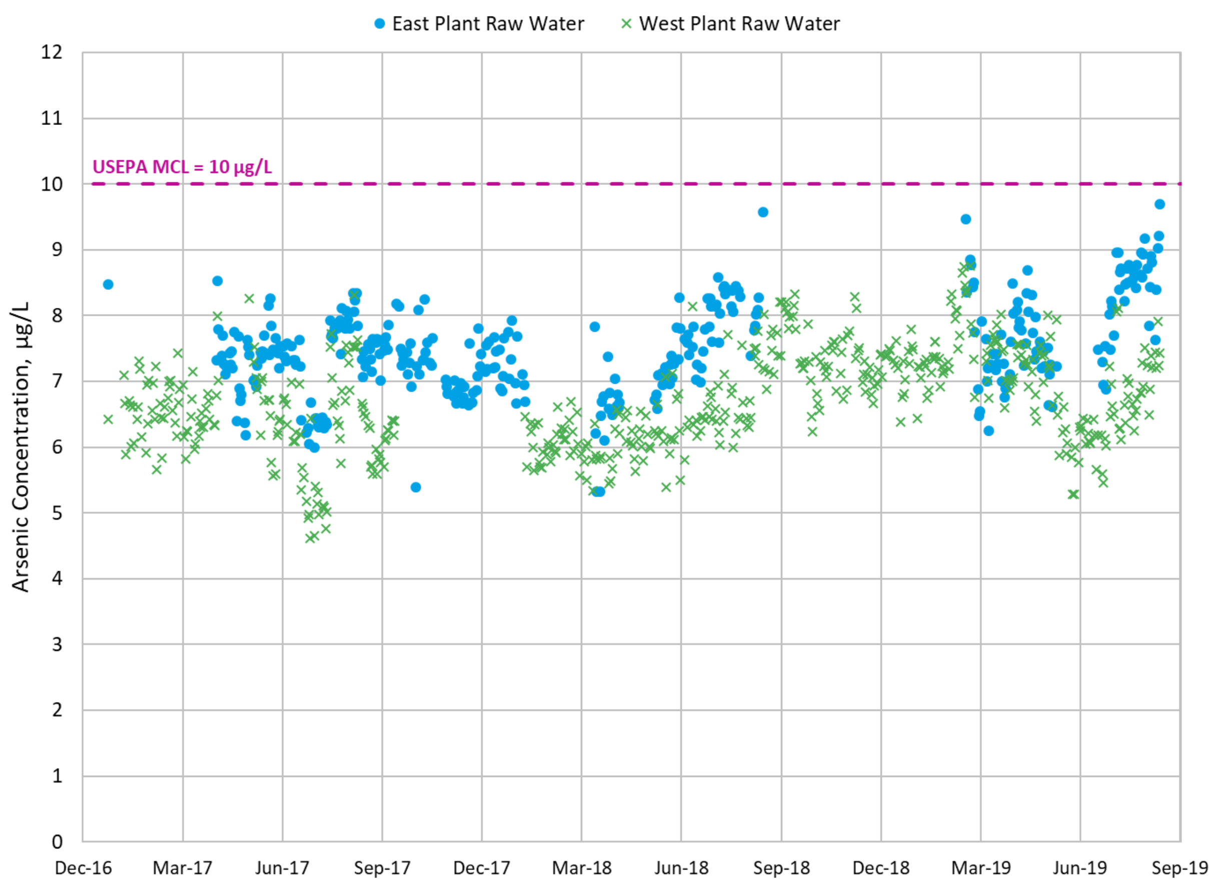

Figure 5-9 Raw Water Arsenic Concentration from the East and West Plant from January 2017 through December 2018

In 2016, LWS conducted a study to evaluate treatment alternatives for arsenic removal to meet proposed finished water quality goals of 8 µg/L, 4 µg/L and non-detect levels. The evaluation focused on arsenic removal through enhanced coagulation and provided a high-level comparison of alternative arsenic treatment technologies, including adsorption through activated alumina and iron oxide coated sand, ion exchange, and reverse osmosis from a high-level perspective.

From this evaluation, enhanced coagulation with ferric chloride was identified as the preferred alternative. The ferric chloride dose required ranged from 5 mg/L to 15 mg/L, depending on the influent arsenic concentration and finished water quality goal. Pilot testing was conducted to evaluate the impacts on filter performance from incorporating a coagulant feed with direct filtration. During the pilot, the filters experienced significant reductions in filter run time, indicating the need for a coagulation, flocculation and sedimentation process upstream of filtration in order to accommodate the ferric chloride addition for arsenic removal. The enhanced coagulation process could potentially produce residuals with high concentrations of arsenic that may require additional treatment or need to be hauled away for disposal in a hazardous waste landfill. Further evaluation should be conducted to determine the preferred approach to residuals management.

Alternatively, granular ferric hydroxide (GFH) media adsorption, which was ruled out in the previous study, may prove to be a viable alternative. This system would consist of vertical pressure vessels filled with GFH media, designed for a portion of the total plant flow depending on the arsenic finished water quality goal. Media replacement frequency would depend on the finished water quality goal for arsenic. When GFH media is exhausted, it is typically hauled away and replaced with new media. In most cases, the exhausted media is disposed of in a landfill as a non-hazardous waste, provided that the toxicity characteristic leaching procedure (TCLP) test indicates that arsenic leaching potential is less than 5 mg/L.

Given the relatively high concentrations of arsenic in the HCWs and continued expansion of water supplied from HCWs, it is recommended that LWS conduct further evaluations of viable arsenic removal technologies to determine the most cost-effective treatment approach.

5.7.5 Atrazine

Atrazine is a widely used herbicide regulated under the NPDWR by USEPA and has a maximum contaminant limit (MCL) of 3 µg/L. The Platte River experiences elevated levels of atrazine in the late spring/early summer due to runoff from agricultural fields. Between May and July, the average concentration of atrazine in the Platte River is typically 6 µg/L, with spikes as high as 10 to 15 µg/L.

Figure 5-10 demonstrates the concentration of atrazine in the raw water supplied to the East and West Plants from February 2013 to August 2018. The figure shows that the concentration of atrazine in both raw water supplies has remained relatively consistent over the past five years. Since the West Plant is supplied from ground water wells, the raw water delivered to the West Plant is less subject to these spikes in atrazine. The average and maximum atrazine concentrations in the West Plant raw water are 0.27 µg/L and 0.46 µg/L, respectively.

Since the East Plant relies on horizontal collector wells (HCWs), the raw water delivered to the East Plant does experience seasonal spikes in atrazine, as demonstrated in the figure. The concentration of atrazine in the East Plant raw water is considerably lower than the concentration observed in the river due to river bank filtration. The average and maximum atrazine concentrations in the East Plant raw water are 0.51 µg/L and 1.95 µg/L, respectively.

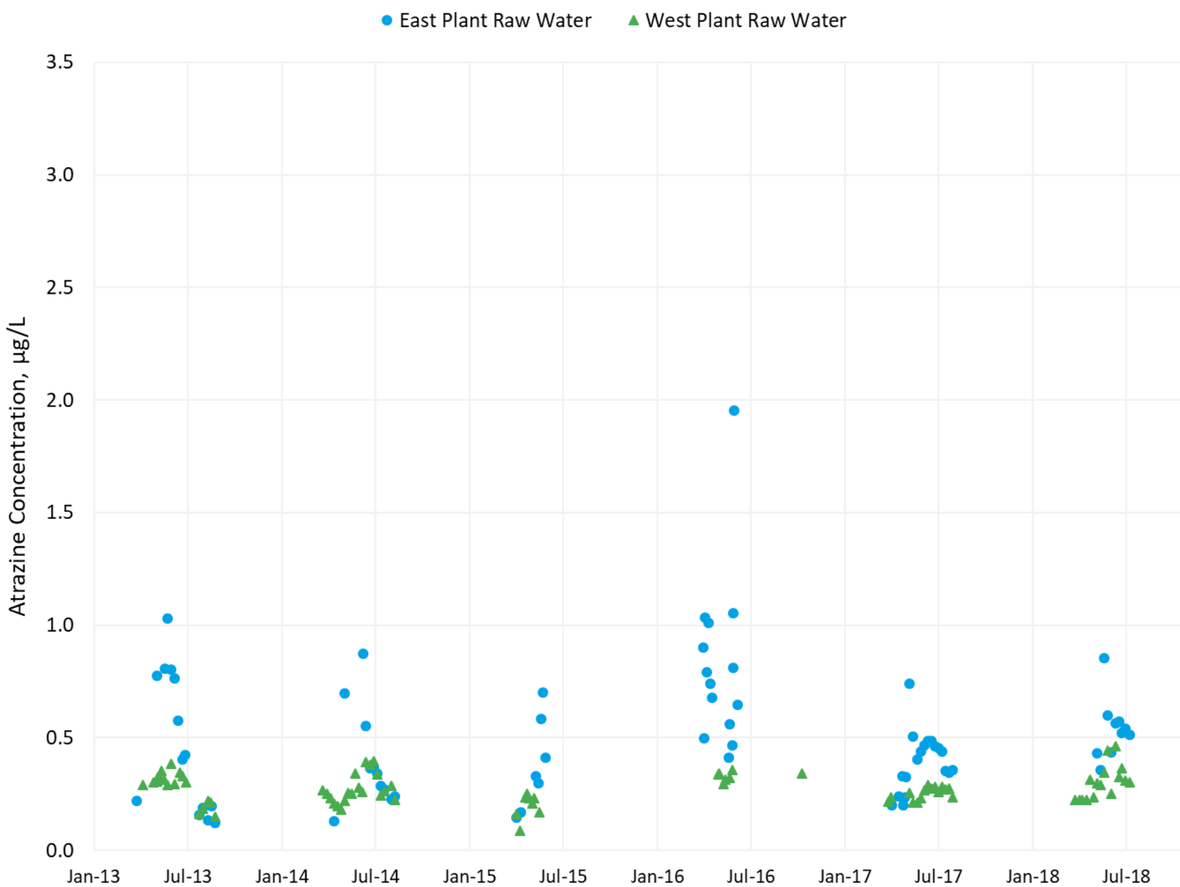

Figure 5-10 Raw Water Atrazine Concentration from the East and West Plant from February 2013 through August 2018

Given the relatively high concentrations of atrazine in the Platte River, LWS has undertaken atrazine management practices during the spring and summer when agricultural runoff contributes to elevated atrazine levels. Since the HCWs are influenced by water quality in the river, the concentration of atrazine is higher in the HCWs than in the groundwater supplied from the vertical wells. The East Plant treatment process includes ozonation, which reduces the concentration of atrazine in the finished water by approximately 50 percent. However, in order to ensure compliance with the MCL, LWS has had to implement wellfield management practices, by which they limit the use of the HCWs during periods of elevated atrazine in the river and utilize only the West Plant for drinking water supply. Based on compliance monitoring data presented in Figure 5-11, the concentration of atrazine in the East and West Plant finished water has been maintained below 0.5 µg/L since Year 2014.

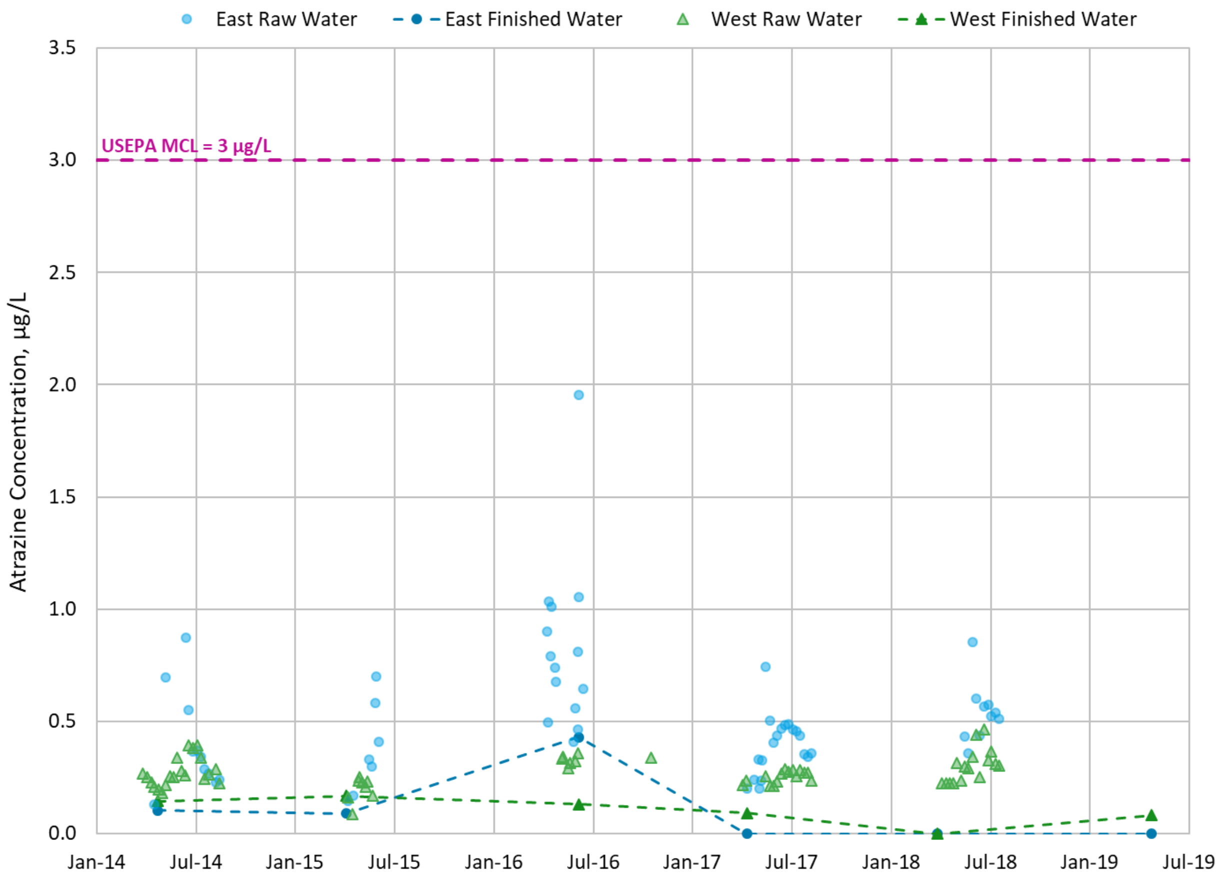

Figure 5-11 East and West Plant Finished Water Atrazine Concentration from January 2014 to July 2019

5.7.6 Total Organic Carbon

Total organic carbon (TOC) is used as a surrogate measure for the amount of natural organic matter (NOM) present in water. NOM reacts with chlorine to form regulated disinfection byproducts (DBPs), including total trihalomethanes (TTHM) and the five regulated haloacetic acids (HAA5). TOC management practices are typically used to reduce the concentration of TOC in the finished water and control the DBP formation in the distribution system based on the TOC removal requirements established in the Stage 1 Disinfectants and Disinfection Byproduct Rule. LWS is not required to meet the TOC removal requirements due to their source water characteristics and ability to maintain TTHM and HAA5 concentrations of less than 40 µg/L and 30 µg/L, respectively.

Figure 5-12 provides a summary of the East and West Plant finished water TOC concentrations from January 2014 to April 2019. The West Plant finished water TOC ranges from 1.70 to 3.70 mg/L with an average concentration of 2.16 mg/L; whereas the East Plant finished water TOC ranges from 0.63 mg/L to 4.68 mg/L with an average concentration of 2.71 mg/L. Historical data indicates that water supplied from the HCWs typically has higher concentrations of TOC than water supplied from the vertical wells and is subject to greater variability due to seasonal changes on the Platte River. As the City continues to expand the use of HCWs for raw water supply, impacts on TOC and DBP management should be evaluated.

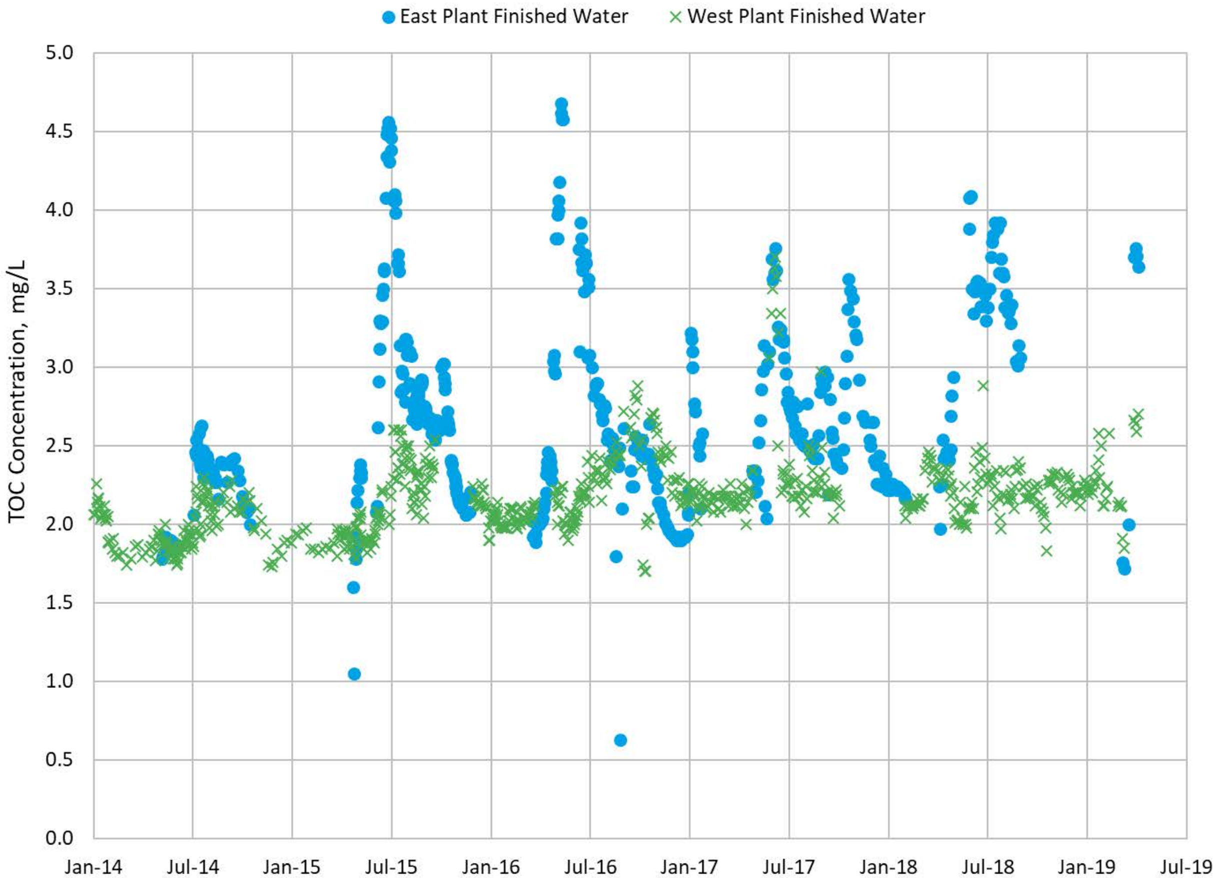

Figure 5-12 East and West Plant Finished Water TOC Concentration from January 2014 to April 2019

5.7.7 Biological Stability

Assimilable organic carbon (AOC) is a parameter used to measure the biological stability of water and can be used as an indicator for potential bacterial regrowth in the distribution system. AOC represents the amount of carbon that is readily taken up by microorganisms for bacterial growth and is measured in μg acetate carbon per liter. Biodegradable dissolved organic carbon (BDOC) can also be used to assess biological stability. BDOC is measured as the net change in DOC consumed by biologically active sand or biofilm on a borosilicate glass bead column.

LWS monitored the concentration of AOC in the East and West Plant finished water from January 2004 to July 2009. Results from AOC monitoring are shown in Figure 5-13. During this period, the average and maximum concentration of AOC in the East Plant finished water was 154 μg/L and 350 μg/L, respectively. The average and maximum concentration of AOC in the West Plant finished water was 80 μg/L and 180 μg/L, respectively. The East Plant is subject to higher concentrations of AOC due to the ozonation process, which oxidizes organic compounds into smaller, more readily biodegradable dissolved organic compounds.

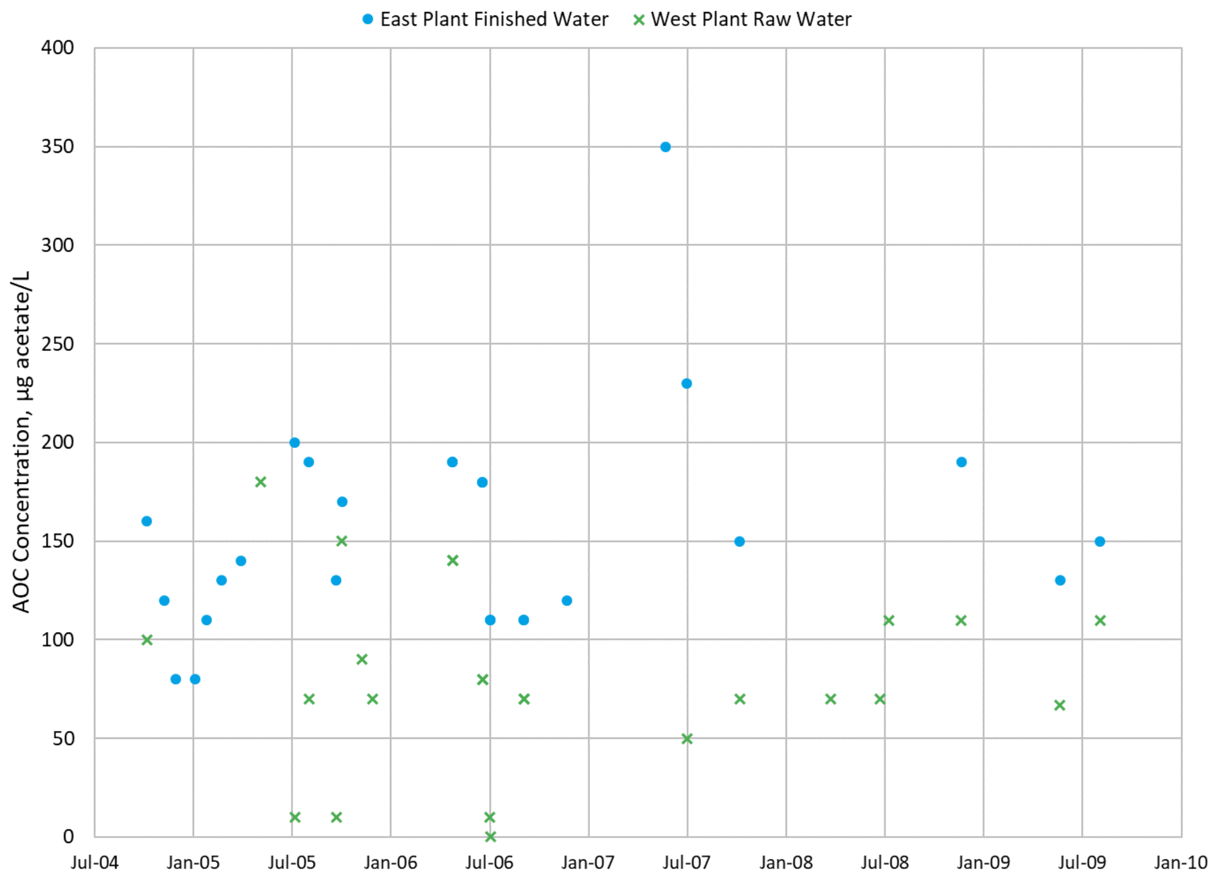

Figure 5-13 East and West Plant Finished Water AOC Concentration from July 2004 to July 2009

The AWWARF Report No. 90794 – Investigation of Biological Stability in the Distribution System defines thresholds for various water quality parameters affecting biological stability. Specifically, the article focuses on water temperature, disinfectant type and residual, AOC and BDOC. Based on a study with water quality analysis from 64 utilities across the United States, it was found that systems with the following finished water quality conditions were more likely to have coliform occurrences.

- Temperature > 15°C
- Total chlorine residual < 1.0 mg/L
- AOC > 100 µg/L
- BDOC > 0.3 µg/L

The research report further categorizes low, moderate and high concentrations of AOC and BDOC as demonstrated in Table 5-29. The impact of BDOC on finished water stability is temperature-dependent. When water temperature is less than 15°C, higher concentrations of BDOC (up to 0.3 µg/L) may sufficiently prevent bacterial regrowth. However, under warmer conditions at temperatures greater than 20°C, maintaining a concentration of BDOC less than 0.15 µg/L is recommended for preventing bacterial regrowth. Based on these definitions, the East Plant finished water has moderate to high AOC, whereas the West Plant finished water has low to moderate AOC.

Table 5-29 Categorization of Low, Moderate and High Concentrations of AOC and BDOC

Category	AOC, µG/L	BDOC, µG/L
Low	< 50	< 0.15
Moderate	50-150	0.15-0.3
High	> 150	> 0.3

5.7.8 Disinfectant Residual

LWS has historically targeted a total chlorine residual of 2.5 mg/L as Cl_2 in the finished water from both East and West Plants. LWS increased their target finished water total chlorine residual to 3.0 mg/L in December 2017 and eventually to 3.5 mg/L in 2019, as a means for controlling nitrification. Figure 5-14 shows the total chlorine residual in the plant finished water from January 2014 through January 2019.

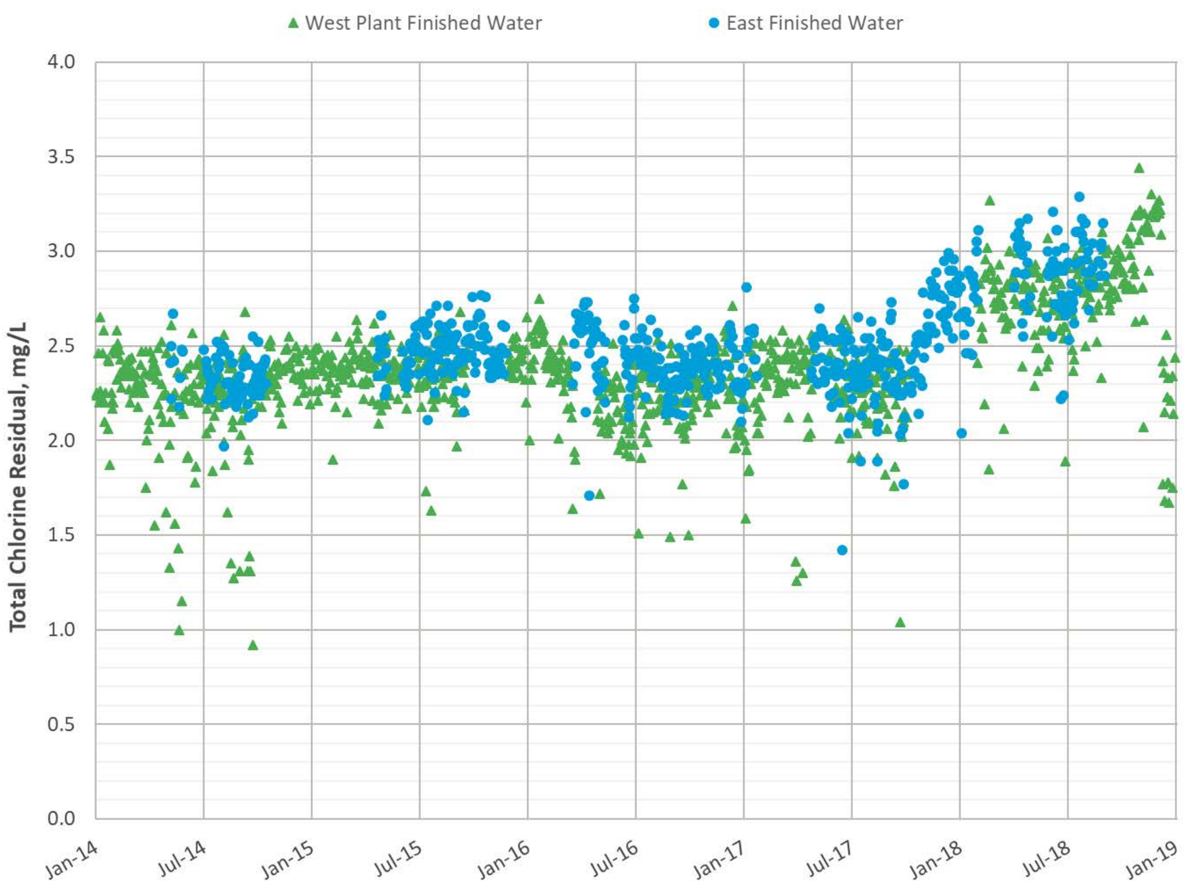

Figure 5-14 East and West Plant Finished Water Total Chlorine Residual from January 2014 to January 2019

Chloraminated systems rely on the breakpoint curve to drive formation of monochloramines. Based on the breakpoint curve, it is typically desirable to operate with a target chlorine-to-ammonia (Cl_2-NH_3) mass ratio of 3 to 5 with most plants adopting a narrower target of 4.0 to 4.5. Operating on the left of this ratio (Cl_2-NH3 ratio ≤ 3) results in excess free ammonia in the finished water, which increases the potential for nitrification. Operating on the right of this ratio (Cl_2-NH_3 ratio > 5) results in the formation of undesirable chloraminated species such as dichloramine and trichloramine, which leads to objectionable taste/odor and less stable residual (faster degradation of total chlorine residual). Figure 5-15 and Figure 5-16 show the relationship between total chlorine, monochloramine and dichloramine for the East and West Plant finished water, respectively. As demonstrated in the figures, LWS is primarily forming monochloramine, which on average makes up approximately 90 percent of the total chlorine residual.

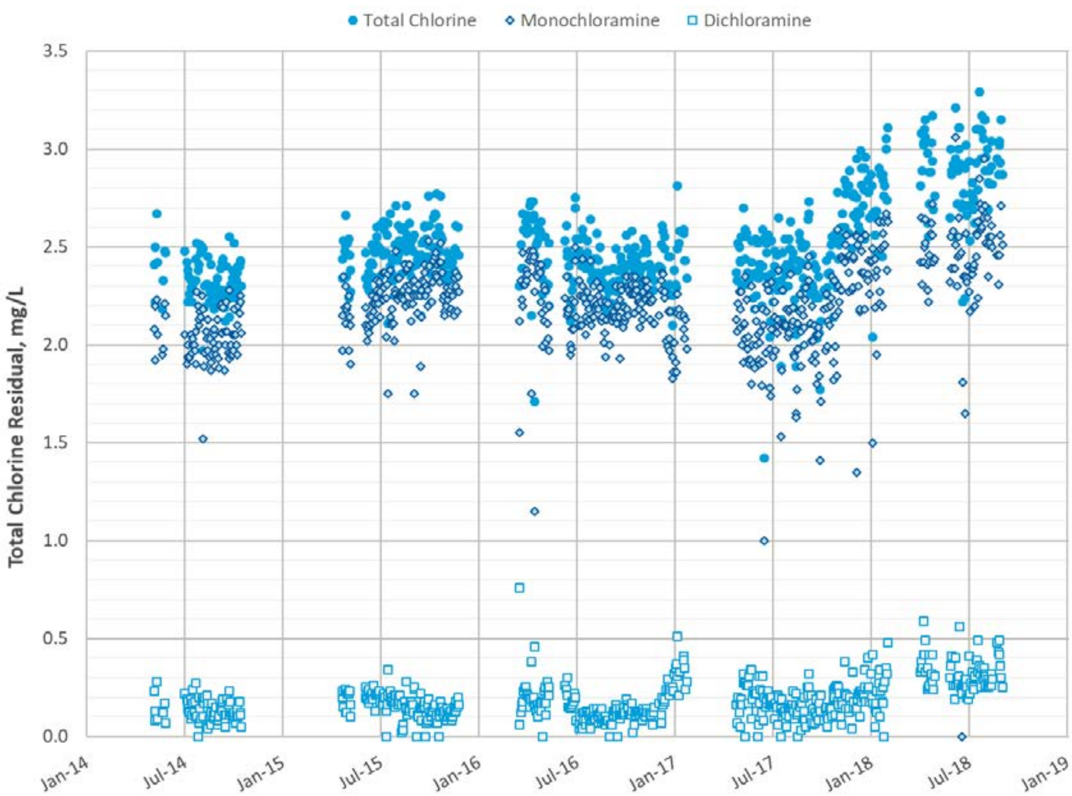

Figure 5-15 East Plant Finished Water Chloramine Speciation from January 2014 to August 2018

Figure 5-16 West Plant Finished Water Chloramine Speciation from January 2014 to January 2019

It is generally recommended to maintain finished water free ammonia concentrations of less than 0.1 mg/L. Based on the finished water quality data provided by LWS, the average concentration of free ammonia from the East and West Plant finished water is 0.08 mg/L and 0.04 mg/L, respectively. This demonstrates a healthy relationship between chlorine residual and ammonia dosing. The high percentage of total chlorine present as monochloramine and low concentrations of free ammonia helps reduce the potential for nitrification.

5.8 Regulatory Summary

This section provides an overview of existing regulations, contaminants undergoing regulatory determination, and potential future regulatory changes. Based on an analysis of the water quality data received and understanding of the plant's treatment systems, it appears that the LWS Ashland plants are in compliance with applicable rules and regulations.

5.9 Existing Regulations

5.9.1 Surface Water Treatment Rule

The Surface Water Treatment Rule (SWTR), published in Year 1989, was the first rule passed by EPA to protect the public against pathogens. Subsequent rules have been passed to supplement the SWTR primarily in response to discovery of DBPs and discovery that some pathogens, such as *Cryptosporidium*, are highly resistant to traditional disinfectants.

The SWTR established MCLGs of zero for *Giardia*, viruses, and Legionella. The following treatment techniques were required to protect against these pathogens:

- Filtration, unless specific avoidance criteria are met.
- Maintenance of a disinfectant residual in the distribution system.
- Removal or inactivation of 99.9 percent (3-log) *Giardia* and 99.99 percent (4-log) viruses.
- Maximum allowable turbidity in the combined filter effluent (CFE) of 5 nephelometric turbidity units (NTU) and 95th percentile CFE of 0.5 NTU or less for plants with conventional treatment or direct filtration.
- Watershed protection and source water quality requirements for unfiltered PWSs.
- The SWTR established two criteria for demonstrating maintenance of a disinfectant residual:
- A minimum residual of 0.2 mg/L entering the distribution system.
- A detectable residual throughout the distribution system.

Disinfection requirements specified in the SWTR are summarized in Table 5-30. Disinfection requirements are based on pathogen removal credits given for filtration and inactivation credits given for disinfection. Conventional treatment receives 2.5-log removal credit for *Giardia* and 2.0-log removal credit for viruses. Disinfection is required to achieve the remaining 0.5-log *Giardia* inactivation and 2-log virus inactivation.

The CT method is used to determine disinfection credits achieved during treatment. In this method, CT is defined as the product of C, the residual disinfectant concentration in mg/L, and T10, the detention time in minutes corresponding to the time for which 90 percent of the water has been in contact with at least the residual concentration. Ratios of T10 to the theoretical hydraulic detention time, T, can be determined with tracer tests. In the absence of tracer test results, EPA provides guidelines for T10/T ratios based on the extent of baffling in a basin. The T10/T ratio is often referred to as a "baffling classification."

Table 5-30 Log Removal/Inactivation Credits and Requirements Under the 1989 SWTR

Process	*Giardia* Cysts	Viruses
Total log removal/inactivation required	3.0	4.0
Conventional sedimentation/filtration credit	2.5	2.0
Direct filtration credit	2.0	1.0
Slow sand filtration credit	2.0	2.0
Diatomaceous earth credit	2.0	1.0

5.9.2 Total Coliform Rule

The Total Coliform Rule (TCR) was published in Year 1989 to improve public health by reducing fecal pathogens in drinking water to minimal levels. The TCR requires testing representative samples across the distribution system for total coliforms at a prescribed frequency. Any positive test result triggers repeat sampling and testing for *Escherichia coli (E. coli)*.

Compliance with the TCR is based on the presence or absence of total coliforms as determined each calendar month. Specific requirements are as follows:

- Total coliform samples must be collected at locations representative of the distribution system according to a written sampling plan.
- Samples must be collected at regular time intervals throughout the month. Monitoring frequency depends on population. The City of Lincoln population was 261, 796 in 2010 and is projected to grow to 371,700 by 2040.
- Systems serving 220,001 to 320,000 people must sample at least 150 times per month.
- Systems serving 320,001 to 450,000 people must sample at least 180 times per month.
- If a sample tests positive for coliforms, a set of repeat samples must be collected within 24 hours. The repeat set must include the original sample location and one sample each within five service connections upstream and downstream of the original sample.
- If any repeat sample tests positive for total coliforms, another set of repeat samples must be collected.
- Any sample that tests positive for coliforms must also be analyzed for fecal coliforms or *E. coli*.
- A monthly MCL violation is triggered if more than 5 percent of samples test positive for total coliforms. Any monthly MCL violation must be reported to the state no later than at the end of the next business day and must be reported to the public within 30 days.
- Any positive repeat result for fecal coliform or *E. coli* signifies an acute MCL violation. An acute MCL violation must be reported to the state no later than at the end of the next business day and must be reported to the public within 24 hours. Acute MCL violation is also triggered if any routine sample tests positive for fecal coliform of *E. coli* followed by a total coliform-positive repeat sample.

5.9.3 Revised Total Coliform Rule

EPA published the Revised Total Coliform Rule (RTCR) in the Federal Register on February 13, 2013 and minor corrections on February 26, 2014. The intent of the RTCR is to increase public health protection through the reduction of potential pathways of entry for fecal contamination into the distribution system. The RTCR establishes a maximum contaminate level (MCL) for *E. coli* and uses *E. coli* and total coliforms to initiate a “find and fix” approach to address fecal contamination that could enter into the distribution system. *E. coli* is considered to be a more specific indicator of fecal contamination and the potential presence of harmful pathogens than total coliform bacteria, the RTCR reflects a shift in compliance requirements that focuses more on the presence/absence of *E. coli* in the distribution system. Monitoring requirements remained the same, but under the RTCR, a system was required to test any total coliform-positive sample for *E. coli*. Any *E. coli*-positive sample must be reported to the state no later than the end of the next business day. Systems with violations are required to conduct assessments to find and fix the source of contamination. All public water systems (PWSs), except aircraft PWSs subject to the Aircraft Drinking Water Rule, must comply with the RTCR starting April 1, 2016.

5.9.4 Interim Enhanced Surface Water Treatment Rule

The IESWTR was developed in conjunction with the Stage1 Disinfectants and Disinfection Byproduct Rule (Stage 1 DBPR) in Year 1998. The purpose of the rule was to increase protection against microbial pathogens, particularly *Cryptosporidium*. Key provisions of the IESWTR are as follows:

- Established an MCLG of zero for Cryptosporidium.
- Set 2-log Cryptosporidium removal requirement for systems that filter.
- Lowered combined filter effluent (CFE) turbidity requirements to 1.0 NTU maximum, and 0.3 NTU at the 95th monthly percentile.
- Required individual filter turbidity monitoring continuously (every 15 minutes).
- Established provisions for disinfection benchmarking.
- Added Cryptosporidium to the definition of GWUDI and in the watershed control requirements.
- Required covers on finished water reservoirs.
- Required sanitary surveys for all systems to be conducted by the state every 3 years.

Monitoring of *Cryptosporidium* has indicated non-detect levels of *Cryptosporidium* in the raw water supplied to the East and West Plants.

5.9.5 Filter Backwash Recycle Rule

The Filter Backwash Recycling Rule was published in 2001 to improve protection against microbial contaminants by establishing requirements for recycling practices. This is not applicable to LWS at this time since they do not currently practice backwash recycle.

The Filter Backwash Recycling Rule has the following three requirements:

- A system must notify the state in writing about its recycle practices if it recycles one of the aforementioned regulated flows.
- Regulated recycle flows must be returned through all processes of the system's conventional treatment.
- Recordkeeping is required for recycle streams.
- The regulated recycle streams are as follows:
- Spent filter backwash.
- Thickener supernatant.
- Liquids from dewatering processes.

5.9.6 Long-Term 2 Enhanced Surface Water Treatment Rule

The Long Term 2 Enhanced Surface Water Treatment Rule (LT2ESWTR) was published in Year 2006 to provide further protection against *Cryptosporidium* and other microbial pathogens. It was intended to supplement previous surface water treatment regulations and to increase treatment requirements for systems with higher *Cryptosporidium* risk.

- Requires monitoring to determine an average Cryptosporidium level.
- An initial 2 years of monthly monitoring is required followed by a second round of monitoring 6 years later to determine if source water conditions have not changed.
- Monitoring results are used to assign the system into one of four bin classifications based on Cryptosporidium risk.
- Additional treatment requirements for Cryptosporidium are required for high risk bin classifications.
- Requires PWSs with uncovered reservoirs to cover the reservoir or provide treatment to achieve 4-log virus, 3-log Giardia, and 2-log Cryptosporidium inactivation, removal, or both.

Since *Cryptosporidium* was found to be non-detect in raw water samples collected from the East and West Plant, both facilities are categorized under Bin 1 and do not require additional treatment for *Cryptosporidium*.

5.9.7 Stage 1 Disinfectant/Disinfection Byproduct Rule

The Stage 1 DBPR was published in Year 1998 to reduce potential health risk from exposure to DBPs. The Stage 1 DBPR prescribed MCLs for DBPs and set maximum residual disinfectant levels (MRDLs) for disinfectants. Stage 2 DBPR also set requirements for total organic carbon (TOC) removal in enhanced coagulation and enhanced softening.

The Stage 1 DBPR set MCLs for two groups of organic DBPs: total trihalomethanes (TTHMs) and five haloacetic acids (HAA5); and for two inorganic DBPs: chlorite and bromate, as shown in Table 5-31. Compliance with TTHM and HAA5 MCLs is based on the running annual average (RAA) of samples from all monitoring locations across the distribution system.

Table 5-31 Maximum Contaminant Levels for Disinfection Byproducts in the Stage 1 DBPR

Disinfection byproducts	MCL (MG/L)
Total trihalomethanes (TTHM)	0.080
Haloacetic acids (HAA5)	0.060
Chlorite	1.0
Bromate	0.010

MRDLs were set for chlorine, chlorine dioxide, and chloramines as shown in Table 5-32.

Table 5-32 Maximum Residual Disinfectant Levels in the Stage 1 DBPR

Disinfectant	MRDL (MG/L)
Chlorine	4.0
Chlorine dioxide	0.80
Chloramines	4.0 (as Cl_2)

Table 5-33 summarizes the TOC removal requirements based on source water TOC and alkalinity. LWS is not required to meet the TOC removal requirements listed in the table, as their system meets alternative compliance criteria specified under 40 CFR 141.135(a)(2). Alternative compliance criteria is met through source water TOC less than 4.0 mg/L and source water alkalinity greater than 60 mg/L as $CaCO_3$, with TTHM and HAA5 maintained at less than 50 percent of the MCLs.

Table 5-33 Percent TOC Removal Required by Enhanced Coagulation and Enhanced Softening in the Stage 1 DBPR

Source Water TOC (mg/L)	Source Water Alkalinity (mg/L as $CaCO_3$)		
	0 -60	>60 - 120	> 120
>2.0 – 4.0	35%	25%	15%
>4.0 – 8.0	45%	35%	25%
>8.0	50%	40%	30%

5.9.8 Stage 2 Disinfectant/Disinfection Byproduct Rule

The Stage 2 DBPR tightened compliance monitoring requirements for TTHM and HAA5 by requiring compliance at each monitoring site in the distribution system.

Each system was required to conduct an Initial Distribution System Evaluation (IDSE) to identify locations with high DBP concentrations. IDSE results were used to determine sampling sites for Stage 2 DBPR compliance. Systems were required to begin Stage 2 DBPR monitoring in April 2012.

MCLs for TTHM and HAA5 remained at the Stage 1 DBPR levels, but the calculation method was changed. Under the Stage 2 DBPR compliance for TTHM and HAA5 is calculated as the RAA at each sampling site, referred to as a locational running annual average (LRAA).

Sampling frequency remained quarterly, but the Stage 2 DBPR increased the required number of sampling sites. For systems serving 250,000 to 999,999 people, the number of sampling sites increased from 4 to 12 per quarter.

5.9.9 Arsenic Rule

The Arsenic Rule was published in 2001 to reduce exposure to arsenic in drinking water. The arsenic MCL was reduced from 50 µg/L to 10 µg/L. Each system must take one arsenic sample per year at each entry point to the distribution system. A system with an arsenic measurement above the MCL must collect quarterly samples.

5.9.10 Radionuclides Rule

Radionuclide regulations were first promulgated by EPA in Year 1976 as part of the SDWA Standards for three groups of radionuclides: beta and photon emitters, radium, and gross alpha radiation. Radon and uranium were added to the list in the 1986 SDWA amendments. The Radionuclides Rule was published in 2000 to reduce exposure to radionuclides in drinking water.

Regulated contaminants in the Radionuclides Rule are listed in Table 5-34. MCLs for Beta/photon emitters, gross alpha particle radioactivity, and combined radium-226 and radium-228 remained at existing levels. Uranium was regulated for the first time.

Table 5-34 Radionuclide Rule MCLs

Regulated Radionuclide	MCL
Beta/photon emitters	4 mrem/yr
Gross alpha particle	15 pCi/L
Combined radium-226/228	5 pCi/L
Uranium	30 µg/L

5.9.11 Lead and Copper Rule

The Lead and Copper Rule (LCR) was published in Year 1991 to minimize lead and copper levels in drinking water by reducing water corrosivity. The LCR set action levels (ALs) of 0.015 mg/L (15 µg/L) for lead and 1.3 mg/L for copper based on 90th percentile values of samples drawn from customer taps. Exceedance of an AL is not a violation, but it triggers further required action. This action could include water quality parameter monitoring, corrosion control treatment, replacement of lead service lines, and source water monitoring and treatment.

When the LCR was originally enacted, systems were required to collect first draw samples for two consecutive 6-month sampling periods from taps at homes considered at risk for lead and copper based on the service line material and premise plumbing. The number of samples required depends on population served. Systems serving more than 100,000 people are required to collect 100 samples for standard monitoring and 50 samples for reduced monitoring. Criteria for reduced monitoring are as follows:

- Any system that meets optimal water quality parameters and is less than the action level for both lead and copper for two consecutive six-month monitoring periods can monitor once a year.
- Any system that meets optimal water quality parameters and is less than the action level for both lead and copper for three consecutive years of monitoring can monitor once every three years.

The system is required to provide analysis results to all customers whose taps were sampled within 30 days regardless of the result. All systems are required to provide an educational statement about lead in drinking water in their consumer confidence report regardless of lead levels.

Since 1991, the EPA has published minor revisions to the LCR. The 2000 revisions clarify that large systems that meet the criteria of §141.81(b)(3), are as follows: "*Any water system is deemed to have optimized corrosion control if it submits results of tap water monitoring conducted in accordance with first draw tap monitoring requirements and source water monitoring conducted in accordance with the source water monitoring requirements within the Rule, that demonstrates for two consecutive 6-month monitoring periods that the difference between the 90th percentile tap water lead level and the highest source water lead concentration is less than the Practical Quantitation Level for lead specified in § 141.89(a)(1)(ii) as 0.005 mg/L (5 µg/L)*."

Any water system may be deemed by the state to have optimized CCT if the system demonstrates to the satisfaction of the state that it has conducted activities equivalent to the corrosion control steps applicable to such system under the relevant sections of the LCR. If the state makes this determination, it shall provide the system with written notice explaining the basis for its decision and shall specify the water quality control parameters representing optimal corrosion control in accordance with § 141.82(f). Water systems deemed to have optimized corrosion control under this paragraph shall operate in compliance with the state-designated optimal water quality control parameters in accordance with § 141.82(g) and continue to conduct lead and copper tap and water quality parameter sampling in accordance with § 141.86(d)(3) and § 141.87(d), respectively.

The EPA has proposed changes to the current LCR that are discussed in Section 5.11.1.

5.9.12 Lead-Free Materials Regulations

The SWDA prohibits the "use of any pipe, any pipe or plumbing fitting or fixture, any solder, or any flux, after June 1986, in the installation or repair of (i) any public water system; or (ii) any plumbing in a residential or non-residential facility providing water for human consumption, that is not lead free" [2]. At the time, lead-free was defined as having less than 8 percent lead content.

The U.S. Federal Reduction of Lead in Drinking Water Act (RLDWA) was enacted in 2011 and took effect in 2014, further reducing the allowable lead content of lead-free materials, as follows:

- Not containing more than 0.2 percent lead when used with respect to solder and flux.
- Not more than a weighted average of 0.25 percent lead when used with respect to the wetted surfaces of pipes, pipe fittings, plumbing fittings, and fixtures.

On 17 January 2017, the EPA published a proposed rule entitled "Use of Lead Free Pipes, Fittings, Fixtures, Solder and Flux for Drinking Water" to establish labeling requirements to differentiate plumbing products that meet the lead-free requirements from those that are exempt from the lead-free requirements and to require manufacturers to certify compliance with the lead-free requirements [1]. This rule would codify revisions to the SDWA prohibition on use and introduction into commerce of certain products that are not lead-free as enacted in the RLDWA of 2011 and the Community Fire Safety Act of 2013 [1].

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[2] United States Environmental Protection Agency, "Use of Lead Free Pipes, Fittings, Fixtures, Solder and Flux for Drinking Water," United States Environmental Protection Agency, 17 January 2017. [Online]. Available: https://www.federalregister.gov/documents/2017/01/17/2017-00743/use-of-lead-free-pipes-fittings-fixtures-solder-and-flux-for-drinking-water. [Accessed 17 January 2017].

5.10 Ongoing Regulatory Determination Process

5.10.1 Drinking Water Candidate Contaminant List

The SDWA requires EPA to publish a Contaminant Candidate List (CCL) every five years identifying contaminants that are currently not subject to any proposed or promulgated national primary drinking water regulations (NPDWR), but that are known or anticipated to occur in public water systems. EPA is required to determine whether to regulate at least five contaminants on the CCL every five years, a process termed "regulatory determination." The regulatory determination process considers available health effects and drinking water occurrence data, as well as availability of suitable analytical protocols. Contaminants for which sufficient data or methods are not available to support a regulatory determination may be carried forward from the current CCL to the next. CCLs are used to set regulatory, research, and occurrence-investigation priorities within EPA.

The SDWA specifies that contaminants on the CCL shall be regulated if the EPA Administrator determines that:

- The contaminant may have an adverse effect on the health of persons.
- The contaminant is known to occur, or there is a substantial likelihood that the contaminant will occur in public water systems with a frequency and at levels of public health concern.
- In the sole judgment of the Administrator, regulation of such contaminant presents a meaningful opportunity for health risk reduction for persons served by public water systems.

If EPA determines that regulation of a contaminant in the CCL is warranted, the Agency must develop and promulgate a NPDWR based on the timeline established by the *1996 SDWA Amendments*.

The first Contaminant Candidate List (CCL 1) was published in draft form in March 1998, and the second Contaminant Candidate List (CCL 2) was finalized in February 2005. Subsequent sections describe the regulatory determinations resulting from the third CCL and provides an overview of the contaminants recently added to the fourth CCL for regulatory determination.

5.10.1.1 Candidate Contaminant List 3

EPA implemented a different process to develop CCL 3 than was used for CCL 1 and CCL 2. This new process considered evaluations from previous CCLs and included substantial expert input and recommendations from various groups, including the National Academy of Science's National Research Council, the National Drinking Water Advisory Council (NDWAC), and the Science Advisory Board. Contaminants of emerging concern contained in CCL 3 (September 2009) include 116 microbial pathogens, inorganic compounds, synthetic organic chemicals, disinfection byproducts (DBPs), hormones, and pharmaceuticals.

Preliminary regulatory determinations for contaminants on CCL 3 were published in the Federal Register on October 20, 2014. With this action EPA made regulatory determinations for five unregulated compounds. A positive determination was made to regulate strontium and negative determinations were made for dimethoate, 1,3-dinitrobenzene, terbufos, and turbufos sulfone. Regulatory determinations for other contaminants listed on CCL 3 were not made because they did not meet one or more of several criteria including availability of nationally representative finished water occurrence data, a completed health assessment, or a widely available analytical method for analysis.

On January 4, 2016, EPA published in the Federal Register the final determinations not to regulate four of the 116 CCL 3 contaminants – dimethoate, 1.3-dinitrobenzene, terbufos, and turbufos sulfone. EPA delayed the final regulatory determination on strontium to consider additional data and decide whether there is a meaningful opportunity for health risk reduction by regulating strontium in drinking water.

5.10.1.2 Contaminant Candidate List 4

The fourth Contaminant Candidate List (CCL 4) was published in draft form on February 2, 2015 (80 FR 6076). The Draft CCL 4 lists 100 chemicals or groups of chemicals and 12 microbial contaminants. EPA solicited nominations for contaminants to include in CCL 4 in May 2012 (77 FR 27057) and two of the nominated chemicals, nonylphenol and manganese, were ultimately selected for inclusion in Draft CCL 4. EPA previously made a negative regulatory determination for manganese in 2003 as part of CCL 1 (68 FR 42898); however, included it in Draft CCL 4 due to new health effects data that showed some potential neurological effects. Other contaminants included in Draft CCL 4 include those from CCL 3 not selected for regulatory determination. The Final CCL 4 was published on November 17, 2016 and it included 97 chemicals or chemical groups and 12 microbial contaminants.

In March 2020, the EPA announced that the following contaminants from CCL 4 would not be regulated: 1,1-dichloroethane, acetochlor, methyl bromide, metolachlor, nitrobenzene, and RDX.

5.10.2 Unregulated Contaminant Monitoring Rules

The Unregulated Contaminants Monitoring Rule (UCMR) program was developed in coordination with the CCL regulations. The data collected by the UCMR process is used to support analysis and review of contaminant occurrence, to guide the CCL process, and to support determination of whether to regulate a contaminant to protect public health. The Safe Drinking Water Act Amendments of 1996 required EPA to establish criteria for a program to monitor unregulated contaminants and to identify not more than 30 contaminants to be monitored every 5 years. EPA published a list of unregulated contaminants for the first UCMR cycle (UCMR 1) in September 1999 and a second cycle (UCMR 2) in January 2007. Since Year 2013, EPA has published a third and fourth list for monitoring of unregulated contaminants under the UCMR.

5.10.2.1 UCMR 3

EPA published the UCMR 3 in May 2012. The structure of UCMR 3 is similar to previous UCMRs. UCMR 3 requires all systems serving greater than 10,000 people to monitor for 21 List 1 contaminants and systems serving greater than 100,000 people to monitor for the seven List 2 contaminants. One notable difference between UCMR 3 and previous rules is that consecutive systems are required to conduct monitoring. Participating systems will conduct UCMR 3 monitoring during one consecutive 12-month period between 2013 and 2015. UCMR 3 included six perfluorinated compounds, including perfluorooctanesulfonic acid (PFOS) and perfluorooctanoic acid (PFOA). The EPA established health advisories for PFOS and PFOA, recommending individual or combined concentrations of less than 70 nanograms per liter (ng/L) in drinking water supplies. Further discussion on the potential for regulatory determination on PFAS compounds is provided in Section 5.11.3.1.

5.10.2.2 UCMR 4

EPA published the final UCMR 4 in the Federal Register on December 20, 2016. UCMR 4 monitoring will occur from 2018-2020 and includes monitoring for a total of 30 chemical contaminants: 10 cyanotoxins (nine cyanotoxins and one cyanotoxin group) and 20 additional contaminants (two metals, eight pesticides plus one pesticide manufacturing byproduct, three brominated haloacetic acid (HAA) disinfection byproducts groups, three alcohols, and three semivolatile organic chemicals (SVOCs)). UCMR 4 requires all community water systems (CWSs) and non-transient non-community water systems (NTNCWSs) serving greater than 10,000 people to monitor for the 20 additional contaminants, and it requires that systems served by surface water and ground water under the direct influence of surface water (GWUDI) also monitor cyanotoxins. Of the CWSs and NTNCWSs serving 10,000 or fewer people, a nationally representative set of 800 randomly selected SW and GWUDI systems will monitor for cyanotoxins and a different set of 800 randomly selected systems will monitor for the 20 additional contaminants. Sampling for the selected cyanotoxins will occur twice a month for four consecutive months during the timeframe of March through November, while the typical quarterly monitoring cycle will be used for the additional 20 contaminants.

5.11 Potential Future Drinking Water Regulations

The Safe Drinking Water Act and its amendments require that the EPA reevaluate existing drinking water regulations on a periodic basis and develop and promulgate new standards and regulations as necessary to protect public health. Several regulations have been proposed by EPA and are in various stages of development, review, and approval.

5.11.1 Proposed Lead and Copper Rule Revisions

On October 10, 2019, the EPA released proposed LCR revisions that were published in the federal register on November 13, 2019. The proposed LCR includes several revisions with a focus on switching from reactive to proactive measures to improve finished water quality at the customers' tap. Some of major revisions in the proposed LCR include:

- Public water systems (PWSs) must develop a publicly available lead service line (LSL) inventory (including lead goosenecks and downstream galvanized iron service lines on both PWS's side and homeowner's side).
- Retain the current lead AL of 15 µg/L, and add a new lead trigger level of 10 µg/L.
- If the 90th percentile lead level exceeds the AL, then the PWS must fully replace 3 percent of LSLs annually for consecutive 6-month monitoring periods.
- PWSs must "find-and-fix" individual sites with tap lead levels greater that the AL by conducting additional sampling to locate the lead source and working with their Primacy Agency to identify if corrective actions are needed.
- PWSs must replace the water system-owner portion of an LSL when a customer chooses to replace their portion of the LSL.
- "Testing out" of LSLs based on sampling results would be prohibited, and instead LSLs should be included in an inventory for replacement.
- Partial LSL replacements would no longer be allowed except in rare circumstances.
- LCR compliance sampling modifications would include a new Tier structure with LSLs as Tier 1 and copper pipe with lead solder as Tier 3; additionally, pre-flushing and removal of aerators would be prohibited, and the use of wide-mouth bottles would be required.
- PWSs must notify customers within 24 hours of a lead AL exceedance, and notify individual customers within 24 hours if their tap sample exceeded the lead AL.
- PWSs must test for lead at 20 percent of schools and 20 percent of childcare facilities.

- Calcium hardness would no longer be an accepted corrosion control treatment (CCT), and orthophosphate would be the only accepted phosphate-based corrosion inhibitor.
- Water quality parameter (WQP) monitoring data would be reviewed during sanitary surveys, and WQPs related to calcium hardness would be eliminated.

The new lead trigger level of 10 µg/L was proposed to prompt water systems to take proactive actions to reduce lead levels prior to exceeding the lead AL. If the 90th percentile lead concentration exceeds the new trigger level of 10 µg/L, the PWS would be required to complete the following:

- Conduct a corrosion control study to either re-optimize their existing CCT or develop a CCT (i.e., small/medium systems that did not previously treat for corrosion).
- Complete annual LCR monitoring at the standard number of sites.
- Conduct public outreach on ways to minimize lead leaching.
- Work with the PWS's Primacy Agency to set an annual goal for replacing LSLs.

There are no proposed changes to the LCR revisions based on copper sampling or the copper concentrations measured. The public comment period on the proposed LCR was open until February 12, 2020. The final LCR is anticipated to be promulgated in 2020.

5.11.2 Radon

EPA proposed new regulations for radon in October 1999. Two alternative compliance approaches were included in the proposed radon rule:

- States can elect to develop programs to address the health risks from radon in indoor air through adoption and implementation of a multimedia mitigation program. Under this approach, individual water systems would be required to reduce radon levels in the treated water to 4,000 pCi/L or lower. EPA will encourage states to adopt this approach, as it is considered the most cost-effective way to achieve the greatest reduction in radon exposure risk.
- If the State elects not to develop a multimedia radon mitigation program, individual water systems will be required to reduce radon levels in their system's treated water to 300 pCi/L, or to develop local multimedia mitigation programs and to reduce radon levels in drinking water to 4,000 pCi/L.

Systems with radon levels at or below 300 pCi/L would not be required to treat their water to remove radon. States will likely be granted fairly wide latitude in developing and implementing the multimedia programs, and it is expected that the programs will differ significantly from state to state. The need for radon treatment will be based on results of quarterly monitoring. If the state regulatory agency commits to the multimedia mitigation and alternative MCL compliance approach within 90 days of final promulgation of the rule, it will be granted an additional 18 months to achieve compliance.

Considerable controversy currently surrounds the regulation of radon in drinking water supplies, and modification of this regulation as currently proposed could significantly alter the requirements contained in the final rule. There is no recent information on the status of this proposed regulation, and no revised timeline for its implementation has been issued by EPA.

5.11.3 Contaminants on the Regulatory Horizon

On January 4, 2016, EPA delayed the final regulatory determination on strontium to consider additional data and decide whether there is a meaningful opportunity for health risk reduction by strontium in drinking water. The Final CCL 4 was published on November 17, 2016 and it included 97 chemicals or chemical groups and 12 microbial contaminants. Cyanotoxins are in the CCL 4 and included in the UCMR 4 monitoring. In December 2016, EPA announced the review results for the Agency's third Six-Year Review (Six-Year Review 3) and eight NPDWRs were chosen as candidates for regulatory revision. These eight NPDWRs include chlorite, *Cryptosporidium* (under the Surface Water Treatment Rule (SWTR), interim enhanced surface water treatment rule (IESWTR and LT1), haloacetic acids (HAA5), heterotrophic bacteria, *Giardia lamblia*, *Legionella*, total trihalomethanes (TTHM), and viruses (under the SWTR).

In addition to the 76 NPDWRs reviewed in detail for the Six-Year Review 3, 12 other NPDWRs were included in the review but were not given detailed consideration because of other recent or ongoing regulatory actions (e.g., lead, copper, total coliforms (under Aircraft Drinking Water Rule (ADWR) and RTCR), *E. coli*, and eight carcinogenic volatile organic compounds (cVOCs)). The Six-Year Review 3 also evaluated unregulated DBPs including chlorate and nitrosamines.

5.11.3.1 Per- and Polyfluoroalkyl Substances (PFAS)

Per- and polyfluoroalkyl substances are a class of thousands of man-made chemicals that are used in the manufacture of many industrial and consumer products, including firefighting foams, water- and oil-resistant coatings, cookware, food packaging, medical devices, cosmetics, lubricants, inks and paints. PFAS chemicals consist of a carbon chain (an alkyl group) that is highly substituted with fluorine atoms and contains other functional groups, such as carboxylic acids, sulfonic acids, and ethers. Their properties make them heat stable, non-biodegradable, bioaccumulative, and very persistent in the environment. They are also highly mobile in water and difficult to remove as conventional treatment processes are ineffective at reducing concentrations.

Due to their widespread application, PFAS are now found in many drinking water sources across the United States and thus impact both water and wastewater treatment facilities. As a result, concern from federal and state regulators over these chemicals has steadily increased over the past decade. In February 2019, the USEPA issued a PFAS Action Plan aimed at comprehensively addressing PFAS in the environment. The USEPA has proposed regulating PFAS under the Safe Drinking Water Act (SDWA), the Toxic Substances Control Act (TSCA), the Comprehensive Environmental Response, Compensation and Liability Act (CERCLA, also known as Superfund), and the Clean Air Act.

Currently, there are no federal maximum contaminant levels (MCLs) established for PFAS chemicals under the SDWA. In 2016 the USEPA established non-enforceable drinking water health advisory levels for two prevalent PFAS chemicals, perfluorooctanoic acid (PFOA) and perfluorooctane sulfonic acid (PFOS). The health advisory level for the total concentration of both PFOA and PFOS in drinking water is 70 ng/L. However, the USEPA is in the process of making a regulatory determination for PFOA and PFOS as part of the PFAS Action Plan. The proposed regulatory determination is currently under interagency review and has not been made public. Still, the USEPA has signaled that they intend to establish an MCL for PFOA and PFOS, and potentially others.

State-level regulators, in some cases, have outpaced the USEPA in establishing their own guidance and regulations. Almost half of U.S. states have established some form of PFAS guidance values for groundwater and/or drinking water, but the approaches vary. A handful of states have established or proposed state-wide drinking water MCLs. To date, the State of Nebraska has not established any PFAS related drinking water guidance values or regulations.

5.11.3.2 Cyanotoxins

A chemically diverse group of over 100 cyanobacterial metabolites have been identified as cyanotoxins, which have been variously classified as neurotoxins, hepatoxins, and contact irritants. Assuming EPA waits until the UCMR 4 monitoring is complete in 2020, the Agency could either make a positive regulatory determination or simply move directly to a proposed rule. A cyanotoxin rule would typically involve a two-year development period (2022) and a final rule could follow in approximately another two years (2024). If the Agency elects to make a positive regulatory determination prior to developing a proposed rule, then the timing of the regulatory determination rulemaking would figure into this timeline and delay the proposed rule by two to seven years. There is also increasing focus at the state level on harmful algal blooms and recreational water use.

5.11.3.3 Nitrosamines

Five organic nitrogen-containing compounds (4 nitrosamines and nitrosopyrrolidine) that have been detected in treated drinking water are listed on CCL 4. Formation of these compounds is associated with disinfection with free chlorine in the presence of naturally occurring ammonia in the source water or ammonia added to treated water to form a combined-chlorine residual. Formation of these nitroso-compounds requires a nitrogenous organic precursor. Dimethylamine has been shown to be particularly reactive in formation of N-nitrosodimethylamine (NDMA) in drinking water, with formation from several other less reactive precursors possible.

Regulation of nitrosamines in drinking water remains controversial for several reasons. Recent research on human exposure to nitrosamines indicates that drinking water contributes a very small percentage (less than 0.01 percent) of total exposure compared with natural formation in the body and consumption in certain foods. Therefore, it is unclear whether or not a regulation for nitrosamines would meet the SDWA criteria for "a meaningful opportunity for health risk reduction for persons served by public water systems". Likely strategies for reducing nitrosamine formation in drinking water, such as limiting or discontinuing use of polyDADMAC polymers or chloramine disinfectant residual, would also present simultaneous compliance issues with other currently regulated contaminants.

MCLs for individual nitrosamines or as a chemically similar group of several compounds would be established during the rulemaking process. The body of research on animal and human responses to nitrosamine exposure indicates the MCLs for nitrosamines in drinking water would be at the nanogram per liter (ng/L) level. While Health Canada has established a maximum allowable concentration of 40 ng/L in drinking water, several agencies have adopted non-enforceable guidelines and advisory levels for NDMA in drinking water as indicated below:

- World Health Organization guideline of 100 ng/L.
- Massachusetts guideline level of 10 ng/L.
- State of California notification level of 10 ng/L and public health goal of 3 ng/L.
- EPA Regions 3 and 6 nonenforceable screening level of 0.42 ng/L of NDMA.
- Arizona water quality criterion of 30 ng/L in NPDES permits.

A decision not to regulate nitrosamines as part of the preliminary regulatory determinations for contaminants on CCL 3 was published in the Federal Register on October 20, 2014. However, EPA evaluated existing Microbial/Disinfection Byproducts (MDBP) regulations and unregulated DBPs including nitrosamines as part of Six-Year Review 3. Because nitrosamines are DBPs that may be introduced or formed in public water systems related to disinfection practices, EPA believes it is important to evaluate these DBPs in the context of the review of existing MDBP regulations. Nitrosamines are included in the CCL 4.

The AWWA Governmental Affairs Office recommends that a utility consider sampling for nitrosamines if it did not participate in UCMR 2, to develop an understanding of nitrosamine occurrence and formation patterns within its system (AWWA, 2012). If it has not already done so, LWS should consider implementing a sampling program to analyze NDMA in the distribution system in anticipation of a potential future NDMA regulation.

5.11.3.4 Strontium

Strontium occurs in drinking water supplies due to dissolution of naturally-occurring mineral deposits, and due to its commercial and industrial uses in pyrotechnics, steel production, as a catalyst, and as a lead scavenger. EPA delayed the final CCL 3 regulatory determination on strontium to consider additional data and decide whether there is a meaningful opportunity for health risk reduction by regulating strontium in drinking water. A final rule on strontium is expected in 2019 or 2020.

5.11.3.5 Chlorate

Chlorate compounds are used in agriculture as defoliants or desiccants and may occur in drinking water related to use of disinfectants such as chlorine dioxide. A decision not to regulate chlorate as part of the preliminary regulatory determinations for contaminants on CCL 3 was published in the Federal Register on October 20, 2014. However, EPA evaluated existing MDBP regulations and unregulated DBPs including chlorate as part of Six-Year Review 3. Because chlorate is a DBP that may be introduced or formed in public water systems related to disinfection practices, EPA believes it is important to evaluate this DBP in the context of the review of existing MDBP regulations. Chlorate is included in the CCL 4.

5.11.3.6 Perchlorate

On February 11, 2011, EPA published its decision to move forward with the development of a regulation for perchlorate, a contaminant evaluated under CCL 2. Under the current regulatory schedule, a proposed MCL for perchlorate would have been expected sometime in 2014, and a final MCL no later than 2016, with compliance required by 2019. However, EPA is still finalizing its peer review of the modeling research recommended by a Science Advisory Board in conjunction with the Food and Drug Administration. A panel meeting of the peer reviewers was held on January 10 and 11, 2017, and a subsequent peer review will be scheduled to evaluate methods to develop a maximum contaminant level goal (MCLG) for perchlorate in drinking water.

Finished water quality data for perchlorate was not available in the dataset provided for the project. If perchlorate monitoring has not already been conducted, it is recommended that LWS monitor perchlorate to determine if they will be in compliance with a potential new perchlorate rule.

5.11.3.7 Fluoride

In January 2011, the United States Department of Health and Human Services (HHS) announced a proposed recommendation that fluoride levels in drinking water be set at an optimal level of 0.7 mg/L. Concurrent with the HHS announcement, EPA announced plans to initiate a review of the current MCL and MCLG for fluoride. HHS's proposed recommendation would replace the Year 1962

US Public Health Standard of 0.7 to 1.2 mg/L, under which the optimal fluoride level is determined based upon the ambient air temperature of the geographic region. HHS believes that this revised optimal concentration will provide the best balance of public protection from dental caries (tooth decay) and the desire to limit the risk of dental fluorosis (spotting/pitting damage to tooth enamel), particularly in children.

Starting in Year 2015, the HHS's recommended optimal fluoridation level of drinking water is 0.7 mg/L. While the HHS guidance is advisory rather than regulatory, EPA could elect to modify current regulations governing maximum fluoride levels in response to HSS recommendations and to the agency's review of recent research results.

In December 2016, EPA announced the review results for the third Six-Year Review, and it was determined that a revision to the NPDWR for fluoride is not appropriate at this time. EPA determined that the potential revision of the fluoride NPDWR is a lower priority that would divert significant resources from the higher priority rulemakings that the Agency intends to undertake, but the Agency will continue to monitor the evolving science, and, when appropriate, will reconsider the fluoride NPDWR's relative priority for revision.

5.11.3.8 Hexavalent Chromium

The existing regulation for total chromium in drinking water was reevaluated by EPA as part of Six-Year Review 2. However, since the Agency had initiated a reassessment of health risks associated with chromium exposure, EPA decided not to revise the NPDWR while that effort was in progress. EPA began a rigorous and comprehensive review of hexavalent chromium health effects following the release of the toxicity studies by the National Toxicology Program in 2008. In September 2010, EPA released a draft scientific assessment for public comment and external peer review.

Hexavalent chromium (Cr^{6+}) has come under increased scrutiny recently with the release of an Environmental Working Group study in December 2010 that found levels of hexavalent chromium exceeding the non-enforceable public health goal set by the California Department of Public Health (CDPH) in the tap water of 25 of 35 US cities tested. Based on additional recent research, the schedule for the hexavalent chromium human health assessment was revised by EPA in Feb 2012, with the final version now expected to be approved and posted in the near future. When this human health assessment is finalized, EPA will carefully review the conclusions and consider all relevant information to determine if a new standard needs to be set. Hexavalent chromium levels in public drinking water supplies are currently being monitored as part of UCMR 3. EPA Six-Year Review 3 determined that a revision to the existing regulation for total chromium was not appropriate for revision at this time as the health effects assessment is still ongoing.

In a separate regulatory action, the CDPH adopted a drinking water MCL for hexavalent chromium of 10 μg/L, which became effective July 1, 2014. The regulations adopted by CDPH specify initial monitoring requirements, approved analytical methods and detection limits, and best available technologies for treatment. Compliance with the MCL is based on a running annual average (RAA) of hexavalent chromium measurements averaged quarterly.

5.11.3.9 Volatile Organic Compounds

In January 2011, the EPA Administrator announced that carcinogenic Volatile Organic Compounds (cVOCs) will be the first contaminants regulated as a group rather than as individual compounds under the Agency's new Drinking Water Strategy. Eight currently regulated cVOCs and eight currently unregulated cVOCs have been proposed for regulation as a group. In December 2016, EPA announced the review results for the Six-Year Review 3. The reviews of eight cVOCs were included but were not given detailed consideration because of other recent or ongoing regulatory actions. The eight cVOCs mentioned in the Six-Year Review 3 include 1,2-Dichloroethane (Ethylene dichloride), 1,2-Dichloropropane, Benzene, Carbon Tetrachloride, Dichloromethane (Methylene chloride), Tetrachloroethylene (PCE), Trichloroethylene (TCE), and Vinyl chloride. The ultimate form of this regulation remains to be determined.

5.11.3.10 Methyl Tertiary Butyl Ether

Methyl tertiary butyl ether (MTBE) is an oxygenate additive used in gasoline to increase the octane number. It has been widely used in gasoline in the United States as a replacement for lead; however, its use has declined in recent years due incorporation of ethanol in fuels. MTBE is very soluble and has been detected in numerous water supplies but is most commonly found in ground water supplies.

In 1997, EPA issued a drinking water advisory for MTBE of 20 to 40 µg/L based on taste and odor. MTBE was included in CCL 1 and CCL 2 for evaluation, with negative regulatory determinations because its regulation would not present a meaningful opportunity for health risk reduction for persons served by public water systems. Because of several prominent cases of drinking water contamination with MTBE in the past, public interest related to MTBE regulation remains active. Therefore, MTBE was carried over to CCL 3 and CCL 4 for further evaluation; however, no schedule for revision of the health risk assessment for MTBE has been set.

5.11.3.11 Legionella

Legionella bacteria can cause a serious type of pneumonia called Legionnaires' disease, and also a less serious infection called Pontiac fever that has symptoms similar to a mild case of the flu. The bacterium grows best in warm water conditions including large plumbing systems, cooling towers (air-conditioning units for large buildings), and hot water tanks and heaters. EPA's third six-year review notice (January 11, 2017) highlights an opportunity to further reduce the risk posed by *Legionella*. The notice suggests a linkage being drawn between maintaining a secondary disinfectant residual and reducing the risk posed by *Legionella*.

5.12 Water Treatment Plant Improvements, Expansion and Rehabilitation

As indicated in Figure 5-17, the existing treatment capacity of 120 mgd for the combined East and West Plants is capable of meeting projected demands through the Year 2037. The *2014 Master Plan* had identified the next plant expansion to occur at the West Treatment Plant by means of filter rehabilitation. The scope of this master plan update included additional focus on condition assessment of the existing treatment plants, along with input from operations, to take a second look at this approach and compare expansion of the two plants. This section also addresses improvements to the East Plant for arsenic removal.

5.12.1 East Plant Improvements for Arsenic Removal

LWS will need to implement a treatment system in the future to address the relatively high concentrations of arsenic in the HCWs and expected concentrations of arsenic in the future HCWs. Previous studies evaluated the use of enhanced coagulation with ferric chloride to meet proposed finished water quality goals of 8 µg/L, 4 µg/L and non-detect levels. At the required dosages of ferric chloride (5 to 15 mg/L), filter run times and filter productivity were significantly reduced. As such, arsenic treatment through enhanced coagulation is not feasible with direct filtration and would require implementation of a clarification basin upstream of filtration.

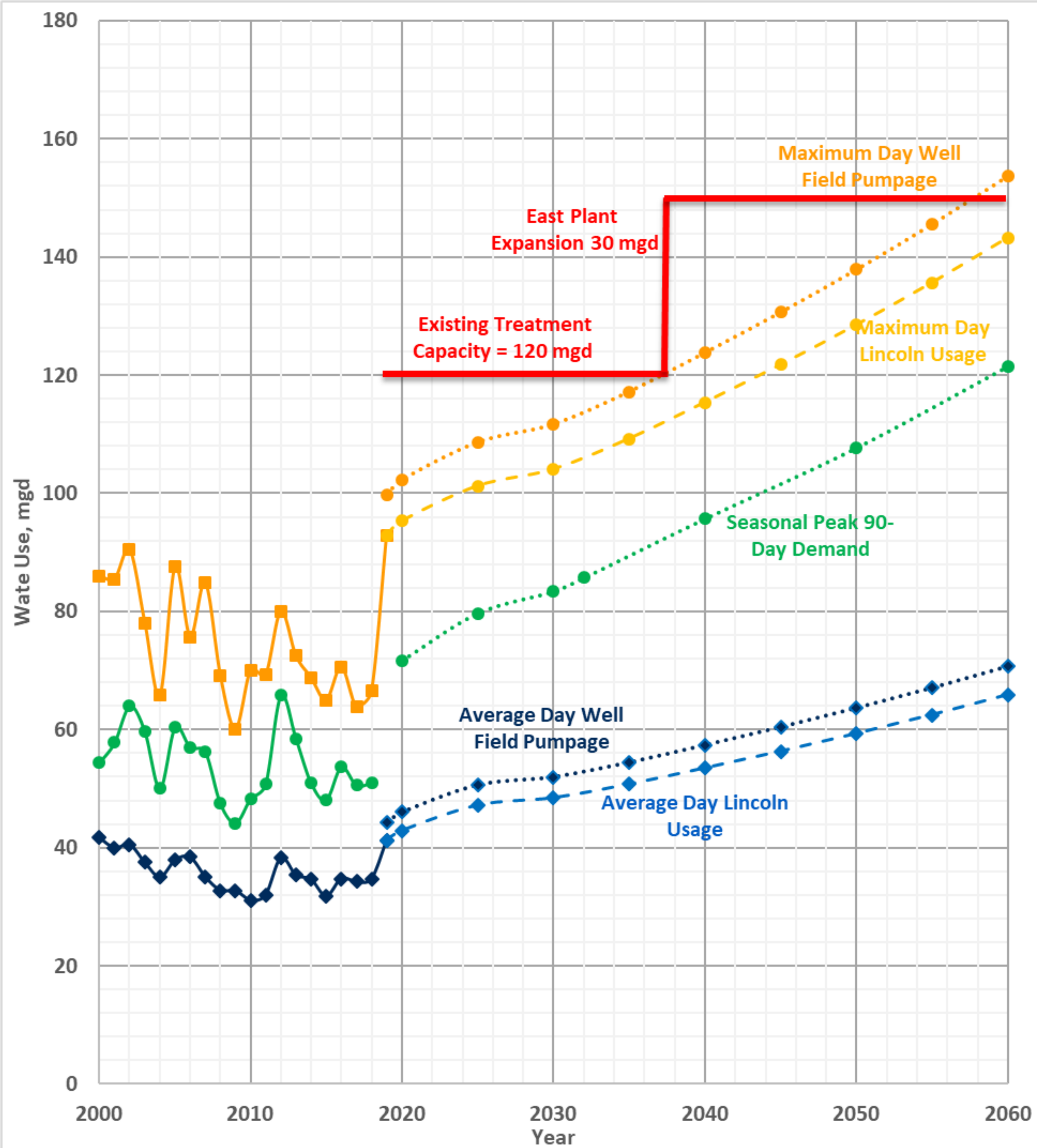


Figure 5-17 Future Treatment Expansion

Additional bench-scale testing is recommended to further investigate treatment alternatives and identify a cost-effective solution for arsenic treatment. In the absence of a formal process evaluation, a conceptual opinion of probable construction cost (OPCC) has been developed for CIP planning purposes. The conceptual OPCC is based on implementation of an arsenic adsorption system located downstream of the existing filters at the East Plant. The adsorption system would consist of vertical pressure vessels filled with a granular ferric oxide (GFO) or granular ferric hydroxide (GFH) media. Based on preliminary assumptions, the vessel design considers an empty bed contact time (EBCT) of 4 minutes and treatment capacity of 35 MGD (58 percent of maximum plant flow) to achieve a blended water arsenic concentration of 6 µg/L. The system is expected to include a transfer pump station and 18 vessels, each 12 ft in diameter. Implementation of an arsenic treatment system is scheduled to occur in Year 2025.

5.12.2 Water Plant Expansion

Throughout the condition assessment activities, multiple concerns were identified by staff, primarily regarding the ability to physically process over 70 mgd through the West Water Treatment Plant, based upon previous operational knowledge from the 1980's. Specifically, when the West WTP was pushed to rates around 70 mgd, a bypass was utilized which circumvented the entire treatment process including aeration, chlorine contact, and filtration. This operational practice was subsequently discontinued as the safe drinking water act (SDWA) was amended and the bypass has been disabled.

In light of these restrictions, in order to expand the West WTP some other modifications would be required in addition to the filter rehabilitation. Other recommended improvements include replacement of the existing clearwell transfer pumps (which would increase capacity and simplify CT calculation), addition of a fourth aerator and contact basin, chemical feed modifications, and an allowance for hydraulic improvements to ensure the facility could convey the flows. The total capital cost for expansion of the West WTP by 12 mgd is summarized in table D-1. The planning level opinion of probable capital cost is $10,749,000 for a 12 mgd expansion, which equates to an expansion cost of $0.90/gallon.

Alternatively, the East WTP currently has a capacity of 60 mgd (originally 50 mgd prior to filter re-rating). The plant was configured such that 16 additional filters can be added to provide additional capacity of 120 mgd. As part of the study B&V provided costing analysis of adding either two filters (15 mgd) or four filters (30 mgd), additional ozone capacity and associated infrastructure. The cost to add only two filters was not deemed to be in the City's best interest as it would be inefficient with respect to building walls, foundations, ozone system expansion, etc. Therefore, we would recommend that the next expansion of the East Water Treatment Plant should be 30 mgd. The planning level opinion of probable capital cost for this expansion would be $24,804,000 which equates to $0.83/gallon. Expansion of the East WTP would also be more beneficial from a treatment perspective as the City will add one or two more collector wells in the interim, increasing their reliance on groundwater which is under the influence of surface water.

It is therefore our recommendation that the City plan on expansion of the East Water Treatment Plant starting in Year 2032, which allows sufficient time for design and construction prior to the need in Year 2037 as shown in Figure 5-17. The opinion of probable construction cost for this improvement is $24,804,000 in Year 2020 dollars.

5.12.3 Water Plant Rehabilitation

It has been almost 30 years since any major rehabilitation projects have occurred at the two water treatment plants. Based upon the condition assessment work completed as part of this study we recommend budgeting $2,285,000 for a rehabilitation project at the West Water Treatment Plant and $669,000 for a rehabilitation project at the East Water Treatment Plant, both within the first six years of your capital improvement project. The improvements associated with these rehabilitation projects are summarized in Appendix D and have a cost basis of Year 2020.

6.0 Distribution System Facilities and Analyses

The LWS service area is currently divided into six service levels - Low, High, Belmont, Southeast and more recently the Cheney and Northwest Service Levels. The Cheney Service Level was created in Year 2001 and serves the southeast portion of the service area. The Northwest Service Level was created in Year 2002 near the NW 12th Street Reservoir, to serve a new development on high ground in that area. A schematic hydraulic profile of Lincoln's water distribution system facilities and service levels is shown on Figure 6-1 located on the following page.

6.1 High Service Pumping and Transmission

The high service pumps at the Ashland WTP are located in three separate buildings. Pumps 1 through 6 are located in the North Pumping Station. Pumps 7, 8, and 9 are located in the West Pumping Station. Pumps 10, 11, and 12 are located in the South Pumping Station. Data on the WTP high service pumping units is shown in Table 6-1.

Table 6-1 WTP High Service Pumps

Pump No.	Drive Type	Rated Capacity		Head (ft)	Pump Motor	
		(gpm)	(mgd)		(hp)	(rpm)
1	Electric - 2400 V	14,000	20.2	115	600	900
2	Electric - 2400 V	9,800	14.1	205	700	1200
3	Electric - 480 V	14,000	20.2	130	700	900
4	Electric - 2400 V	14,000	20.2	233	1250	900
5	Electric - 2400 V	14,000	20.2	233	1250	900
6	Electric - 2400 V	14,000	20.2	233	1250	900
7	Diesel Engine	15,000	21.6	345	1950	900
8	Electric - 4160 V	15,000	21.6	345	1750	888
9	Diesel Engine	15,000	21.6	345	1950	900
10	VFD – 480 V	14,000	20.2	130	600	710
11	VFD – 2400 V	14,500	20.9	350	1750	720
12	VFD – 2400 V	14,500	20.9	350	1750	720

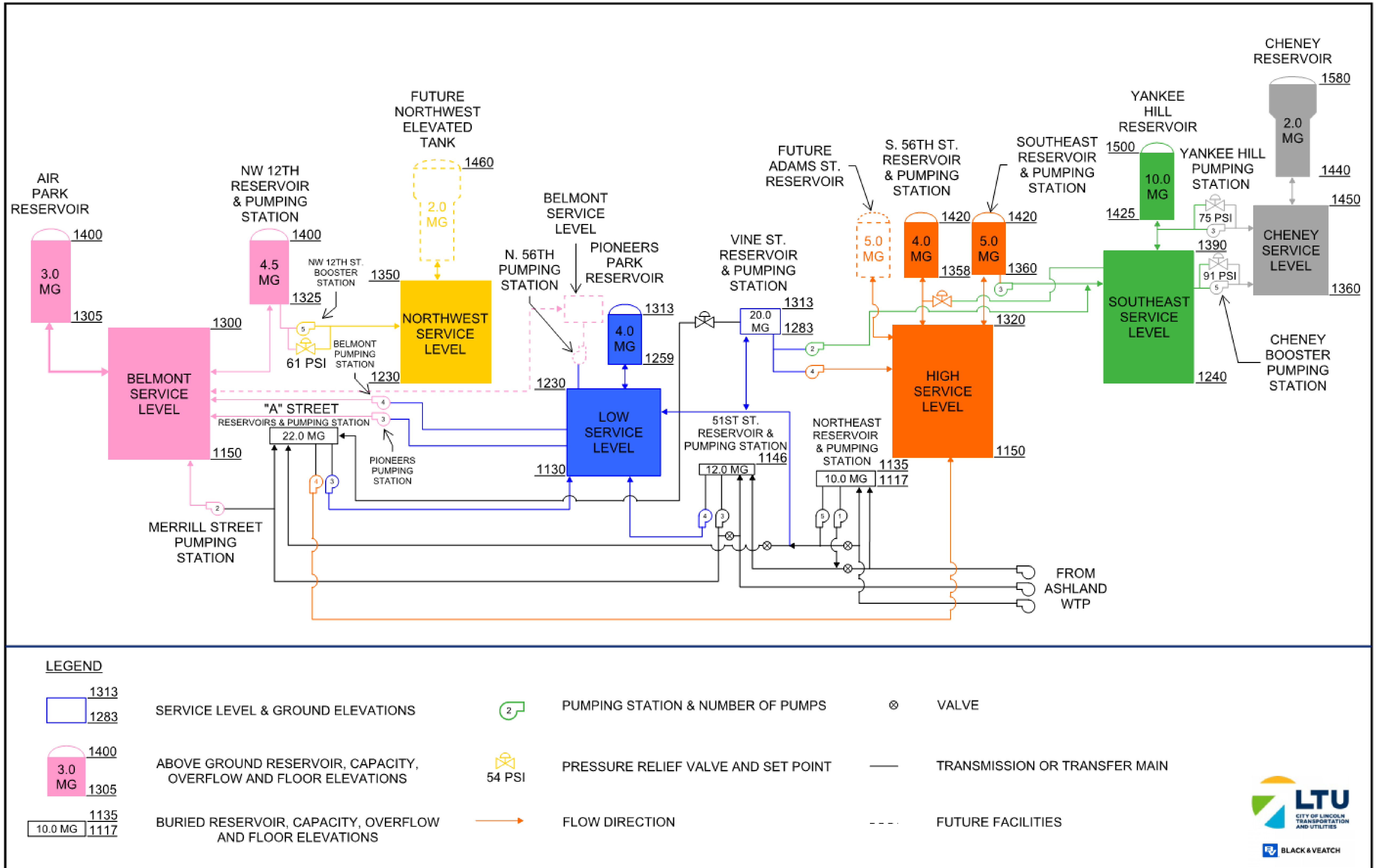


Figure 6-1 **Existing Transmission and Distribution System Schematic**

Treated water from the Ashland treatment facilities is pumped to Lincoln through approximately 17 miles of parallel 36-inch and 48-inch transmission mains, plus 17 miles of 54/60-inch main that parallels the two other mains for a portion of the distance to Lincoln. An established 100 psi operating pressure limitation for the 36-inch main requires that it be operated separately from the 48-inch (except at low pressures) and also separate from the 54/60-inch mains, which have working pressures of 150 psi.

Previous studies recommended construction of the 54-inch/60-inch transmission main, approximately 23 miles in length, from the Ashland WTP to the Vine Street Reservoir and Pumping Station. The segment from the Ashland WTP to an interconnection with the existing 48-inch main at Greenwood, approximately 7.8 miles in length, was completed in 1994. A second segment, 60-inch main extending 10 miles from Greenwood to the Northeast Reservoir, was completed in Year 2010. The remaining portion of approximately 5 miles of 54-inch from the Northeast Reservoir to the Vine Street Reservoir, not including approximately 1 mile previously constructed, was evaluated for this project and is recommended to be constructed around Year 2032 as discussed in Chapter 8.

Under lower flow conditions, approximately 48 mgd pumped from the Ashland WTP can be delivered directly to the Low Service Level. Under higher flow conditions, which result in greater head losses in the transmission mains, the water must be re-pumped into the Low Service Level by pumps located at the Northeast, 51st Street, and "A" Street locations. And under even greater flow rates, a transfer pump at the Northeast location is used to deliver flow to the 51st Street Reservoir, and transfer pumps at the 51st Street location are used to deliver flow to the "A" Street Reservoirs. Additional information on the transmission system storage and pumping facilities are described later in this chapter, in the section on the Low Service Level.

6.2 Service Levels

Ground elevations within the existing service area range from about 1,130 feet (USGS datum) along Salt Creek to about 1,450 feet in the Cheney Service Level. The highest ground is located in the northwest and southeast portions of the service area.

Service level boundaries are established to maintain acceptable distribution system pressures. The boundaries should have sufficient flexibility to allow minor modifications to provide adequate service, particularly at higher elevations and in developing areas. The service area is currently divided into four major service levels - Low, High, Belmont, and Southeast, and two smaller service levels including the Cheney and Northwest Service Levels. The static hydraulic gradient for all of the service levels are established by the maximum water service elevation of floating storage facilities within the service area, except the Northwest Service Level which is controlled by a PRV but has floating storage recommended with the recommendations of this report. The ground elevations served and static hydraulic gradient for each service level are shown in Table 6-2.

Table 6-2 Service Levels

Service Level	Ground Elevation(1) (ft)	Static Hydraulic Gradient Elevation (ft)
Belmont Service Level	1130 – 1290	1,400(2)
Low Service Level	1130 – 1230	1,313(2)
High Service Level	1150 – 1320	1,420(2)
Southeast Service Level	1240 – 1390	1,500(2)
Cheney Service Level	1340 – 1430	1,580(2)
Northwest Service Level	1240 – 1320	1,460(3)

(1)Principal part of service level, USGS datum.
(2)Established by overflow elevation of floating storage within service level.
(3)Currently established by PRV setting at pumping station discharge.

6.3 Pumping Stations and System Storage

6.3.1 Low Service Level

The Low Service Level services the area bordering Salt Creek and encompasses the main business district, the University of Nebraska, and major industrial areas.

The 51st Street, Northeast, and "A" Street Pumping Stations supply the Low Service Level. Direct pumping from Ashland can also supply the Low Service Level during lower demand periods. The Low Service Level is also served by the Vine Street Reservoir and the Pioneers Park Reservoir, with reservoir overflows of 1313 which establish the static hydraulic gradient.

6.3.1.1 Northeast Pumping Station and Reservoir

The Northeast Pumping Station and Reservoir is located east of the intersection of 98th Street and U.S. Highway 6. In Year 1997, a facility expansion was completed that expanded the storage volume from 5.0 MG to 10.0 MG and included the addition of a fourth Low Service Level pump. The reservoir is supplied from the Ashland Water Treatment Plant located approximately 17 miles away through the 48-inch and 54/60-inch transmission mains. The reservoirs have overflow elevations of 1,135 feet, sidewater depth of 18 feet, and normally operate between 12 and 15.5 feet.

The Northeast Pumping Station contains one transfer pump, No. 1, with a rated capacity of 31,250 gpm (45 mgd) at 60 feet. This transfer pump was replaced in Year 2007 and discharges to the 48-inch transmission main, which extends to the 51st Street Reservoir. A variable speed drive allows the pumping capacity to vary from about 60 percent to 100 percent of the rated capacity at maximum speed (range of 27 mgd to 45 mgd).

The Northeast Pumping Station contains five Low Service Level distribution system pumps, Nos. 2 through 6. Pump No. 6 is equipped with an eddy current adjustable speed drive, but the pump is not currently used due to failure of the electronic controls for the drive. Pump No. 2 was installed in 2006. Both the transfer and distribution system pumps take suction from the adjacent reservoir. Data for the Northeast pumping units is given in Table 6-3.

Due to the hydraulic capacity of the 54/60-inch transmission main, the Northeast Pumping Station is bypassed during peak and time of day periods to avoid higher energy charges. As Low Service system demands increase in the future, opportunities for bypass may be more limited than at present.

Table 6-3 Northeast Pumping Station

Pump No.	Make	Rated Capacity (gpm)	Rated Capacity (mgd)	Head (ft)	Pump Motor (hp)	Pump Motor (rpm)
1(1)	Ruhrpumpen	31,250	45	60	600	705
2(2)	Ruhrpumpen	14,000	20.2	255	1,200	890
3(2)	Fairbanks	14,000	20.2	255	1,250	900
4(2)	Fairbanks	10,500	15.1	245	800	900
5(2)	Fairbanks	10,500	15.1	245	800	900
6(2)(3)	Fairbanks	10,500	15.1	245	800	900

(1)Transfer pump with variable speed drive.
(2)Low Service Level distribution system pumps.
(3)Pump No. 6 is currently out of service and not included in firm pump calculations

6.3.1.2 51st Street Pumping Station and Reservoirs

The 51st Pumping Station and Reservoirs are located east of the intersection of 48th Street and U.S. Highway 6. The 6.0 million gallon, 5.0 million gallon, and 1.0 million gallon ground storage reservoirs are supplied through the 36-inch and 48-inch transmission mains from the Ashland water treatment plant and Northeast Pumping Station depending on operations. The 5.0 and 1.0 million gallon reservoirs have overflow elevations of 1,148 feet and sidewater depths of 14.2 feet. The 6.0 million gallon reservoir has an overflow elevation of 1,148 feet and a sidewater depth of 15.33 feet. The pumping station contains three transfer pumps, Nos. 1 through 3. The transfer pumps were replaced in 2001 with new units each with a rated capacity of 10,500 gpm (15.1 mgd) at 185 feet. The transfer pumps discharge to a 36-inch low pressure transmission/transfer main which extends to the "A" Street Reservoirs. The 51st Street Pumping Station contains four Low Service Level distribution system pumps, Nos. 4 through 7. New pumps and motors were installed in 2001 with rated capacity of 7,000 gpm (10.1 mgd) at 230 feet. Both the transfer and distribution system pumps take suction from the 51st Street Reservoirs. Data on the 51st Street pumping units is given in Table 6-4.

Table 6-4 51st Street Pumping Station

Pump No.	Make	Rated Capacity		Head	Pump Motor	
		(gpm)	(mgd)	(ft)	(hp)	(rpm)
1[1]	Ingersoll-Dresser	10,500	15.1	185	750	900
2[1]	Ingersoll-Dresser	10,500	15.1	185	750	900
3[1]	Ingersoll-Dresser	10,500	15.1	185	750	900
4[2]	Ingersoll-Dresser	7,000	10.1	230	500	900
5[2]	Ingersoll-Dresser	7,000	10.1	230	500	900
6[2]	Ingersoll-Dresser	7,000	10.1	230	500	900
7[2]	Ingersoll-Dresser	7,000	10.1	230	500	900

(1)Transfer pumps – new in 2001.
(2)Low Service Level distribution system pumps – new pumps and motors in 2001.

6.3.1.3 "A" Street Pumping Station (Low Service Level) and Reservoirs

The "A" Street Pumping Station and Reservoirs are located near "A" Street, in the vicinity of Antelope Park. The three ground storage reservoirs have a total capacity of 22.0 million gallons and are supplied through the 36-inch transfer main from the 51st Street Pumping Station and through a 36-inch transfer main from the Vine Street Reservoirs. Reservoirs No. 4 and 5 were demolished early in the Year 2017 which reduced the total storage capacity by 6.0 million gallons. The remaining three reservoirs have different overflow elevations. However, the reservoirs are interconnected and float together establishing a common hydraulic gradient. Data on the "A" Street Reservoirs is shown in Table 6-5.

Table 6-5 "A" Street Reservoirs

Reservoir No.	Capacity MG	Ceiling or Overflow Elevation (ft)	Floor Elevation (ft)
6	6.0	1190.8	1174.8
8	8.0	1190.8	1171.5
9	8.0	1190.1	1175.0

The "A" Street Pumping Station, constructed in Year 1984, is a dual level pumping facility that discharges to the Low and High Service Levels. The station contains two Low Service Level pumps, Nos. L1 and L2, each with a rated capacity of 6,300 gpm (9.1 mgd) at 155 feet and two high service pumps Nos. H1 and H2 with a rated capacity of 6,300 gpm (9.1 mgd) at 265 feet. Three "satellite" pumps are located at the "A" Street facilities in three separate buildings. Satellite 8 discharges to the Low Service Level. Satellites 9 and 10 discharge to the High Service Level. All Low and High Service Level pumps take suction from the adjacent reservoirs. Data on the "A" Street pumping units is shown in Table 6-6.

Table 6-6 "A" Street Pumping Station

Pump NO.	Make	Rated Capacity (gpm)	Rated Capacity (mgd)	Head (ft)	Pump Motor (hp)	Pump Motor (rpm)
Low Service Level						
L1(1)	Patterson	6,300	9.1	155	350	1,200
L2(1)	Patterson	6,300	9.1	155	350	1,200
Sat. 8(1)	Flowserve	7,200	10.4	155	450	1,200
High Service Level						
H1(2)	Patterson	6,300	9.1	265	600	1,800
H2(2)	Patterson	6,300	9.1	265	600	1,800
Sat. 9(2)(3)	Flowserve	6,300	9.1	250	500	1,200
Sat. 10(2)(3)	Flowserve	6,300	9.1	250	500	1,200

(1)Low Service Level.
(2)High Service Level.
(3)Pumps Replaced in 2010.

6.3.1.4 Vine Street Reservoir

The Vine Street Reservoir is located just northeast of the intersection of Skyway Road and Vine Street. The reservoir was expanded from 10.0 MG to 20.0 MG in Year 2001. It floats on the Low Service Level, has an overflow elevation of 1,313 feet, a sidewater depth of 30 feet, and is normally operated between 18 and 27 feet.

The reservoir provides suction storage for the adjacent Vine Street Pumping Station, which supplies the High and Southeast Service Levels. The reservoir can also be used as a supply to transfer water through the 36/24-inch gravity transfer main to the "A" Street Reservoirs. A flow control/transfer vault is located adjacent to the Vine Street Pumping Station which regulates the flow of water from the Vine Street West Pumping Station into the 36/24-inch transfer main. Typically, water is transferred by gravity through the check valve in the transfer vault from the Vine Street to "A" Street Reservoirs. With both the Vine Street and "A" Street Reservoirs full, and using only the Low Service Level hydraulic gradient, the maximum delivery by gravity through the 36/24-inch transfer line is about 15 mgd according to the hydraulic model (18 mgd according to meter data). Using the discharge gradient from the Vine West pumps, the maximum transfer is increased to about 21 mgd through the PRV in the transfer vault. This pumped method of transferring water to "A" Street is not used very often as the gravity method does not require additional energy.

6.3.1.5 Pioneers Park Reservoir

The Pioneers Park Reservoir is located near the north entrance to Pioneers Park. The four million gallon reservoir floats on the Low Service Level, has an overflow elevation of 1,313 feet, a sidewater depth of 54 feet and is normally operated between 46 and 51 feet.

6.3.2 High Service Level

The High Service Level serves the areas south and southeast of the Low Service Level. It is supplied by the "A" Street and Vine Street Pumping Stations. The High Service Level static hydraulic gradient of 1,420 feet is established by the overflow elevations of the Southeast and South 56th Street Reservoirs.

6.3.2.1 "A" Street Pumping Station (High Service Level)

The "A" Street Pumping Station contains two High Service Level pumps, Nos. H1 and H2, each with a rated capacity of 6,300 gpm (9.0 mgd) at 265 feet. The "A" Street facilities also contain two satellite pumping stations, Nos. 9 and 10, each with a rated capacity of 6,300 gpm (9.0 mgd) at 250 feet that discharge to the High Service Level. Data on the "A" Street pumping units is shown above in Table 6-6.

6.3.2.2 Vine Street Pumping Stations

The Vine Street Pumping Stations are located at the Vine Street Reservoir site and take suction from the Vine Street Reservoir and the 54/48-inch Low Service transmission main from the Northeast Pumping Station.

The High Service Level Station (Vine Street West) contains four pumps. Pump No. 1 has a rated capacity of 10,500 gpm (15.0 mgd) at 115 feet and is equipped with an eddy current coupling which is inoperable. As part of an ongoing electrical rehabilitation project at the facility, Pump No. 1 is being removed. Pump Nos. 2 through 4 have a rated capacity of 14,000 gpm (20.2 mgd) at 115 feet. Space is available for a fifth pump.

The Southeast Service Level Station (Vine Street East) was constructed in Year 2001 in conjunction with expansion of the Vine Street Reservoir. The station contains two pumps each rated 7,000 gpm (10.1 mgd) at 210 feet. Formerly, one variable speed drive was used to operate either of the two pumps but was taken out of service in Year 2012. There is space available for a third pump. The facility is designed to accommodate 20 mgd pumps in each of the three pump slots.

Vine Street Pumping Station (West) is currently being rehabilitated under a separate construction project. This will include removal of Pump No. 1 which is no longer functional, and complete replacement of the Motor Control Line-Up and Motor Control Center which serves the facility. Data on the Vine Street pumping units is shown in Table 6-7.

Table 6-7 Vine Street Pumping Stations

Pump NO.	Make	Rated Capacity (gpm)	Rated Capacity (mgd)	Head (ft)	Pump Motor (hp)	Pump Motor (rpm)
High Service Level						
H2(1)	Worthington	14,000	20.2	115	500	1,175
H3(1)	Worthington	14,000	20.2	115	500	1,175
H4(1)	Worthington	14,000	20.2	115	500	1,175
Southeast Service Level						
SE1(2)	Ingersoll	7,000	10.1	210	450	895(4)
SE2(2)	Ingersoll	7,000	10.1	210	450	895(4)

(1)High Service Level.
(2)Southeast Service Level.
(3)Pump H1 is currently being removed under an electrical rehabilitation project.
(4)Common variable speed drive taken out of service 2012.

6.3.2.3 South 56th Street Reservoir and Pumping Station

The South 56th Street Reservoir is located southwest of the intersection of 56th Street and Pine Lake Road. The 4.0 million gallon reservoir floats on the High Service Level, has an overflow elevation of 1,420 feet, a sidewater depth of 62 feet, and is normally operated between 53 and 59.5 feet.

In Year 1998 a re-pumping station was added at the reservoir site. The station contains three pumps each rated 3,125 gpm (4.5 mgd) at 50 feet. The pumping station is intended to be used to increase pressures in the southern portion of the High Service Level under high flow conditions. Records indicate that the station was never used and is slated for demolition, except that an existing PRV from the Southeast Service Level to the High Service Level shall be maintained. Data on the South 56th Street pumping units is shown in Table 6-8.

Table 6-8 South 56th Street Pumping Station

Pump No.	Make	Rated Capacity (gpm)	Rated Capacity (mgd)	Head (ft)	Pump Motor (hp)	Pump Motor (rpm)
1	General Signal	3,125	4.5	50	50	1,170
2	General Signal	3,125	4.5	50	50	1,170
3	General Signal	3,125	4.5	50	50	1,170

6.3.2.4 Southeast Reservoir

The Southeast Reservoir is located near the intersection of South and 84th Streets. The 5.0 million gallon reservoir floats on the High Service Level, has an overflow elevation of 1,420 feet, a sidewater depth of 60 feet, and is normally operated between 51 and 58 feet. The reservoir also provides suction storage for the adjacent Southeast Pumping Station, which supplies the Southeast Service Level.

6.3.3 Belmont Service Level

The Belmont Service Level serves the northwest part of the City, including Lincoln Municipal Airport.

The Belmont Service Level is supplied by the Belmont, Merrill Street, and Pioneers Pumping Stations. The Belmont Service Level static hydraulic gradient of 1,400 feet is established by the overflow elevation of the Air Park and NW 12th Street Reservoirs.

6.3.3.1 Belmont Pumping Station

The Belmont Pumping Station is located southwest of the intersection of 14th and Superior Streets. The Belmont Pumping Station takes suction from 30-inch and 24-inch Low Service Level mains. It contains four pumps that pump to two 24-inch discharge mains.

The impeller in Pump No. 1 was replaced in Year 1999. A new impeller was installed in Pump No. 2 in Year 1990. Pump No. 3 was replaced in Year 2001 and Pump No. 4 was installed in Year 1990. A shared adjustable frequency drive for Pump Nos. 2 and 4 was removed in Year 2001. Data on the Belmont pumping units is shown in Table 6-9.

Table 6-9 Belmont Pumping Station

Pump No.	Make	Rated Capacity		Head	Pump Motor	
		(gpm)	(mgd)	(ft)	(hp)	(rpm)
1	Allis-Chalmers	4,200	6.1	135	200	1,170
2	Allis-Chalmers	4,200	6.1	135	200	1,170
3	Ingersoll-Dresser	6,300	9.1	135	300	1,185
4	Allis-Chalmers	6,300	9.1	135	300	1,185

6.3.3.2 Merrill Pumping Station

The Merrill Pumping Station is located near the intersection of 26th and Merrill Streets and is capable of supplying water to the Low Service Level. The pumps take suction from the 51st Street Pumping Station 36-inch transfer main. As part of the Year 2001 pumping station modifications project, a shared adjustable frequency drive was removed, and constant speed motors were installed on both units, allowing both to be operated at the same time. City staff has reported that the Merrill Pumping Station is not currently used since it is undersized for current demands, but should be maintained until scheduled main improvements in the vicinity are completed. The pumping station is scheduled for demolition, but the existing standpipe shall be maintained to provide surge protection of the 36-inch cast iron main from 51st Street to “A” Street. Data on the Merrill pumping units is shown in Table 6-10.

Table 6-10 Merrill Pumping Station

Pump No.	Make	Rated Capacity (gpm)	Rated Capacity (mgd)	Head (ft)	Pump Motor (hp)	Pump Motor (rpm)
1	Allis-Chalmers	2,600	3.7	215	200	1,760
2	Allis-Chalmers	2,600	3.7	215	200	1,760

6.3.3.3 Pioneers Pumping Station

The Pioneers Pumping Station is located southeast of the intersection of Coddington and West Van Dorn Streets, and became operational in Year 2005. The station contains three pumps that boost from the Low Service Level to the Belmont Service Level. There is space for addition of a fourth pump in the station. Data on the Pioneers Pumping Station is shown in Table 6-11.

Table 6-11 Pioneers Pumping Station

Pump No.	Make	Rated Capacity (gpm)	Rated Capacity (mgd)	Head (ft)	Pump Motor (hp)	Pump Motor (rpm)
1	Fairbanks-Morse	1,400	2.0	105	60	1,195
2	Fairbanks-Morse	2,100	3.0	105	75	1,190
3	Fairbanks-Morse	3,500	5.0	105	125	1,185

6.3.3.4 Air Park Reservoir

The Air Park Reservoir is located northwest of the intersection of West Superior and Northwest 54th Streets. The 3.0 million gallon reservoir floats on the Belmont Service Level, has an overflow elevation of 1,400 feet, a sidewater depth of 95 feet, and is currently operated between 77 and 90 feet.

6.3.3.5 NW 12th Street Reservoir

The NW 12th Street Reservoir is located north of the intersection of Alvo Road and Northwest 12th Street. The reservoir was placed into service in summer Year 2000. The 4.5 million gallon reservoir floats on the Belmont Service Level, has an overflow elevation of 1,400 feet, a sidewater depth of 75 feet, and is currently operated between 57 and 70 feet. The NW 12th Street Reservoir has an electronically actuated fill valve that can be used as an altitude valve when desired to balance flows with the Air Park Reservoir.

6.3.4 Southeast Service Level

The Southeast Service Level serves the high ground elevations in the southeastern section of the City. The Southeast Service Level is supplied by the Southeast Pumping Station and the Southeast Pumps from the Vine Street East Pumping Station. The Southeast static hydraulic gradient of 1,500 feet is currently established by the overflow elevation of the Yankee Hill Reservoir. Upon completion of the Yankee Hill Reservoir in Year 2003, the Pine Lake Reservoir which previously served the Southwest Service Level was demolished.

6.3.4.1 Vine Street Pumping Station

As previously noted in paragraph 4.3.3.2, two 10-mgd pumps at the Vine Street Pumping Station serve the Southeast Service Level.

6.3.4.2 Southeast Pumping Station

The Southeast Pumping Station is located northwest of the Southeast Reservoir, from which the pumps take suction.

The impeller in Pump No. 1 was replaced in Year 1999. A new impeller was installed in Pump No. 2 in Year 1999. Pump No. 3 was replaced in Year 2001 and Pump No. 4 was installed in Year 1988. A shared adjustable frequency drive for Pump Nos. 2 and 4 was removed in Year 2001. Data on the Southeast pumping units is shown in Table 6-12.

Table 6-12 Southeast Pumping Station

Pump No.	Make	Rated Capacity (gpm)	Rated Capacity (mgd)	Head (ft)	Pump Motor (hp)	Pump Motor (rpm)
1	Allis-Chalmers	4,200	6.1	155	200	1,180
2	Allis-Chalmers	4,200	6.1	155	200	1,170
3	Ingersoll-Dresser	6,300	9.1	155	350	1,185
4	Allis-Chalmers	6,300	9.1	155	350	1,185

6.3.4.3 Yankee Hill Reservoir

The Yankee Hill Reservoir is located south of the intersection of 84th Street and Yankee Hill Road. The 10.0 million gallon reservoir floats on the Southeast Service Level and has an overflow elevation of 1,500 feet, a sidewater depth of 75 feet, and is normally operated between 64 and 71 feet.

6.3.5 Cheney Service Level

The Cheney Service Level was placed into service in Year 2001 to serve high ground in the southeast corner of the City. A portion of the existing Southeast Service Level was converted to the Cheney Service Level. The Cheney Booster Pumping Station (BPS) was installed in Year 2001 in the northeast corner of the intersection of South 84th Street and Pine Lake Road. The Cheney Service Level initially operated as a closed system with no floating storage. In Year 2018, the Yankee Hill Pumping Station was constructed at the site of the Yankee Hill Reservoir. It now serves the Cheney Service Level with Cheney BPS remaining for backup service.

The static hydraulic gradient is established by the Cheney Elevated Reservoir which has an overflow 1,580 feet.

6.3.5.1 Yankee Hill Pumping Station

The Yankee Hill Pumping Station was constructed in Year 2018 at the site of the Yankee Hill Reservoir and has been recently put into service. The Yankee Hill Pumping Station now serves as the primary pumping station for the Cheney Service Level and contains four pump bays with three pumps installed. Current pumping station firm capacity is 4 mgd. The pumping station is designed for ultimate total capacity of 24 mgd (18 mgd firm). Data on the current Yankee Hill pumping units is shown in Table 6-13. The station also contains a PRV which is needed when Cheney Reservoir is offline, and it can also be used to cycle Cheney Reservoir for water quality. The Yankee Hill Pumping Station and the Northwest 12th Pumping Station are the only pumping stations in the distribution system which have connected backup power generation.

Table 6-13 Yankee Hill Pumping Station

Pump No.	Make	Rated Capacity		Head (ft)	Pump Motor	
		(gpm)	(mgd)		(hp)	(rpm)
1	Fairbanks-Morse	700	1	110	40	1,800
2	Fairbanks-Morse	2,100	3	115	100	1,200
3	Fairbanks-Morse	2,100	3	115	100	1,200
4	Future					

6.3.5.2 Cheney Booster Pumping Station

The Cheney Booster Pumping Station is a pre-packaged below-grade pumping station constructed in Year 2001, containing five pumps with a firm capacity of 6.2 mgd. As noted above, the Cheney BPS is now a standby pump station serving backup duty to the newer Yankee Hill Pumping Station. Data on the Cheney pumping units is shown in Table 6-14.

Table 6-14 Cheney Booster Pumping Station

Pump No.	Make	Rated Capacity		Head(1) (ft)	Pump Motor	
		(gpm)	(mgd)		(hp)	(rpm)
1	Paco	130(2)	0.2	175	10	3,500
2	Paco	650	0.9	175	40	3,500
3	Paco	1,400	2.0	175	100	1,750
4	Paco	2,150	3.1	175	125	1,750
5	Paco	2,150	3.1	175	125	1,750

(1)Although pumps are rated at 175 feet of head, the discharge PRV throttles about 75 feet of head at 91 psi to maintain a hydraulic gradient of about 1600 feet.
(2)Pump used only for very low flow conditions.

6.3.5.3 Cheney Elevated Reservoir

The Cheney Elevated Reservoir is located at Breagan Road and South 98th Street. The 2.0 million gallon reservoir floats on the Cheney Service Level and has an overflow elevation of 1,580 feet, a sidewater depth of 40 feet, and is normally operated between 26 and 36 feet.

6.3.6 Northwest Service Level

The Northwest Service Level was placed into service in 2002 to serve new development on high ground in the northern portion of the city near the NW 12th Street Reservoir. The NW 12th Street BPS was installed in Year 2002 at the NW 12th Street Reservoir site. The Northwest Service Level is operated as a closed system with no floating storage. The static hydraulic gradient is established by the pressure reducing valve (PRV) setting on the pumping station discharge. LWS reports that the valve set-point is set at 61 psi which equates to a hydraulic gradient of about 1,460 feet.

6.3.6.1 NW 12th Street Booster Pumping Station

The NW 12th Street Pumping Station is a pre-packaged above-grade pumping station containing five pumps with a firm capacity of 6.3 mgd. Data on the NW 12th Street pumping units is shown in Table 6-15 . Operations of the facility are automated based upon discharge pressures.

Table 6-15 NW 12th Street Booster Pumping Station

Pump No.	Make	Rated Capacity		Head[1]	Pump Motor	
		(gpm)	(mgd)	(ft)	(hp)	(rpm)
1	Paco	150[2]	0.2	100	7.5	3,600
2	Paco	650	0.9	100	25	1,800
3	Paco	1,400	2.0	100	50	1,800
4	Paco	2,200	3.2	100	75	1,800
5	Paco	2,200	3.2	100	75	1,800

(1)Although pumps are rated at 100 feet of head, the discharge PRV throttles about 50 feet of head at 61 psi to maintain a hydraulic gradient of about 1460 feet.
(2)Pump noted to no longer be used.

6.3.7 Pumping Capacity Summary

A summary of total and firm capacities for existing distribution system pumping stations is summarized in Table 6-16. Firm capacity is the capacity with the largest pump out of service.

Table 6-16 Distribution System Pumping Capacity Summary

Service Level	Pumping Station	Number of Pumps	Installed Capacity (mgd)	Firm Capacity (mgd)
Low	51st Street[(1)]	4	40.4	30.3
	Northeast[(1)(3)]	4	70.6	50.4
	"A" Street	3	28.6	18.2
	Total		139.6	98.9
High	"A" Street	4	36.4	27.3
	Vine Street	3	60.6	40.4
	S. 56th Street	3	13.5	9.0
	Total		110.5	76.7
Belmont	Belmont	4	30.4	21.3
	Merrill	2	7.4	3.7
	Pioneers	3	10.0	5.0
	Total		47.8	30.0
Southeast	Vine Street	2	20.2	10.1
	Southeast	4	30.4	21.3
	Total		50.6	31.4
Cheney	Yankee Hill	3	7	4.0
	Cheney[(2)]	5	9.1	6.0
Northwest	NW 12th Street	4[(2)]	9.3	6.1

(1)Transfer pumps not included.
(2)Does not include capacity of small pump.
(3)Northeast Pump No. 6 currently not included as eddy current coupling is inoperable.

6.3.8 Storage Capacity Summary

A summary of floating storage capacities by service level is given in Table 6-17. It is noted that a number of reservoirs provide both floating storage and suction storage to different service levels. Vine Street Reservoir enables stabilization of system pressures in the Low Service Level and can potentially supply peak hourly demands by gravity when the water level in the reservoir is near the overflow. However, because of its proximity to the Vine Pumping Station and the high discharge from the pumping station to the High Service Level, the reservoir will function primarily as a suction storage reservoir. During emergencies, the Vine Street and Southeast Reservoirs will be effective in supplying water by gravity to the Low and High Service Levels, respectively. Under these emergency conditions, marginal pressures would be expected at higher ground elevations.

Table 6-17 Distribution System Floating Storage Capacity Summary

Service Level	Reservoir	Capacity (MG)
Low	Vine Street	20.0
	Pioneers Park	4.0
	Total	24.0
High	Southeast	5.0
	High Service (S. 56th)	4.0
	Total	9.0
Belmont	Air Park	3.0
	NW 12th Street	4.5
	Total	7.5
Southeast	Yankee Hill	10.0
	Total	10.0
Cheney	Cheney	2.0
	Total	2.0
Grand Total		**52.5**

Additional ground storage is provided on the transmission system from the Ashland WTP for repumping to the distribution system as summarized in Table 6-18.

Table 6-18 Transmission Ground Storage Facilities

Reservoir	Capacity (MG)
Northeast	10.0
51st Street	12.0
"A" Street	22.0
Total	**44.0**

The total storage volume - including the transmission ground storage facilities and the distribution system floating storage - is 96.5 MG.

6.4 Distribution System Evaluations

Distribution system evaluations were performed to update the recommendations from the *2014 Master Plan* based on the updated demand projections. Desktop storage and pumping evaluations were performed to determine pumping and storage needs through Year 2032. The hydraulic model was updated to include any recent improvements that have been completed since the *2014 Master Plan* and two design year extended period simulations (EPS) were developed, for Year 2020 and for Year 2032.

6.4.1 Pumping Capacity Evaluation

Facility information for firm pumping capacity, which was detailed in the previous sections, was compared against the maximum day demands by each Service Level, and the pumpage needed into each Service Level, to determine if there are any capacity improvements required in the 12-year CIP. Shown in Table 6-19 is the pumping capacity evaluation for Year 2020 and in Table 6-20 for Year 2032. As can be seen, from a capacity stand-point, there are no pumping capacity deficits through the 12-year CIP that need to be addressed, beyond the pumping improvements already recommended and presented in Chapter 8.

Table 6-19 Year 2020 Pumping Capacity Evaluation

Service Level	Total Firm Pumping Capacity	Maximum Day Demand	Total Pumpage Required(1)	Capacity Surplus/ Deficit
Northwest	6.3	2.5	2.5	3.8
Belmont	26.3	13.6	16.0	10.3
Low	98.9	24.9	95.3	3.6
High	67.7	32.4	54.3	13.4
Southeast	31.4	16.7	21.9	9.5
Cheney (2)	10.2	5.2	5.2	5.0

(1)Total pumpage calculated as the sum of all Service Levels demands that must be transferred through the Service Level, as well as the Service Level's own maximum day demand (Belmont pumpage required include Belmont MD and Northwest MD).
(2)Includes the new Yankee Hill Pumping Station.

Table 6-20 Year 2032 Pumping Capacity Evaluation

Service Level	Total Firm Pumping Capacity	Maximum Day Demand	Total Pumpage Required(1)	Capacity Surplus/ Deficit
Northwest(2)	5.0	2.9	2.9	2.1
Belmont	36.3	16.7	19.6	16.7
Low	114.0	29.4	106.3	7.7
High (3)	67.7	33.4	57.3	10.4
Southeast	41.5	17.5	23.8	17.7
Cheney	7.0	6.4	6.4	0.6

(1)Total pumpage calculated as the sum of all Service Levels demands that must be transferred through the Service Level, as well as the Service Level's own maximum day demand (Belmont pumpage required includes Belmont MD and Northwest MD).

(2)New Northwest Pumping Station firm capacity of 5.0 mgd.

(3)Vine Street Pump H1 excluded from firm capacity because it is anticipated to be removed between 2020 and 2032.

6.4.2 Storage Capacity Evaluation

Storage capacity evaluations were updated from the *2014 Master Plan* to include the updated demand projections and estimated storage needs. Discussion is presented in the following sections related to the storage requirements, but one important aspect in consideration of the desktop, or "on-paper," results is the balancing of storage for equalization and emergency against the impact that new storage volume will have on water quality.

6.4.2.1 2020 Storage Evaluation

Table 6-21 presents this updated information on the storage evaluation for Year 2020. The key rows in Table 6-21 are the last two: Total Surplus/Deficiency - Equalization and Operational, and Total Surplus/Deficiency - All Components.

Table 6-21 Storage Evaluation 2020

Description	Service Level					
	Northwest	Belmont	Low	High	Southeast	Cheney
Average Day Demand, mgd	0.8	7.8	12.3	13.5	6.2	2.1
Maximum Day Demand, mgd	2.5	13.6	24.9	32.4	16.7	5.2
Maximum Hour Demand, mgd	4.9	19.6	35.7	65.7	40.3	13.5
Available Firm Capacity Into SL	6.3	26.3	114.0	82.8	31.4	10.2
Operational Storage(1)	0.0	0.4	0.2	0.3	0.7	0.2
Equalization Storage(2)	0.0	0.0	0.0	0.0	0.9	0.3
Emergency Storage(3)	0.8	4.5	8.3	10.8	5.6	1.7
Fire Storage(4)	0.4	0.4	0.4	0.4	0.4	0.4
Total Storage Requirements	1.2	5.4	9.0	11.6	7.6	2.7
Total Existing Storage @ 35 psi(5)	0.0	2.9	2.4	3.7	10.0	1.0
Total Existing Storage @ 20 psi(6)	0.0	6.1	22.8	7.4	10.0	1.5
Total Surplus/Deficiency - Equalization and Operational	0.0	2.5	2.2	3.4	8.4	0.5
Total Surplus/Deficiency - All Components	-1.2	0.7	13.8	-4.2	2.4	-1.2

(1)Operational storage is the volume above the normal high operating level in each tank.
(2)Required equalization storage is equal to [MH – Total Available Source] * 150 minutes, set as 0 if Available Firm Capacity into SL is greater than MH.
(3)Required emergency storage is 8 hours x MD.
(4)Required fire flow storage is equal to 3,500 gpm for 2 hours.
(5)Total existing storage @ 35 psi from the *2014 Master Plan*.
(6)Total existing storage @ 20 psi from the *2014 Master Plan*.

The first key category of *Total Surplus/Deficiency -Equalization and Operational Storage* identifies areas where storage would be needed to meet the equalization of summer peak hourly demands, up to a design (extremely hot and dry year) maximum day and maximum hour condition. If any Service Level had a deficiency in this category, it would generally call for the addition of storage unless there were surplus pumping capacity that could meet the needs with reliable backup power. However, there are no areas in the system where operational and equalization storage is a concern in Year 2020 in the desktop evaluation. In fact, all but Cheney and Southeast can sustain their maximum hour period without storage should an emergency occur, and a tank were needed to be taken out of service and removed from the system. This shows that there is a significantly robust pumping system in place which will meet current maximum day conditions, and sufficient storage for equalization. A process of system optimization as well as optimization of the CIP using T.O.U controls for a design maximum day demand was not performed in the *2014 Master Plan* or in this update. The City may want to consider using such an approach in the next Master Plan with the recognition that optimization using T.O.U. controls will require a significant level of effort and a large amount of energy-cost data to develop an energy-cost model that can be optimized against the planning CIP to be developed.

The second category, *Total Surplus/Deficiency - All Components* is more nuanced to diagnose when desktop evaluation goals don't look to be met. The reason for this is that emergency storage does not necessarily need to be located in the Service Level where the deficiency is shown on paper. It would be ideal if it were, and improvements in storage are recommended within the 12-year CIP to move towards meeting that ideal. As an example of this, High Service Level shows a deficiency of 4.2 MG of storage in Year 2020. The reason for this, is that the large amount of emergency storage required based on the storage goals developed in the *2014 Master Plan* and used in this update. However, there is a surplus of emergency storage in the Southeast Service Level. Should there be the need to move emergency storage into the High Service Level, reducing pumping into the Southeast Service Level, opening a boundary valve, or pulling directly into Southeast Reservoir with the ST-6 improvement (PRV Southeast SL to High SL - Vault near Southeast PS) would essentially shift the surplus storage from Southeast into High which would satisfy more than half of the emergency storage deficiency. Unless the emergency was a complete outage of Vine Street, Low Service Level also has a much larger storage surplus and flexibility in operations with the ability to move water up to Southeast Service Level, which can be wheeled back to the High Service Level.

It should be noted that many utilities set slightly different goals when it comes to fire and emergency storage. One of the major differences is that the larger of the fire or emergency, is taken and set as a single goal for an emergency/fire category to be added to equalization to come up with a total storage goal. The argument for this is that it is very conservative to assume that the maximum day demand, an emergency event, and a fire occur simultaneously, and designing for this level of risk to meet goals for every year can become cost-prohibitive for a CIP. For the purposes of this update, the same method was followed as in the *2014 Master Plan* to add operational, emergency, fire and equalization to obtain the total surplus/deficiency. The City may want to consider refining these goals in the next Master Plan or may decide to keep them as conservative goals with a much lower risk factor when system planning, albeit with a byproduct of higher CIP costs.

Table 6-21 shows three areas where total storage deficiencies occur "on paper," but there are caveats as to whether they are actual deficiencies:

High Service Level shows a total storage deficiency of 4.2 MG. As discussed previously, the ability to shift some emergency storage from Southeast to High indicates that this deficiency can be partially mitigated through operations in the next several years until the Adams Road Reservoir is completed, currently scheduled for Year 2030.

The Northwest Service Level shows a total storage deficiency related to the emergency requirements. The new Northwest Pumping Station, planned for Year 2020, in conjunction with the existing booster station will increase the available pumping capacity into this Service Level in the interim years until the Northwest Storage Reservoir is put into service, in 2026 at which point the existing booster station may be decommissioned and storage requirements will be met.

The Cheney Service Level shows a total storage deficiency as it did in the *2014 Master Plan*. This is due to the combination of fire storage requirements and the emergency storage requirements. Equalization and operational storage in the Cheney Service Level shows very little deficiency. Improvements to storage within the Cheney Service Level were recommended in Year 2040 in the *2014 Master Plan*. Because of the large impact that adding additional storage to the Cheney Service Level would have on water quality in an already challenging area, it is recommended to maintain the storage improvement in the Cheney Service Level in Year 2040, beyond the 12-year CIP. More so than other Service Levels, growth in the Cheney Service Level has generally occurred at a slower rate than predicted in previous planning projections (*LPlan 2040* and prior planning documents used in previous Master Plans). With the updated planning projections placing slightly more emphasis on infill and redevelopment, growth in the Cheney Service Level should be monitored over the next few years to verify that the rate of growth keeps pace with projections. Should the growth match or exceed the planning projections, in the next Master Plan, it should be determined whether this tank needs to move up some years. If historical precedence continues and growth in the Cheney Service Level occurs much more gradually than the planning projections, this facility might be a candidate to defer beyond the Year 2040 planning horizon. Of note, the Year 2020 Census data will be available in Year 2021 for use in the next Master Plan and data-driven planning projections will be much more straightforward to develop, rather than having to rely on benchmarked base-year estimations of population by Service Level.

Other strategies for Northwest and Cheney Service Levels in the interim years (until storage can be completed) could include targeted "Smart-watering" programs. Other utilities have had success piloting a small low-budget public-relations aspect of informing customers at a high level about how proper setting of irrigation days and times based on address can help. A well-informed targeted public that can be led to understand how storage improvement capital cost ultimately affects rates and how potential deferment is a win-win for all stakeholders, will be likely to participate. Reaching out to local sprinkling installers and discussing irrigation settings based on address has also been done effectively in developing/newly developed areas. These together can reduce drain rates during the first hours of irrigation in targeted areas, like Cheney, resulting in higher pressures for longer.

6.4.2.2 2032 Storage Evaluations

The Storage Evaluation for Year 2032 (12-year CIP) is provided in Table 6-22. Three Service Levels stand-out as having a deficiency in storage in a desktop evaluation.

Belmont Service Level has a total storage deficiency of 0.3 MG. However, with the addition of the I-80 Booster Pumping Station and the Belmont loop, there is a significant amount of surplus pumping capacity and there is also surplus emergency storage in the Northwest Service Level after the addition of the Northwest Storage Facility. Emergency storage surplus from the Northwest Service Level can be used to offset the deficiency in the Belmont Service Level by similar methods described above.

High Service Level shows a deficiency in the desktop evaluation, but the same discussion presented with the 2020 evaluation holds true in Year 2032, and the deficiency is much smaller.

Cheney shows a deficiency in the desktop evaluation in both equalization and total storage. The discussion presented in the previous section details the plan for the Cheney Service Level regarding storage.

Table 6-22 Storage Evaluation 2032

Description	Service Level					
	Northwest	Belmont	Low	High	Southeast	Cheney
Average Day Demand, mgd	1.0	13.6	13.4	12.9	6.5	2.6
Maximum Day Demand, mgd	2.9	16.7	29.4	33.4	17.5	6.4
Maximum Hour Demand, mgd	5.8	19.2	41.4	66.9	42.7	16.8
Available Firm Capacity Into SL	5.0	36.3	114.0	67.7	41.5	7.0
Operational Storage (1)	0.0	0.4	0.2	0.3	0.7	0.2
Equalization Storage(2)	0.1	0.0	0.0	0.0	0.1	1.0
Emergency Storage(3)	1.0	5.6	9.8	11.1	5.8	2.1
Fire Storage(4)	0.4	0.4	0.4	0.4	0.4	0.4
Total Storage Requirements(5)	1.5	6.4	10.4	11.9	7.1	3.8
Total Existing Storage @ 35 psi(6)	1.2	2.9	2.4	5.4	10.0	1.0
Total Existing Storage @ 20 psi(7)	1.7	6.1	22.8	11.4	10.0	1.5
Total Surplus/Deficiency - Equalization and Operational	1.1	2.5	2.2	5.1	9.2	-0.2
Total Surplus/Deficiency - All Components	0.3	-0.3	12.4	-0.5	2.9	-2.3

(1)Operational storage is the volume above the normal high operating level in each tank
(2)Required equalization storage is equal to [MH – Total Available Source] * 150 minutes, set as 0 if Available Firm Capacity into SL is greater than MH.
(3)Required emergency storage is 8 hours x MD
(4)Required fire flow storage is equal to 3,500 gpm for 2 hours
(5)Numbers may not add exactly due to rounding
(6)Total existing storage @ 35 psi from the *2014 Master Plan*
(7)Total existing storage @ 20 psi from the *2014 Master Plan*

6.4.3 Distribution System Modeling Evaluations and Model Updates

The 12-year CIP was evaluated through distribution system modeling using extended period simulations (EPS) for maximum day demand conditions. These EPS scenarios model the system demands and operations for a continuous 24-hour period and include the peak hourly demand (MH), replenishment conditions, and the average total demand over the day equals the maximum day demand for the given year.

The model was updated to include recent pipeline projects that would have a hydraulic impact on the system, generally 12-inch and larger mains. Updates were also made to the facilities surrounding the WTP to improve spatial accuracy. Some updates were necessary to improve modeling for the pumping and transmission network and closer resemble the way it is operated to provide supply into the system (i.e. opening and closing of certain valves to isolate transmission sections). The model demand allocation was updated based on the Year 2018 metered sales data, for which there was a 99 percent spatial match on accounts by usage volume.

6.4.3.1 Model Update - Hourly Peaking Patterns

Diurnal usage patterns by Service Level were developed using SCADA data from 2018 during an average week condition and a maximum week condition. The diurnal pattern represents the hourly usage characteristics for each Service Level and applied to the model junctions provides a realistic hourly usage throughout the 24-hour EPS. Figure 6-2 provides the hourly peaking factors to be applied to the maximum day demands by Service Level for the maximum day EPS evaluations. Figure 6-3 provides the hourly peaking factors to be applied to the average day demands by Service Level for the water age modeling evaluations which will be discussed in the following chapter. As expected, the hourly peaking factors are much greater for a maximum day demand condition than they are for an average day demand condition which is related to the irrigation component of use.

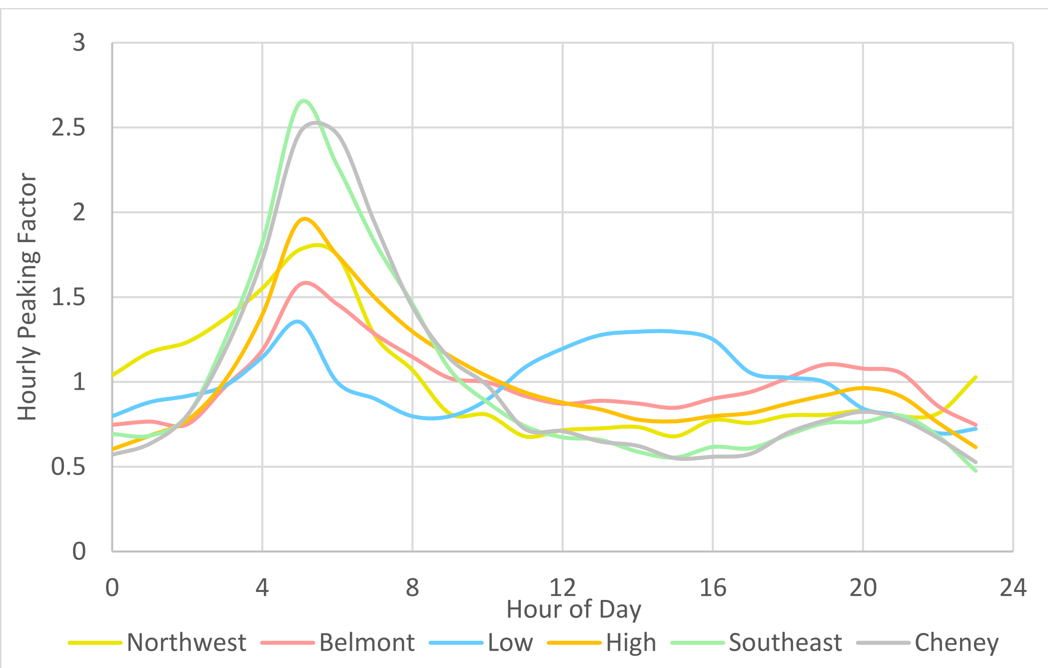


Figure 6-2 Maximum Day Hourly Peaking Factors

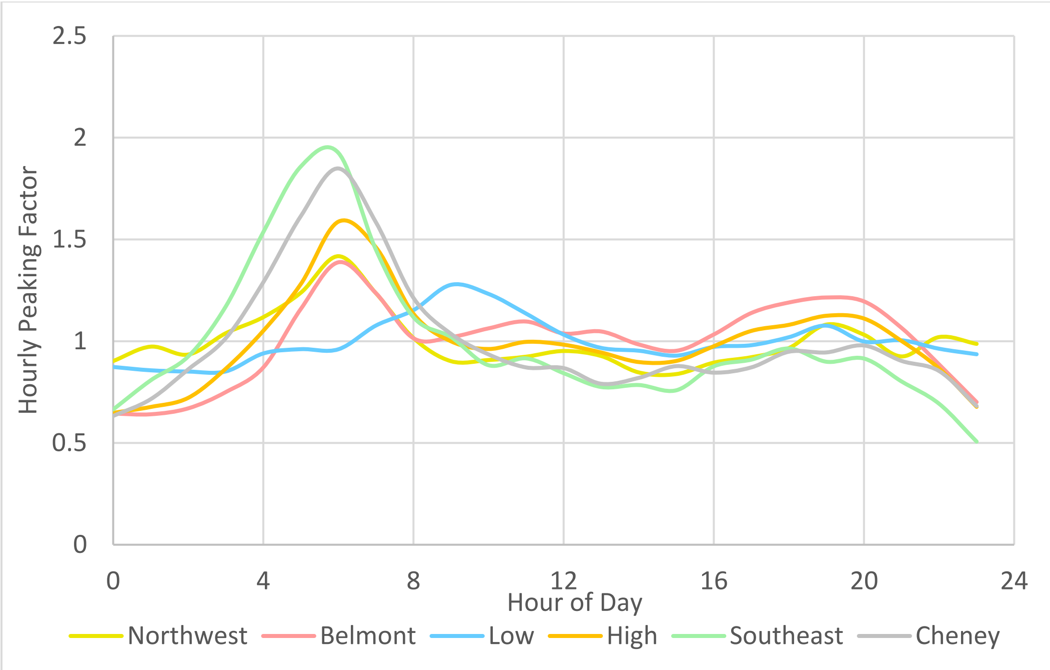


Figure 6-3 Average Day Hourly Peaking Factors

6.4.3.2 Model Update - Modeled Demands by Service Level

Figure 6-4 shows the maximum day hourly demands by Service Level for Year 2020 resulting from the MD:AD peaking factor multiplied by the hourly pattern for a maximum day.

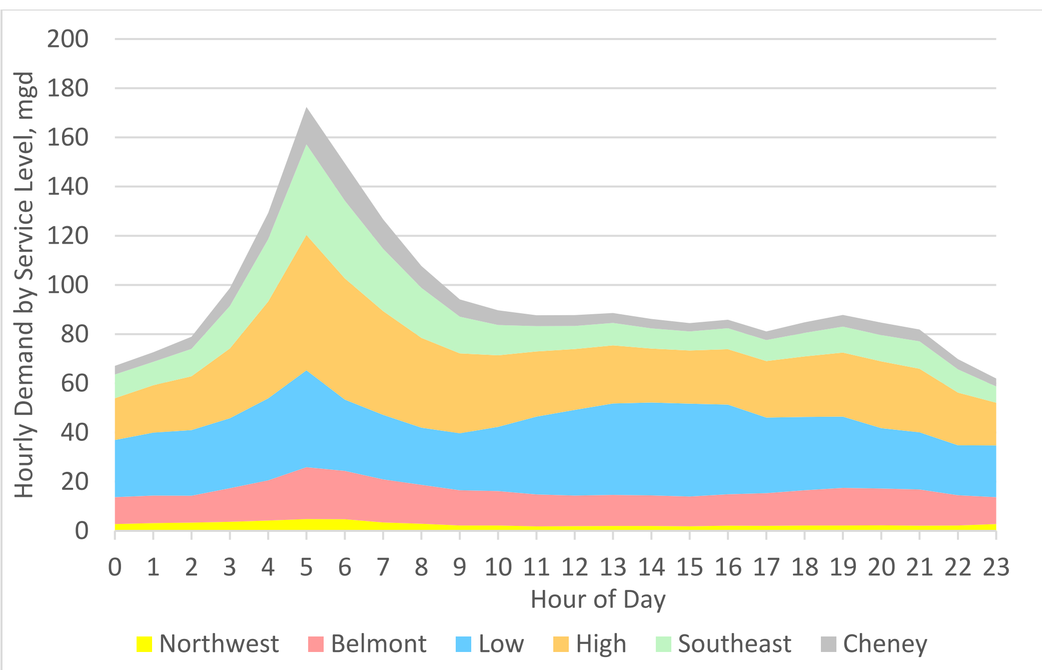


Figure 6-4 Modeled Maximum Day Hourly Usage, 2020

Figure 6-5 shows the average day hourly demands by Service Level for Year 2020, used in the water age modeling evaluations presented in the next Chapter.

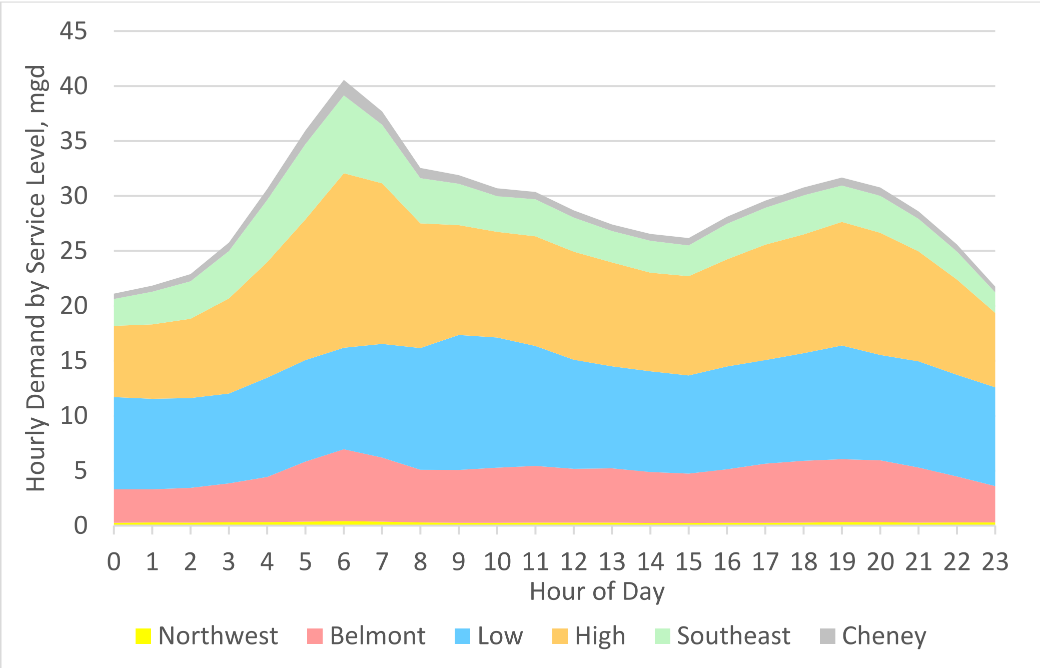


Figure 6-5 Modeled Average Day Hourly Usage, 2020

Figure 6-6 shows the hourly demands by Service Level for the maximum day evaluation for Year 2032.

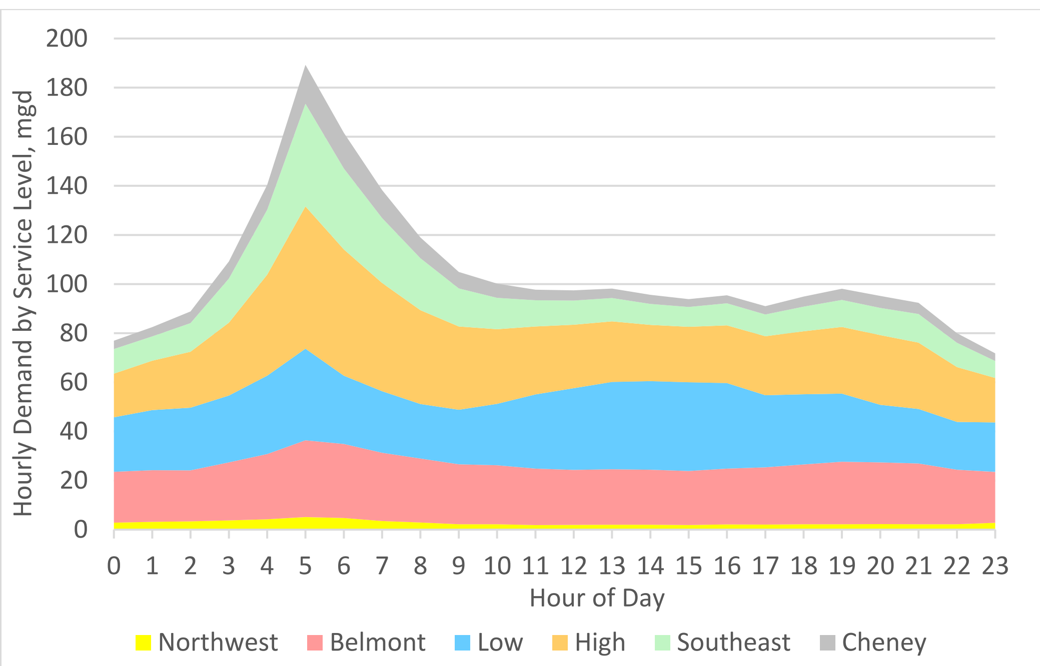


Figure 6-6 Modeled Maximum Day Hourly Usage, 2032

6.5 Focus Area Evaluations

Three focus areas were assessed specifically in this update.

- North 56th Street and I-80.
- Folsom and Old Cheney.
- 27th and Rokeby.

In addition, the City asked Black & Veatch to look at the runway crossing in the Belmont Service Level which was included in the focus areas evaluations.

6.5.1 North 56th Street and I-80

6.5.1.1 *2014 Master Plan* Recommendations

Improvements were recommended in the *2014 Master Plan* in this area to provide for future potential industrial demands north of I-80 along 56th St. The *2014 Master Plan* recommended a 3.0 mgd firm, 6.0 mgd total booster pumping station at I-80 and North 56th St plus improvement mains for development in the area. The timing of this recommendation was for 2018. A 24-inch Belmont North Loop connecting this area was recommended in Year 2035 from North 56th Street to North 14th Street (MT-6). A screen capture of CIP Figure from the *2014 Master Plan* for the area is shown below in Figure 6-7.

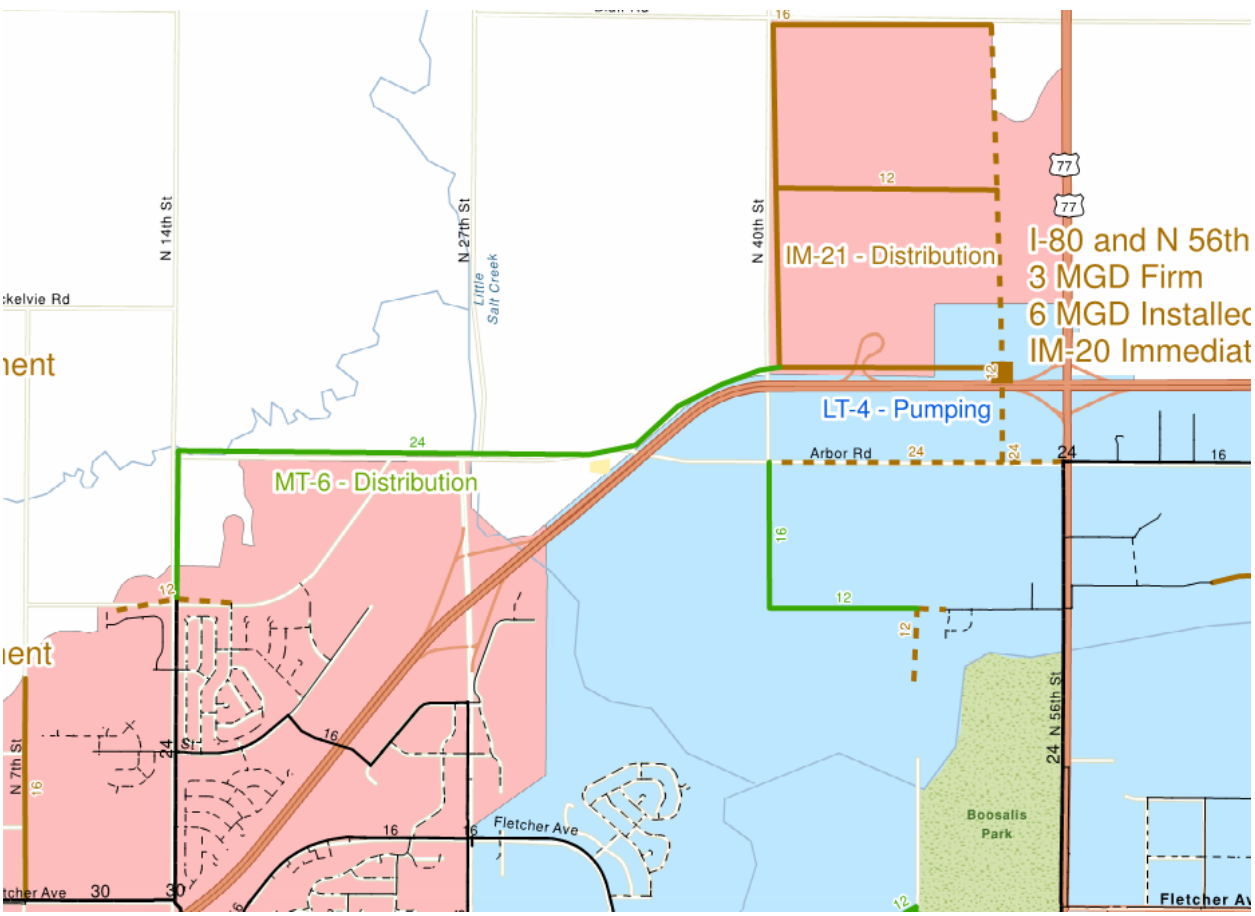


Figure 6-7 ***2014 Master Plan* Recommendations**

6.5.1.2 Update of Large Industrial Demands

Allowances for industrial growth north of I-80 were also included in this update with the exception that the demand projections were increased in this area for evaluation purposes. The demands evaluated for future large users or industrial customers in this development area are provided in Table 6-23.

Table 6-23 Large Use/Industrial Demand Projections

Demand Condition	2020	2025 and Beyond
Average Day Demand	1.75 mgd (25% of 7.0 mgd)	7.0 mgd
Maximum Day Demand	2.7 mgd (25% of 10.8)	10.8 mgd
Maximum Hour Demand	2.7 mgd (25% of 10.8)	10.8 mgd
Seasonal Peak Demand	2.7 mgd (25% of 10.8)	10.8 mgd

6.5.1.3 Recommended Improvements for Reliable Service

With localized large potential demands, it would be critical that this area be provided with reliability in case that a temporary outage of the I-80 and N 56th Street booster pump station (I-80 BPS) occurs, or if there was a main break on the 24-inch main that provides suction for this station. Multiple paths to provide water to an area will also help reduce bottlenecks and the energy cost of moving water. The implementation of a Belmont North Loop, shown in Figure 6-8 in red would provide reliability for consistent service of large demands. The dashed red lines shown on this figure are two different options for looping depending on long-term location of a pumping station. This will give the operational flexibility to meet a large variance in demand north of I-80. The Belmont Loop will expand the Belmont Service Level and will be at the same grade line as the Belmont Service Level. Any development north of I-80 and south of Bluff Road, bounded by N 56th Street on the east and roughly 1st Street on the west, could be part of the Belmont Service Level.

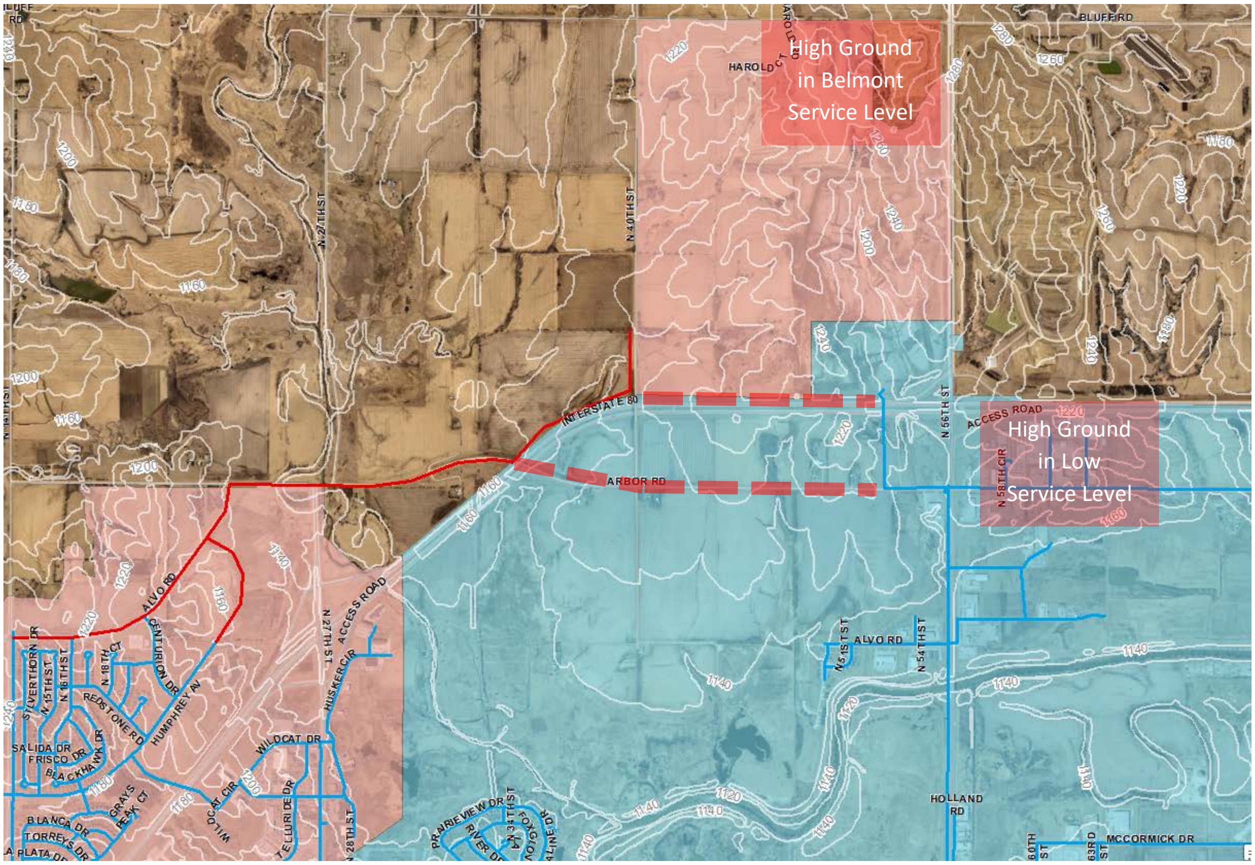


Figure 6-8 **I-80 and N 56th Street Area Map**

6.5.1.4 Modeling Evaluations

Hydraulic modeling scenarios were performed for Year 2020 and 2032 to evaluate the area and the improvements to meet any potential large use/industrial demands. The modeling scenarios were performed as maximum day EPS for a 48-hour period.

The results of the Year 2020 maximum day EPS showed that there is a high ground area in the Low Service Level just north of Arbor Road between N 56th Street and North 70th Street where pressures around 40 psi occur (Figure 6-9). Lincoln Trucking and Arbor Industries fall within this area. Should the I-80 BPS pump between 2 mgd and 4 mgd, these pressures around 40 psi are marginally acceptable, but flows above 4 mgd begin to cause pressure challenges. The Year 2032 evaluations showed that if using the I-80 BPS for almost 11 mgd of transfer, these same areas experience pressures less than 30 psi, with the highest ground around 23 psi.

Because pressures at these locations will be low and drop quickly with higher pumping from the I-80 BPS, three initial options that were evaluated.

- Supplying some of the demand through the Belmont Service Level during maximum demand conditions. Discussion is provided below in the emergency scenario evaluation.
- Constructing a parallel 24-inch improvement along N 56th Street from Superior to Arbor Road and N. 56th Street. This would increase the pressures by about 5 psi. These improvements would be cost-prohibitive for the benefit that they would provide.

- Operating 3 of the 51st Street pumps to Low continuously during periods of high demand. This will increase the pressures by about 5 psi but is energy intensive and operationally challenging.

These options were presented at a workshop with LWS staff and none of these options were noted as optimal solutions, so a fourth option, siting the booster pump station at a location south of I-80, was evaluated. This could provide acceptable pressures to the high ground along Arbor Road while allowing more pumping capacity at the I-80 BPS. Modeling results show that pumping 10.8 mgd on a Year 2032 maximum day, the average suction hydraulic grade line is 1285 feet and the average discharge hydraulic grade line is 1450 feet. This indicates two things: that areas above 1190 feet should be supplied through Belmont Service Level to avoid unacceptably low pressures (less than 40 psi), and that areas lower than 1170 feet should be supplied through the Low Service Level to avoid pressures that are unacceptably high (greater than 120 psi). The location where both requirements are met, is just south of Arbor Rd. along 56th Street. Figure 6-9 shows the area location with existing pipes symbolized with labeled diameters in blue and the future pipes that were in the model for development symbolized with labeled diameters in red. The ideal location of the booster pumping station is within the red rectangle and the yellow rectangle shows where a boundary valve would need to be closed with this site alternative.

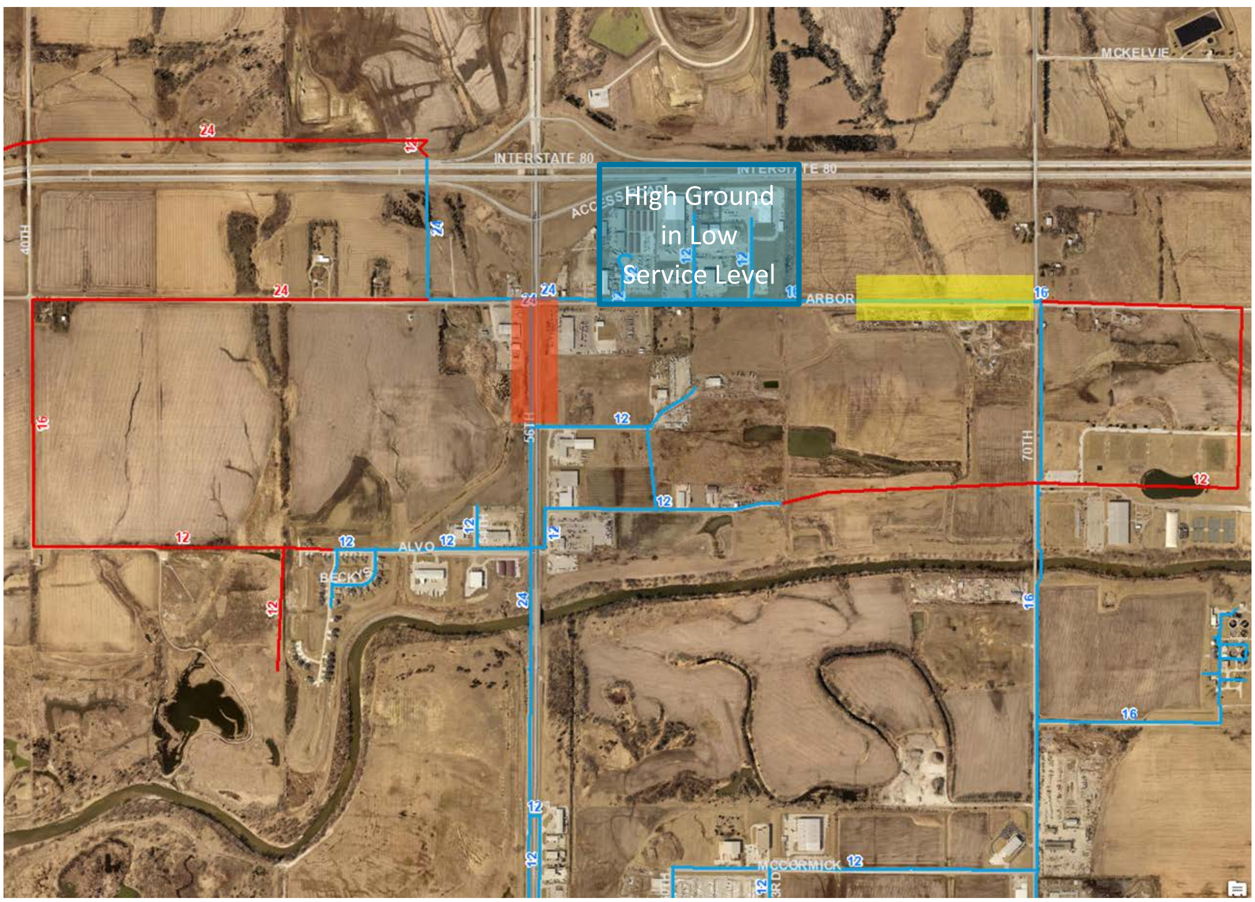


Figure 6-9 Booster Pump Station Alternative Location 1

A second scenario was performed to review the pressures if land cannot be acquired for siting of the booster pump station within the red rectangle shown above in Figure 6-9. This scenario evaluated the booster pumping station along 56th Street, but south of Alvo Rd. The locations of the booster pumping station and the necessary closed valves are show by the red and yellow rectangles respectively in the following Figure 6-10. This boundary configuration does provide the high ground areas along Arbor Rd. with pressures above 40 psi but results in extremely high pressures, up to 140 psi, at areas along Alvo Rd. If it is necessary to construct the booster pump station south of Alvo Rd. because of land acquisition factors, services that are along Alvo Rd. will likely need individual PRVs to reduce the high pressures. However, this would still result in pressures of 140 psi in the distribution system mains.

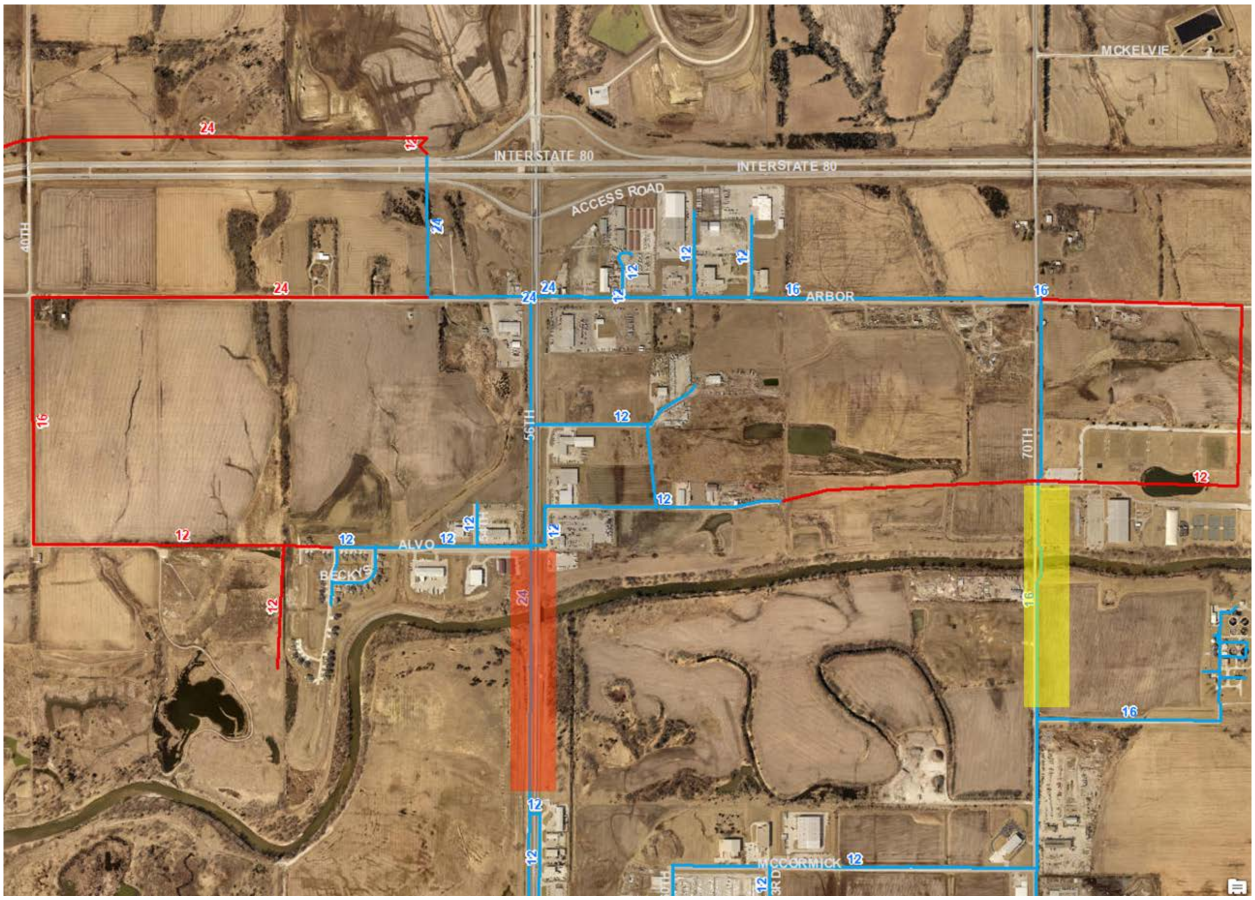


Figure 6-10 Booster Pump Station Alternative Location 2

6.5.1.5 Emergency Scenario

An "emergency" scenario was performed to evaluate supplying a portion of the demand through the Belmont Service Level. Pressures are increased at the Arbor Road location by almost 15 psi and are above 40 psi. However, during this emergency scenario pressures in the development area north of I-80 at high ground may be below 30 psi when moving more supply capacity through the Belmont Service Level.

6.5.1.6 North Loop Sizing

To provide full redundancy, the north loop will need to be 24-inches. If full redundancy is not desirable, a 16-inch will provide approximately 5 mgd to the east before pressures at high ground would drop below 40 psi. Minor sections of the looping as 16-inch mains along Alvo Road at the west in already developed areas could create some headloss, but if the loop mains along Arbor Road are maintained as 24-inch, these bottlenecks can be mitigated. The North Loop, while not currently needed to transfer water into existing areas of the Belmont Service Level, can be used to move water into the Belmont Service Level from the Low Service Level at higher capacities in the long-term plan.

6.5.1.7 Airport Runway Crossing Evaluation

The Year 2020 and Year 2032 EPS scenarios were used to evaluate the impact of removing the 16-inch main that crosses the runway in the Belmont Service Level. This area is shown in Figure 6-11. During the Year 2032 MD EPS scenario, the maximum flow through this main was only about 1.5 mgd, but flow moves through this main in both directions. Primarily during the morning peak, it moves west-east and during the periods of low demands (early morning/late night) it moves east-west. A second scenario was performed with this main closed to represent that it is out of service. Figure 6-12 shows the hydraulic grade line at the junctions to the west and east of the runway. The areas to the east of the runway will experience pressure about 4 psi (10 ft of HGL) lower during maximum hour conditions but there is additional pumping capacity at the Belmont Pumping Station that can be used if this leads to any low-pressure concerns. The areas to the west of the runway will experience a maximum pressure differential of about 2 psi (5 ft of HLG) during storage replenishment hours. Modeling indicates that the general conveyance capacity through this main is not needed for a Year 2032 maximum day non-emergency scenario. If the City is considering removal or abandonment of this main, the impact to water quality, fire flow and reliability should be further evaluated.

Figure 6-11 Airport Runway 16-Inch Main Crossing

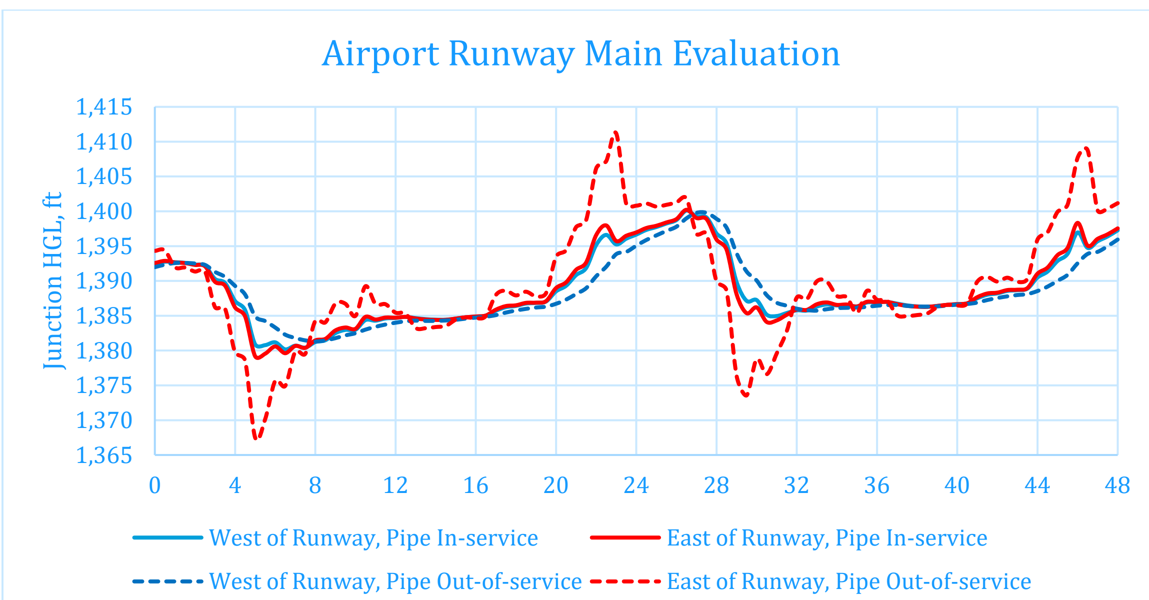


Figure 6-12 Airport Runway Pipe Abandoned/Retired - Pressure Evaluation

6.5.2 Folsom and Old Cheney

This area is currently fed from the Belmont Service Level through a 16-inch main coming south down Folsom from Old Proctor. Figure 6-13 shows the area with existing pipes (black) and system extension pipes (red-immediate, green-6 years, blue-12 years). From the planning projections provided, this area is anticipated to grow from a population of 550 (also existing population) to over 4,000 between Year 2026 and Year 2040. The 16-inch main alone provides enough capacity to serve the projected population at acceptable pressures, especially since this area is at relatively lower ground elevations considering the Belmont Service Level operating hydraulic grade line. However, it is served by a long 16-inch main and has no other redundant feed, so it is completely reliant on this main having no service interruptions. During maximum day conditions, the hydraulic grade line in the Belmont Service Level at Old Cheney and Folsom in the Belmont Service Level west of Wilderness Park operates near the same hydraulic grade line as the High Service Level at Old Cheney and 1st Street, east of Wilderness Park. This despite these areas being in different service levels. Modeling evaluations show that a pipe improvement along Old Cheney through Wilderness Park from the High Service Level to the Belmont Service Level could flow bi-directionally but only transfers 0.7 mgd east to west (High to Belmont) and 0.4 mgd from west to east (Belmont to High) at the maximum. This improvement would provide redundancy in the case that the 16-inch main along Old Folsom was taken out of service for maintenance or a break should occur. Because the operating grade lines of the Belmont Service Level and the High Service Level at the location of the intertie might be different during an average day scenario, a bi-directional control valve is recommended. If reliability is determined not to be a concern for the Folsom and Old Cheney focus area, the 12-inch main along Old Cheney connecting the two service levels is not needed for hydraulic conveyance.

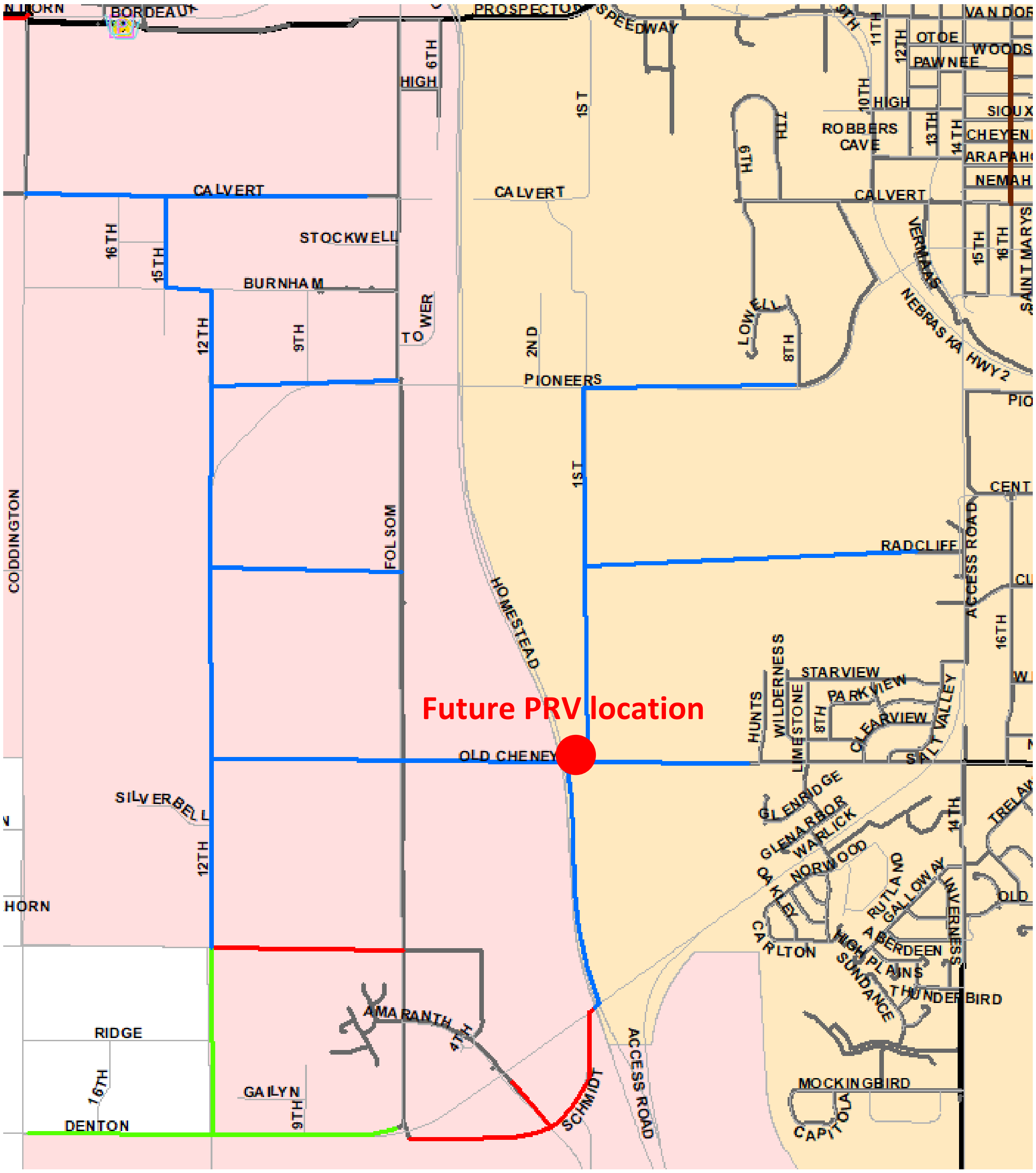


Figure 6-13 **Folsom and Old Cheney Development Area**

6.5.3 27th and Rokeby

Growth is expected in this area which is currently supplied at High Service Level pressure while areas at the north and to the east are supplied at Southeast Service Level pressure. There is an existing PRV between Southeast and High Service Levels at Williamson and Yankee Hill. Figure 6-14 shows the area with existing pipes (black) and system extension pipes (red-immediate, green-6 years, blue-12 years). The orange background is in the High Service Level and the green background is within the Southeast Service Level.

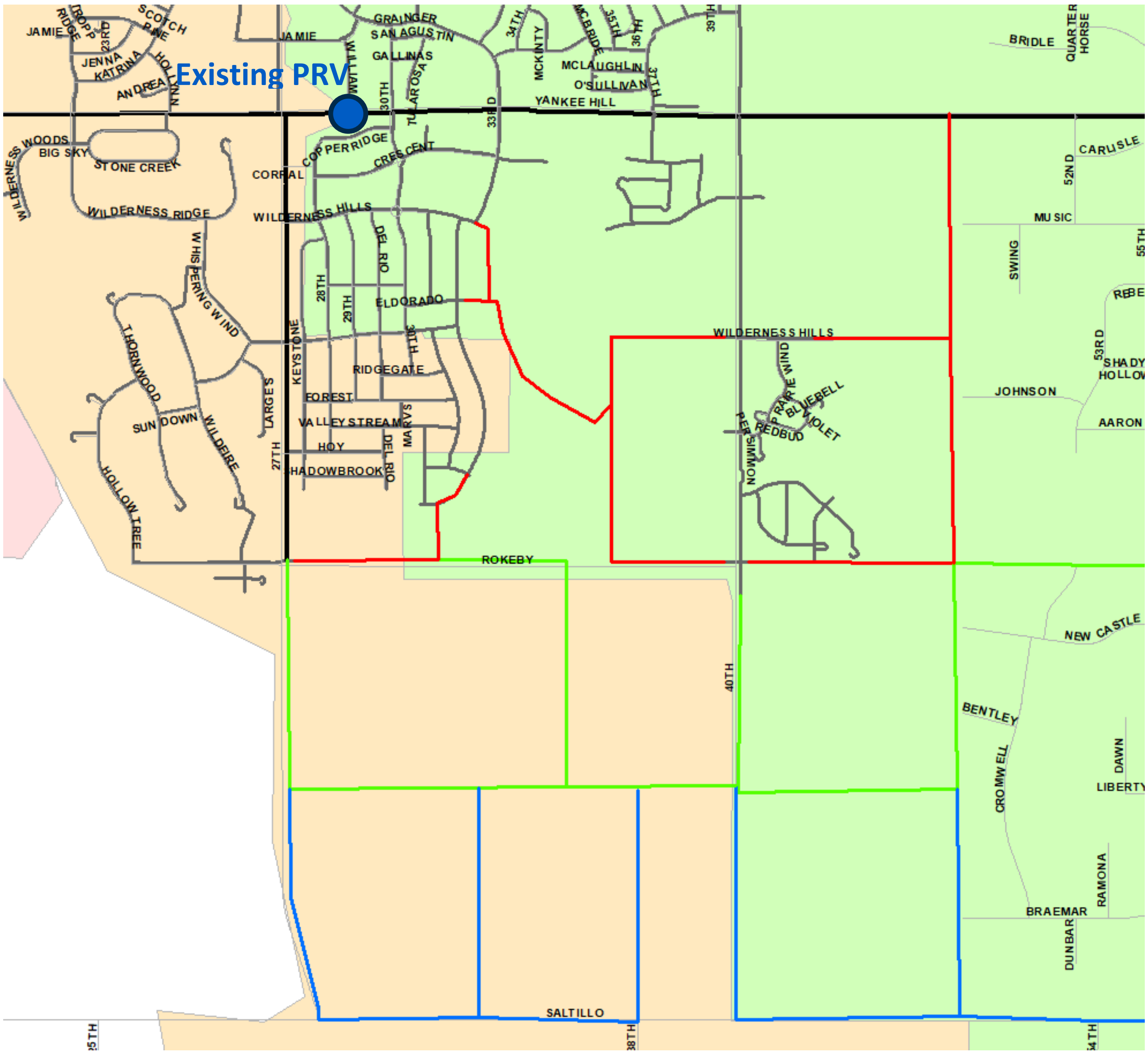
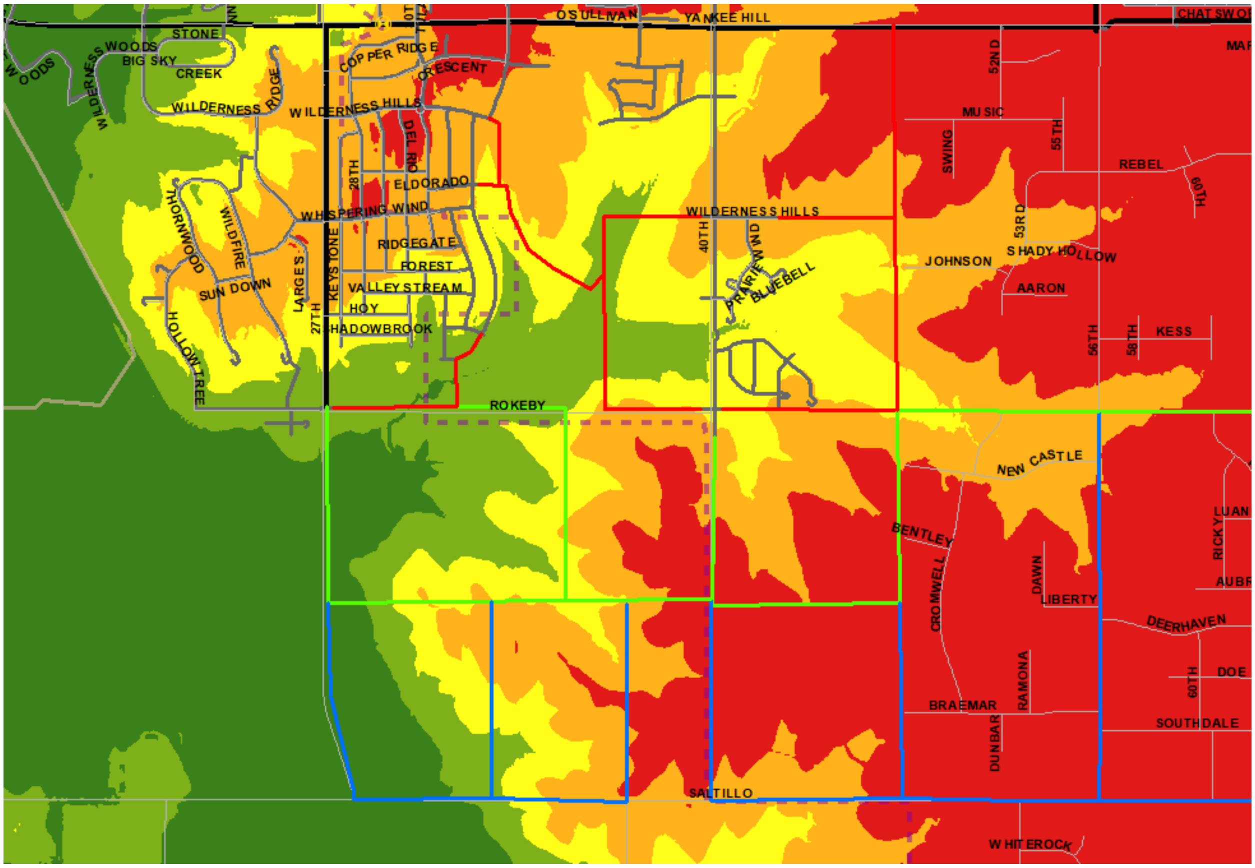


Figure 6-14 27th and Rokeby Existing and Future Pipes w/ Service Level Boundaries

During a Year 2032 maximum day condition, modeled operating hydraulic grade lines in this area in High Service Level will be between 1390 feet to 1420 feet and the operating hydraulic grade line in the Southeast Service Level will be between 1470 feet to 1490 feet. Because the average difference in the hydraulic grade line is only about 75 feet, much of these areas could be served from either service level. To identify an ideal pressure zone boundary within the area, a digital elevation model was used to show the topographic related pressure at the average range of the anticipated operating hydraulic grade line, shown in Figure 6-15.



Map Color	High Service Level		Southeast Service Level	
	Minimum Pressure, psi	Maximum Pressure, psi	Minimum Pressure, psi	Maximum Pressure, psi
	82 or greater	95 or greater	117 or greater	126 or greater
	72	85	106	115
	61	74	95	104
	50	63	85	93
	Less than 50	Less than 63	Less than 85	Less than 93

Figure 6-15 27th and Rokeby Potential Pressures

The evaluation indicates that the only true design criteria for this area on a maximum day in Year 2032 to meet level of service goals is that any of the dark green areas must be within the High Service Level to avoid pressures greater than 120 psi. Ideally, the boundary would follow the orange areas. Another good guideline is that areas in red be within the Southeast Service Level as much as the network will allow and areas in greens should be within the High Service Level. The current network supports this but as development on the periphery occurs, the network should be reviewed to ensure that the ideal boundaries are followed as closely as possible when distribution extensions are put into service.

6.6 Year 2020 Distribution System Modeling

EPS modeling was performed for the Year 2020 design maximum day demand, with hourly peaking patterns that provide a full hourly view of system behavior, including storage draining during equalization periods and refill of system storage during off-peak periods of the day. These maximum day EPS scenarios provide more details than steady state scenarios with fewer assumptions, namely;

- Storage facilities in the same zone will show how the tanks equalize together over time.
- Tank levels for the maximum hour demands in a steady state scenario are assumed. In an EPS scenario, hourly tank levels are calculated based on the drain rates and the level at a maximum hour is a result of the scenario, rather than assumptive input.
- Pump controls do not need to be assumed at a constant rate like they do for a steady state because the ability exists within an EPS to develop controls that turn pumps off (or ramp down a speed) when tanks are at or near high alarm and turn pumps on (or increase a speed) when tanks are at or near low alarm. EPS scenarios give more control to model a system like it could be operated during a given day.

The scenarios were performed for a 48-hour period which would relate to design maximum day conditions occurring for two consecutive days. Although the two consecutive days of maximum daily demand is not a condition that the system design needs to accommodate, it does provide a look at how the system would respond to such a condition.

System controls were set based on the low and high alarms of storage facilities, with firm pumping capacity at Pumping Stations not to be exceeded (i.e. largest pump not used). The goal of the EPS modeling was to verify the desktop evaluations for storage and pumping capacities and see how the system responds to a design (extremely hot and dry year) demand condition. Results were captured for each Service Level in a visual dashboard for tank levels, pump flows, and discharge HGL and are provided at the end of this Chapter. The operation of the storage facilities (by Service Level), along with high alarms (dotted) and low alarms (dashed) are provided in Figure 6-16 through Figure 6-19. The model results support the desktop evaluation in that there is generally excess pumping capacity and storage to meet Year 2020 maximum day demands and the ability to refill storage during replenishment conditions exists. However, the rapid draft rate of Southeast and S. 56th Street during peak hourly demands that can be seen in Figure 6-17 on the left chart, shows that hydraulic restrictions do occur when pumping into the High Service Level. The addition of a pump at the Vine East Station, scheduled for Year 2020, will allow for more pumping to the Southeast Service Level from the Low Service Level which in turn will not need to be transferred through the High Service Level and the Southeast Pumping Station. This will reduce the hydraulic restrictions of having to pump a portion of the supply to the Southeast Service Level through the High Service Level. Additionally, the Adams Street Reservoir, scheduled in Year 2030, will provide equalization and emergency storage in the High Service Level.

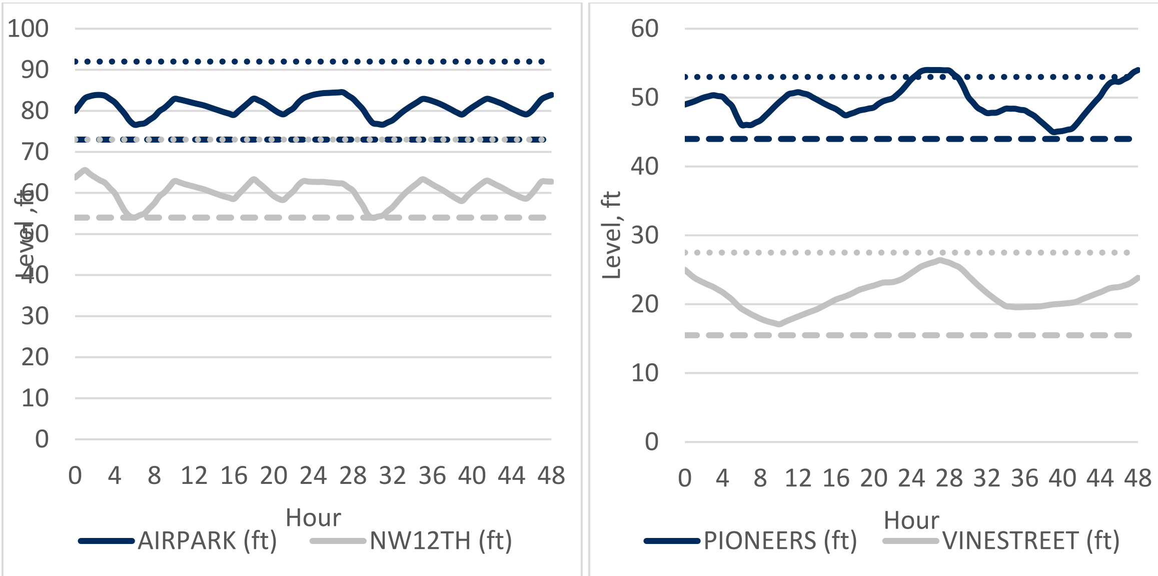


Figure 6-16 **Belmont and Low Storage Levels (2020 MD EPS)**

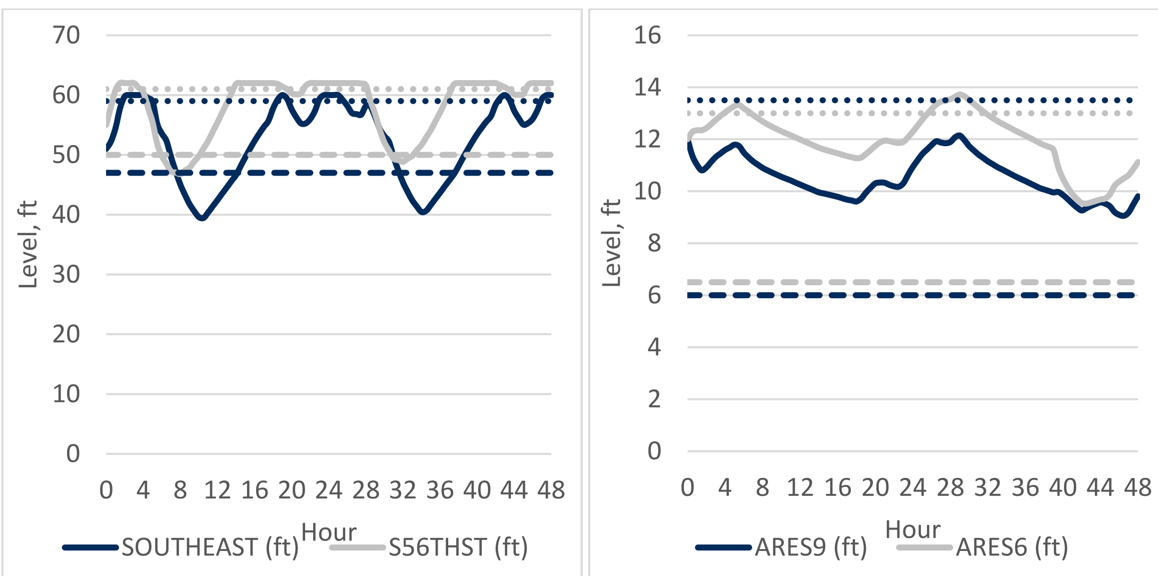


Figure 6-17 **"A" Street and High Storage Levels (2020 MD EPS)**

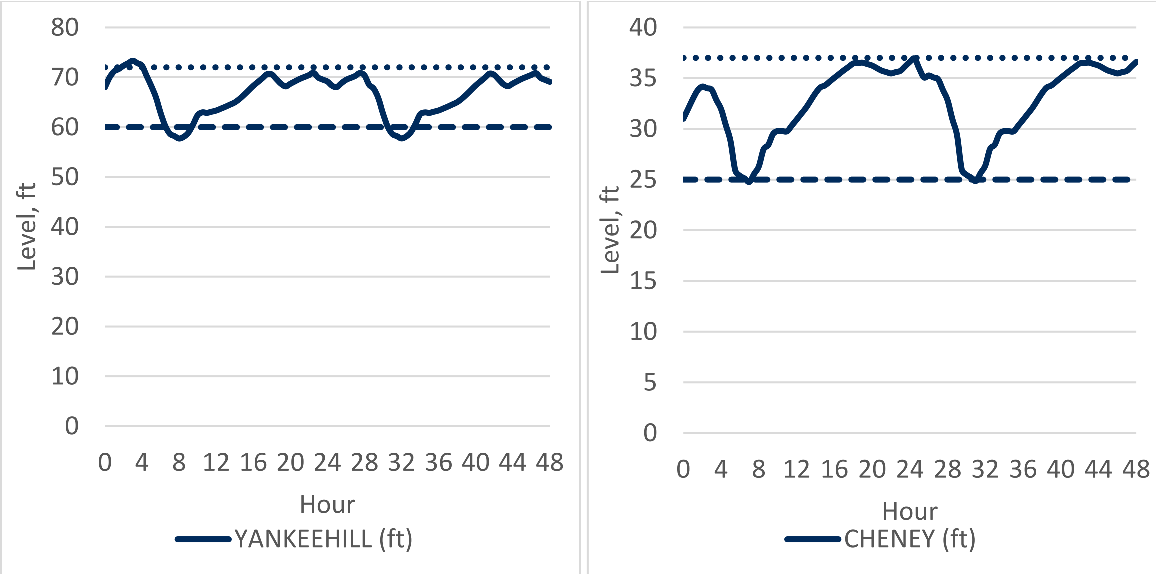


Figure 6-18 Southeast and Cheney Storage Levels (2020 MD EPS)

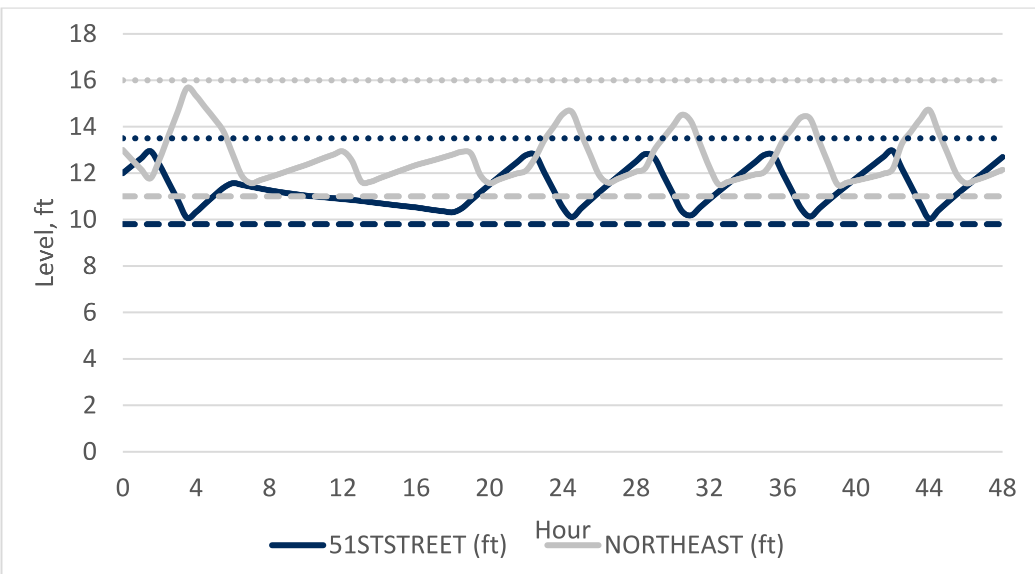


Figure 6-19 51st Street and Northeast Storage Levels (2020 MD EPS)

The average results of the Year 2020 maximum day modeling scenario showing the distribution system flow schematic and transfer through pumping stations and major transmission pipelines is provided in Figure 6-20.

An overall figure showing distribution system pressures, both minimum (left side) and maximum (right side), for the Year 2020 maximum day scenario is provided in Figure 6-21.

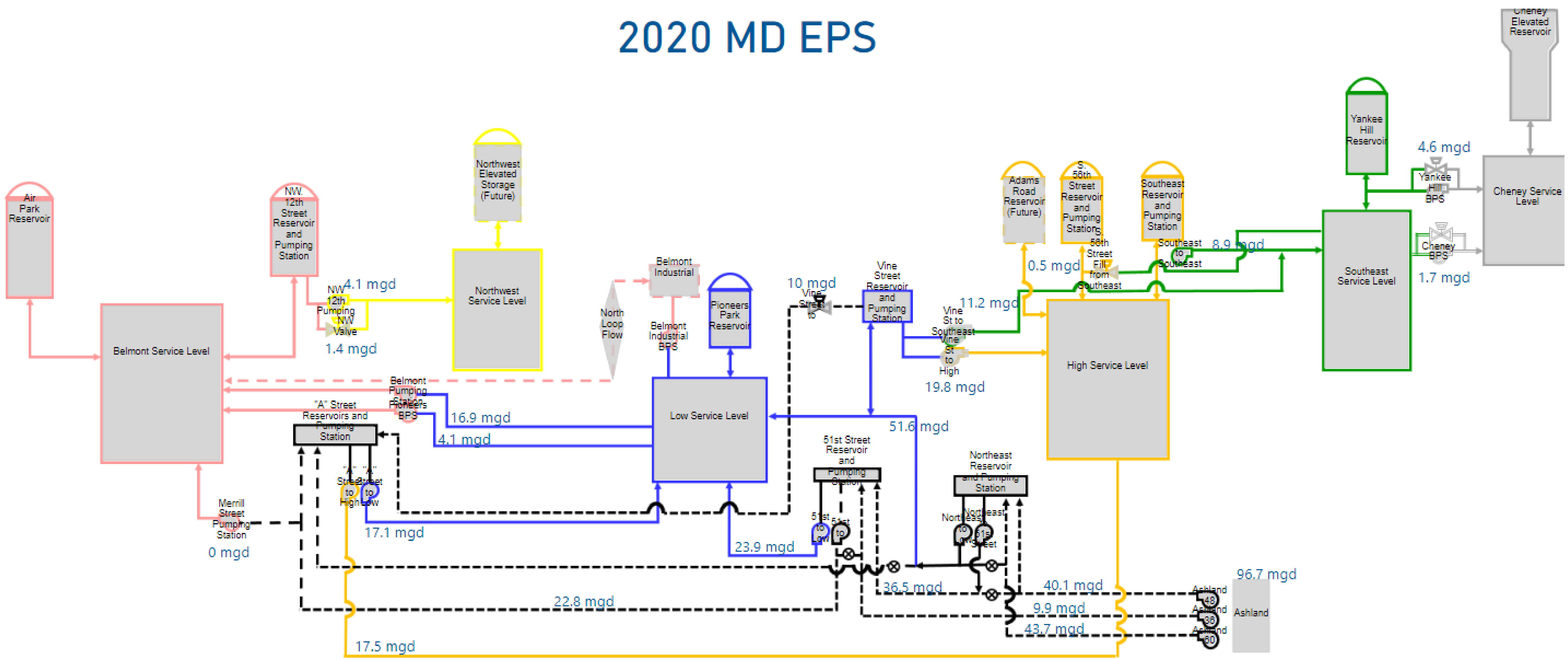


Figure 6-20 **Year 2020 Maximum Day EPS Scenario Schematic**

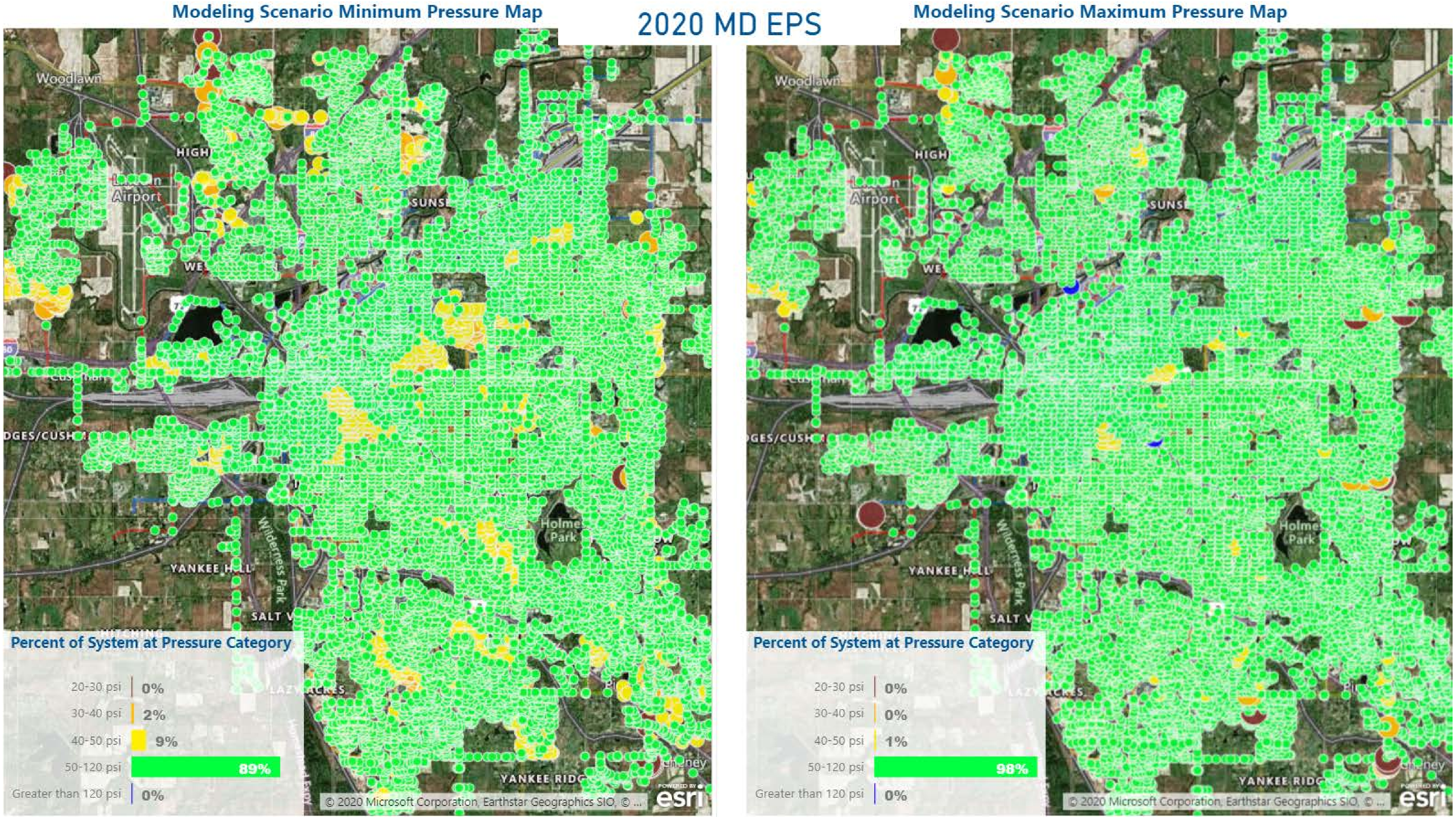


Figure 6-21 Year 2020 EPS Modeling Pressure Results

Year 2020 EPS minimum pressures, generally occurring during maximum hour, and maximum pressures, generally occurring during replenishment times, were also placed in a visual dashboard to review the results by Service Level in more detail. Figures showing these are provided at the end of the chapter. These show the minimum model pressures on the left side of the visual, with a count of how many model junctions fall within each category and the maximum model pressures on the right side of the visual, with a count of how many model junctions fall within each category. Model results only showed two areas which could see pressures greater than 120 psi but not higher than 125 psi. These are in existing areas of the system and do not present a new concern, they have likely been experienced in years past. The notable lower-pressure areas in the distribution system (not directly at facilities or along the transmission network) are listed below:

- Along the Low/Belmont Service Level boundary in the Low Service Level on high ground near North Hill and N 27th to N 31st streets.
- Along the High/Low Service Level boundary in the Low Service Level at "O" Street and 30th to 33rd Street
- Along the High/Low Service Level boundary in the Low Service Level at Washington Street and 21st to 23rd Street.
- Along the High/Low Service Level boundary in the Low Service Level at Vine Street from 42nd Street to 48th Street.
- In high ground areas within the High Service Level north of Prescott at 49th Street.
- Along the High/Southeast Service Level boundary within the High Service Level at London and Chiswick/Queens.
- Along the High/Southeast Service Level boundary within the High Service Level at Laredo Drive and S 30th Street.
- At high ground within the Cheney Service Level at Heritage Lakes and 91st Street.
- At high ground within the Belmont Service Level at NW 12th Street and Research Drive. Pressures are around 35-psi during a maximum hour.
- At high ground within the Belmont Service Level near Thatcher and NW 57th Street. There is an immediate recommended improvement to strengthen the network in the area (IM-9) and extension improvements in the 6-year CIP. AFD addition at Pioneers Pumping Station could also help to keep the area within acceptable pressures.

Pressures are only marginally low in these areas, below 35-psi but above 30 psi, and most of these occur where there is a pressure zone boundary or high ground in existing Service Levels. The areas along boundaries would be good candidates for future monitoring during design years and if it is deemed that low-pressures are resulting in customer complaints, pressure reducing valves could be added at the boundary locations.

As an example of this, a PRV was modeled at the low-pressure area at "O" Street and 33rd Street with a setting that would maintain Low Service Level pressures in this area above 35 psi at the minimum. The PRV only opened during 1 hour of the scenario and transferred approximately 1.5 mgd during that hour. This raised all the marginally low pressures in that area above 35-psi and the average flow transferred from the High Service Level to the Low Service Level over the entire 48-hour scenario was less than 0.1 mgd. These marginally low pressures only occur during the maximum hour of a design year and during a more typical year they would be above 40 psi. The downside of adding PRVs is that they will need to be exercised and maintained but would only operate during the most extreme conditions if an appropriate downstream pressure is set to avoid burning energy unnecessarily.

In summary, the results of the 2020 maximum day EPS scenario support the Vine East Pumping Station East – Pump No. 8 addition within the 6-year CIP and the addition of the Adams Road Reservoir and pipelines in the 12-year CIP. Pipeline improvements in the Belmont Service Level between "O" Street and Partridge are recommended in the 6-year CIP and will provide support to an area which could experience low pressure.

6.7 Year 2032 Distribution System Modeling

Growth from 2020 to 2032 was allocated to the model junctions based on the spatial growth in TAZ population, developed in Chapter 2. Growth was allocated as residential or non-residential to junctions within the growth areas. The 2032 EPS modeling scenario was developed using similar control-based pumping to fall within the low and high operating ranges for storage facilities while restricting pump stations to their firm capacity. Figure 6-22 through Figure 6-25 shows the modeling results of the hourly storage levels for the 2032 maximum day EPS evaluation. Results were captured for each Service Level in a visual dashboard for tank levels, pump flows, and discharge HGL and are provided at the end of this Chapter.

General observations on the ability to maintain storage within the operating ranges are provided after the figures.

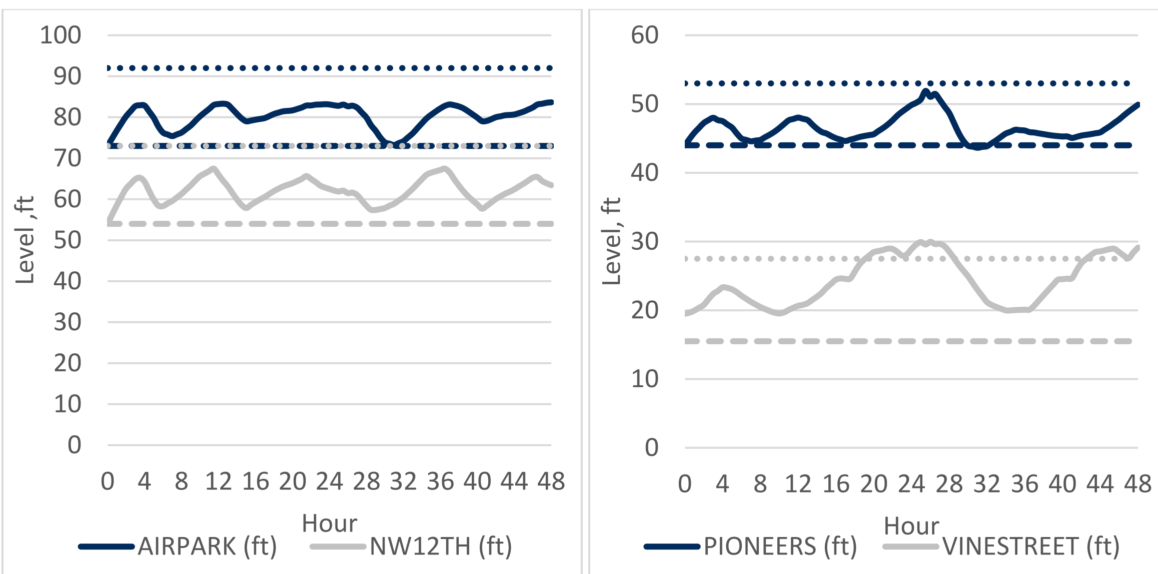


Figure 6-22 Belmont and Low Storage Levels (2032 MD EPS)

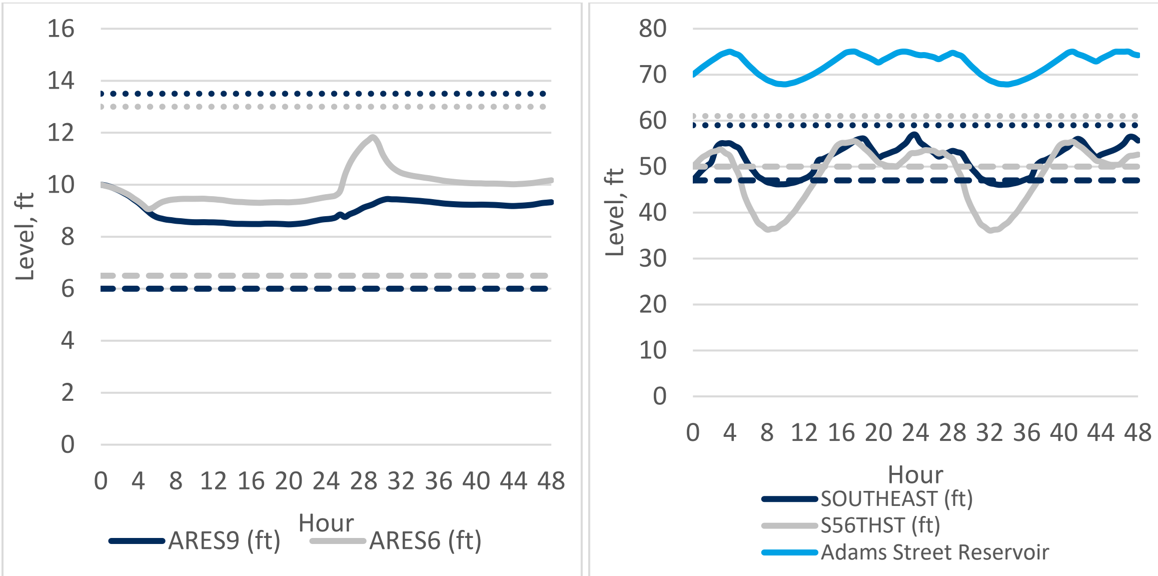


Figure 6-23 "A" Street and High Storage Levels (2032 MD EPS)

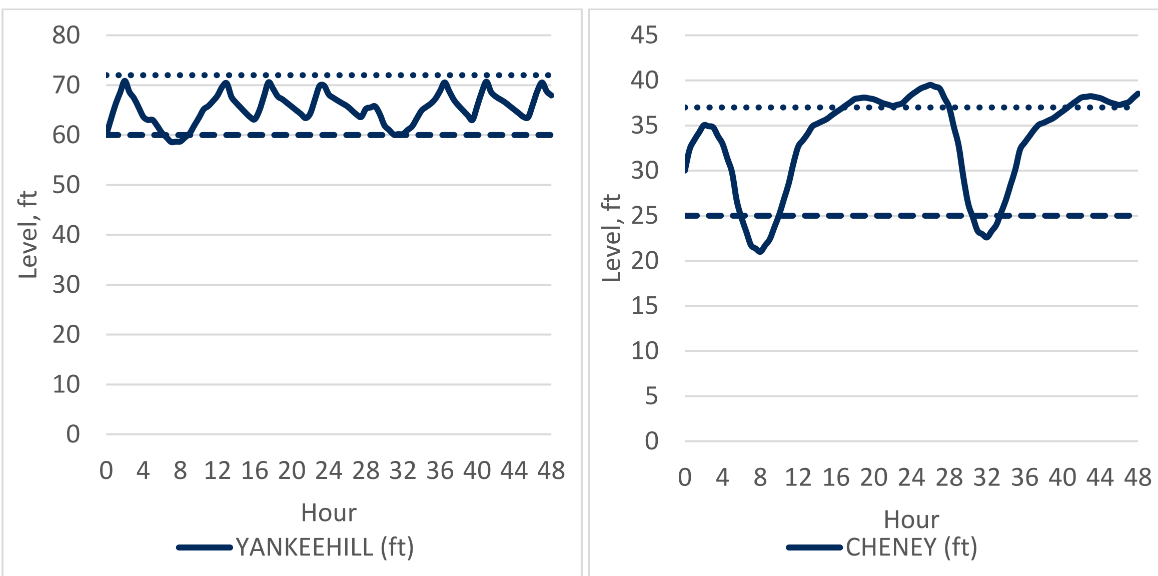
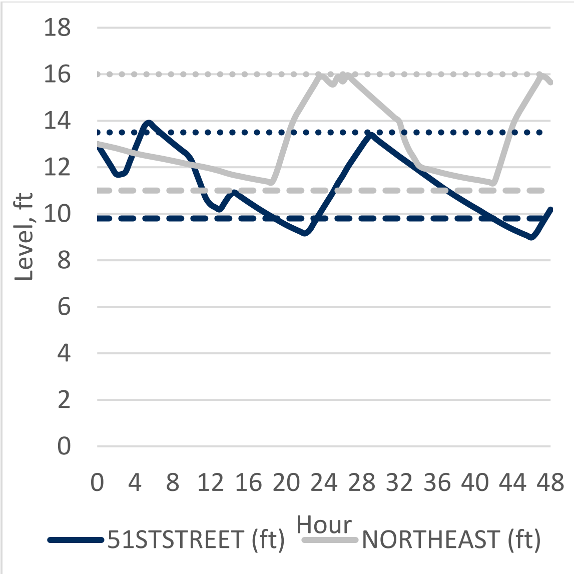


Figure 6-24 Southeast and Cheney Storage Levels (2032 MD EPS)



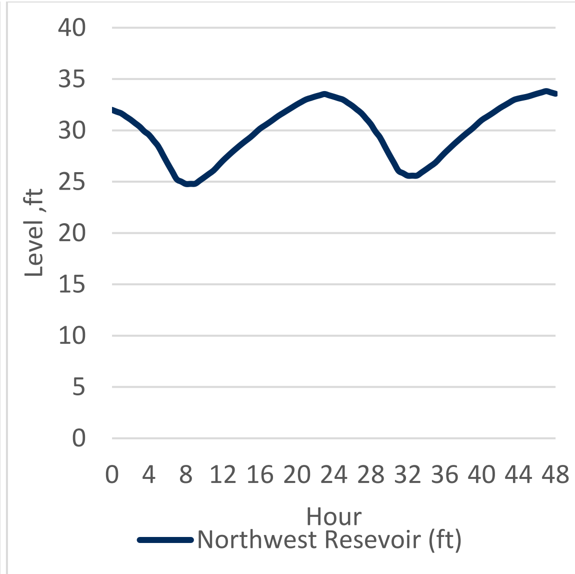


Figure 6-25 **51st Street, Northeast, and the Northwest Storage Levels (2032 MD EPS)**

The addition of pumping capacity at Vine Street Pumping Station East has improved the storage levels in the High Service Level as a by-product of reducing the need to pump some of the supply through the High Service Level to feed the Southeast Service Level. In the Year 2020 scenario, the S. 56th Street Reservoir generally had higher water levels than the Southeast Storage Facility. The reverse occurs in the Year 2032 evaluations, partly due to growth at the system peripheries and partly because less water is needed to be withdrawn from the High Service Level at the Southeast Pumping Station and Reservoir to supply the Southeast Service Level. In order to avoid dropping the water levels in the S. 56th Street Reservoir too low, the control valve between the Southeast Service Level and the High Service Level at the S. 56th Street Reservoir was used to supplement the tank level. The PRV at Yankee Hill and Williamson Drive could also be used in place of the S. 56th Street valve, or a combination of the two would support the pressures in southern High. This has been noted in several previous Master Plans and is not a new consideration. Using this valve may need to occur during design years when peak summer demands are experienced.

The “A” Street transfer was needed to maintain the water levels in the “A” Street Reservoirs. In this 2032 scenario, 8 mgd was transferred from Vine Street to the “A” Street Reservoirs. Capacity exists to transfer more water if necessary, up to 18 mgd, but this was not needed in the Year 2032 modeling scenario.

The Cheney Reservoir drops below the low alarm in the 2032 maximum day EPS scenario. What is interesting about the Cheney Service Level is that even though the reservoir drops lower than in the 2020 scenario, the low pressures that were experienced in 2020 modeling did not occur in 2032. The reason for this is that some of the development extensions provide more conveyance capacity within the Service Level in 2032. This highlights an important concept, which is that any low alarms that are based purely on maintaining levels to mitigate known concerns of low pressures, could be revisited once development extensions are added. This may have an impact in future winter operations by allowing storage to be maintained even lower, thus further reducing water age.

The average results of the Year 2032 maximum day modeling scenario showing the distribution system flow schematic and transfer through pumping stations and major transmission pipelines is provided in Figure 6-26.

An overall figure showing distribution system pressures, both minimum (left side) and maximum (right side), for the Year 2032 maximum day scenario is provided in Figure 6-27.

The same notable lower-pressure areas in the distribution system also occurred in the 2032 EPS scenario with the exception of the high ground area in the Cheney Service Level which has improved to above 40-psi. The pressures for the 2032 modeling evaluations by service level are provided at the end of this chapter.

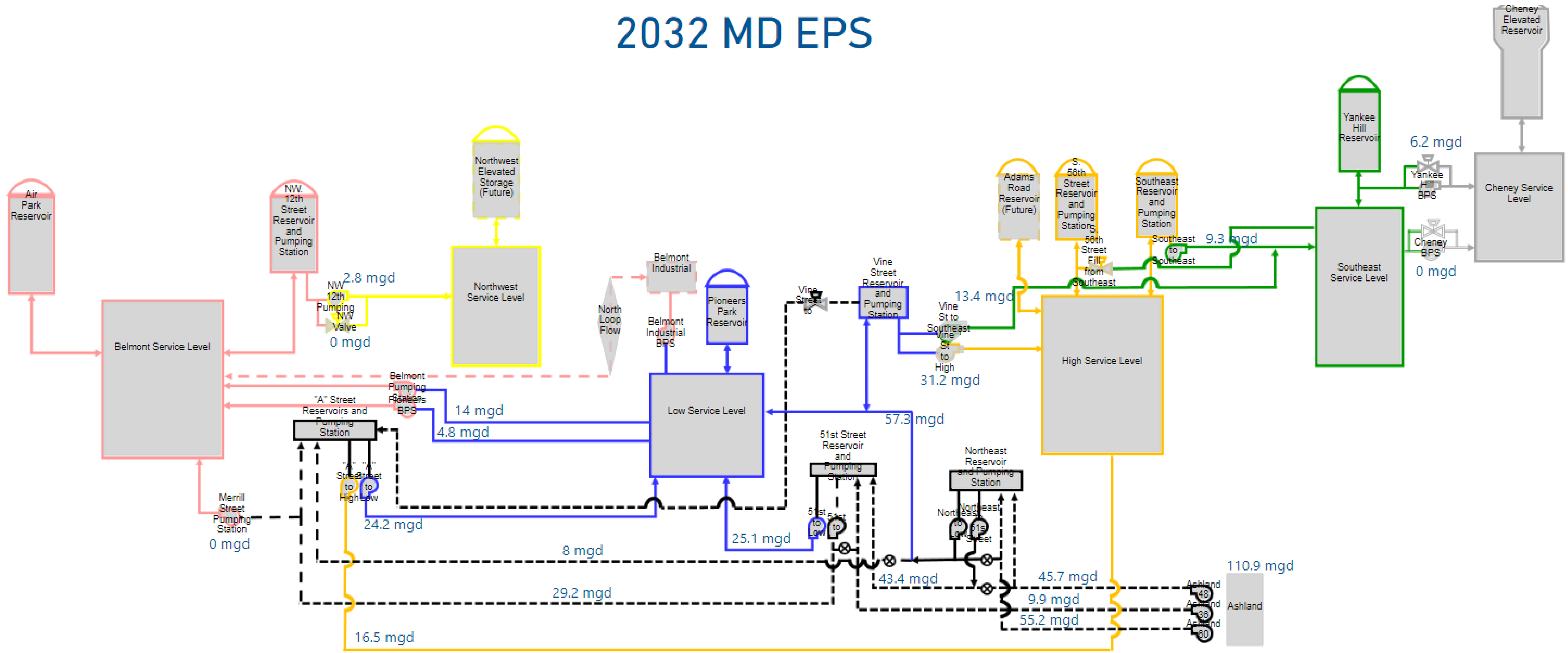


Figure 6-26 **Year 2032 Maximum Day EPS Scenario Schematic**

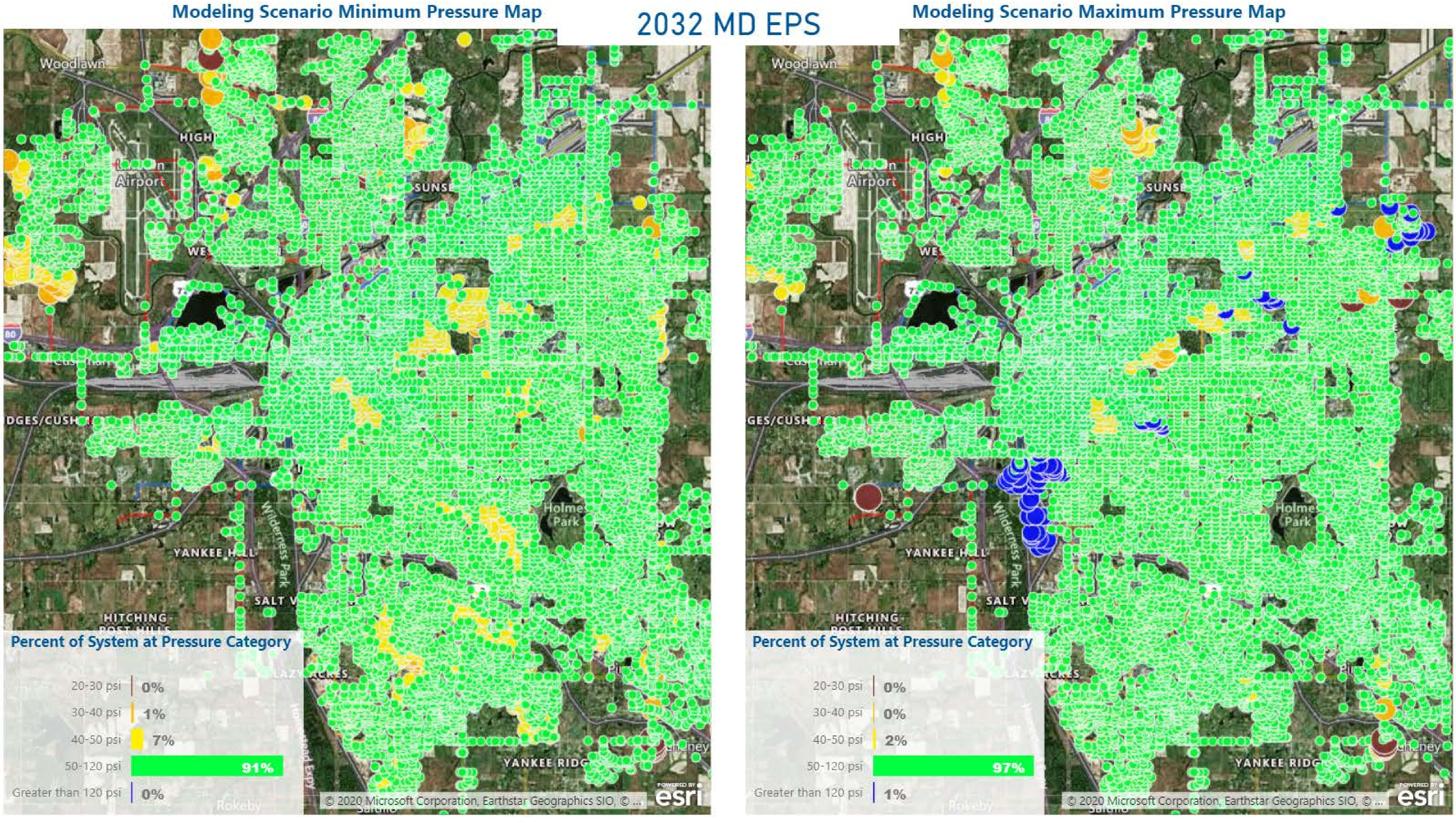


Figure 6-27 Year 2032 EPS Modeling Pressure Results

6.8 Other Modeling and Desktop Evaluations FOR THE 12-Year CIP

Several of the items in the CIP were answered through the 2020 EPS and 2032 EPS base modeling scenarios. Others were individually evaluated to determine their need and usefulness. Several additional scenarios were performed, unique to the improvement being evaluated. This section will discuss each project and any additional evaluations, whether desktop or through modeling, and reference supporting discussion related to each improvement if it has been provided in a previous section.

6.8.1 Valve Replacement and Automation at 51st Street PS

At the 51st Street Pumping Station and Reservoirs, there are some valves which LWS has identified as candidates for replacement. The current valves are manually operated and are at or near the end of their service life. LWS would like to automate the valves at this location to allow for remote operation and the potential bypassing of the 51st Street Pumping Station and Reservoirs.

6.8.2 NW 12th Street Pumping Station

The Northwest 12th Street Pumping Station has adequate capacity to provide the Northwest Service Level through 2032. However, it is noted in the *2014 Master Plan* that this is nearing the end of its useful life as it was intended as a temporary pumping station. A new Pumping Station should be constructed with an existing 5 mgd firm capacity, 8 mgd total. The ultimate capacity should be 8 mgd firm and 12 mgd total, but should be revisited with each Master Plan as growth occurs in the Northwest Service Level.

6.8.3 Vine Street Pumping Station East - Add Pump No. 8 w/ AFD

The addition of this pump was recognized and shown in the previous sections. This pump will increase the firm pumping capacity at Vine Street Pumping Station East to the Southeast Service Level and provide flexibility in operations. With the AFD, flow can be modulated into the Southeast Service Level which can have significant benefits during periods of lower demands in terms of energy and water age. Because there is an empty pump bay in this pumping station, rather than replacing the existing Pump No. 6, the first phase should be to install a new pump with a similar capacity as Pump No. 7.

6.8.4 Innovation Campus - Phase 1 - 16-inch Main

This improvement provides reliability and redundancy to the Innovation Campus once the Merrill Street Pumping Station is decommissioned.

6.8.5 I-80 & 56th Street Pumping Station - Supply Main and PS and Belmont Loop

This improvement has been evaluated and discussed in the focus area section for the North 56th Street and I-80. Much of the supply main has already been constructed up to the I-80 intersection along 56th Street. The Belmont Loop, connecting the area north of I-80 at 56th Street, will connect west then south to the existing area within Belmont at N. 14th Street and Alvo Road. This loop provides reliability and redundancy in case of an outage of the I-80 and N. 56th Street Booster Pumping Station.

6.8.6 16-inch Main on NW 56th Street, "O" St. to Partridge Lane

The need for this improvement and its benefits were discussed in the previous modeling sections.

6.8.7 Decommission Merrill Street Pumping Station

Due to its condition and lack of use, the Merrrill Street Pumping Station is recommended for decommissioning by 2022.

6.8.8 Rehabilitate Eddy Current Drive - Northeast #6

Pump No. 6 at the Northeast Pumping Station has been unusable for almost 20-years due to a faulty eddy current drive. A recent inspection was performed by the manufacturer which determined the drive is still viable but needs control components upgraded. The recommended plan for repair includes installation of a new EC-2000 controller along with a factory rehab and service of the drive and the motor since they have been sitting idle for a significant period of time.

6.8.9 31st and Randolph Valve Vault Relocation to "A" street

There is a 24-inch butterfly valve (No. 797 on the Foreman's Map, Sheet C-4W) located in a vault in the street at the intersection of 31st Street and Randolph Street used to transfer water from Vine Street to "A" Street. This valve is used to throttle gravity flows to "A" Street, which has caused the seat to wear so the valve will not close tight anymore. In addition, the working conditions in the vault are less than desirable with no head room to work. LWS would like to replace this valve with a buried butterfly valve strictly for shut-off purposes and a ball valve and electromagnetic meter installed near "A" Street Reservoirs Nos. 8 and 9 (30th Street and Capital Parkway) for throttling purposes. The vault should be removed from the street.

6.8.10 Add 20.9 mgd WTP South Pumping Station Pump No. 13

This pump was not shown to be needed for maximum day demands by Year 2032. However, it does provide additionally flexibility in operations of the WTP supply into the distribution system.

6.8.11 Add AFDs at Pioneers Pumping Station

The addition of AFDs at the Pioneers Pumping Station was a recommendation in the *2014 Master Plan* but was not evaluated under this master plan update. Prior to implementation of this improvement, we recommend further study and refinement of the concept.

Historically, in the Belmont and Southeast Service Levels, pressure variations are significant when pumps start up without AFD's. Some local industries have reported issues with their fire protection systems due to these pressure variations as pumps turn on and off. Additionally, Belmont and Southeast Pumping Stations discharge into large transmission mains, a 30-inch main in North 14th Street from the Belmont Pump Station and a 48-inch main in South 84th Street from the Southeast Pump Station, and there are cavitation issues. The pumps in these two stations are operating off their pump curves because of the reduced downstream head conditions. Therefore, operations at these two stations are limited to use of only the large pumps to control cavitation. The current operating procedures work around the cavitation issues but do not provide a long-term solution to be able to run the smaller pumps in the stations.

To start, the Pioneers Pumping Station is recommended for addition of AFD's. Although more expensive initially, AFDs are recommended instead of eddy current drives or discharge control valves due to their comparative inefficiencies. The AFDs would match pump curves to the existing and future system head curves. AFDs should be installed on all of the pumps in the pumping stations to maximum flexibility of operations and enable the smaller pumps to be used during lower flow conditions. At a minimum, AFDs should be added to Pump Nos. 1 and 2 at Pioneers Pumping Station as those are the only ones used at this time.

If the VSD installation is successful at Pioneers Pumping Station, VSD addition to Belmont and Southeast Pumping Stations should be evaluated and installed on the smaller pumps, if deemed cost- effective, so that they can be used again during lower flow conditions without cavitation.

6.8.12 Pressure Monitoring Stations

Four additional pressure monitoring stations are recommended within the 12-year CIP. Three of the locations identified in the *2014 Master Plan* as 2025 improvements are still ideal locations for additional pressure monitoring:

- Near Bridle Lane and S 58th Street in the Southeast Service Level where marginally low pressures occur
- Near Holdrege Street and N 57th Street along the Low/High Service Level boundary, in the High Service Level. Marginally high pressures occur at this location
- At the Low/High Service Level boundary in the Low Service Level near "O" Street and 33rd Street where marginally low pressures occur. This area was detailed in the 2020 modeling evaluations where a new PRV was tested.

The fourth additional pressure monitoring station recommended in the *2014 Master Plan* was at the high ground along Arbor Road. With the detailed evaluation of the I-80 and N 56th Street Pumping Station, it was recommended to locate the pumping station south of Arbor Road and convert this area to Belmont. If this improvement concept is followed, there will be no need for pressure monitoring at this location. An alternate location for this station would be at the high ground area near Laredo Drive and S 30th Street near the High/Southwest Service Level boundary.

6.8.13 Decommission South 56th Street PS

The South 56th Street Pumping Station is currently not operated and is impractical to operate as originally designed and built. Therefore, this pumping station is not used but is designed to boost pressures temporarily to the High Service Level from the South 56th Reservoir. When pumps are operated to utilize more of the reservoir volume, it has proven difficult to refill the tank and results in on-going low-pressure areas in the High Service Level. The Year 2032 maximum day EPS evaluations confirmed that rather than pump out of the S. 56th Street Reservoir during peak demand, it is necessary to transfer some supply from the Southeast Service Level and the Pumping Station is not expected to be run in the future. A capital improvement project has been included in the immediate improvements phase to remove the pumps and VSDs from the pumping station and to salvage them somewhere else in the system, if possible.

6.8.14 Northwest Reservoir (2 MG) and Pipeline

The need for storage within the Northwest Service Level for equalization was supported and discussed in the storage evaluation and the Year 2032 modeling evaluations. The previous recommended location of the new Northwest Reservoir was ¾ of a mile north of the existing NW 12th Street Reservoir because it is at high ground. However, putting a new storage facility this far away from the usage locations in the system and constructing almost a mile of 24-inch main would significantly increase water age and have detrimental impact in an area with existing high-water age. The additional cost of constructing the storage nearer to the existing NW 12th Street is likely a balanced alternative for the potential water quality impact that placing it much further away would create. Higher costs of constructing the storage facility at lower ground is also offset by the reduced pipeline costs with a much shorter distance needed. This facility should be as near to the usage customers within the Northwest Service Level as feasible, to mitigate excessive and unnecessary aging of water as it travels to and from the storage facility.

6.8.15 Belmont to Low PRV Station ("O" Street and NW 25th Street)

This improvement was recommended for fire flow considerations in the *2014 Master Plan*. The location of the valve (noted at "O" street and NW 12th Street) should be at the Belmont/Low Service Level boundary which is at "O" Street and about NW 25th Street. This may have just been a typo in the *2014 Master Plan* as it was previously modeled at the proper location. Fire flow evaluations were not performed in this update, but this area does occur at the periphery of the Low Service Level in an area where redundant flow paths are few. Though fire flow was not modeled in this update, from a visual review of the system this improvement for fire flow capacity is reasonable.

6.8.16 Decommission NW 12th Street Pumping Station

Decommissioning of the NW 12th Street Pumping Station, discussed in a previous section, should occur after the new Pumping Station is constructed.

6.8.17 Decommission Cheney Pumping Station

Decommissioning of the Cheney Pumping Station, currently in the 12-year CIP in Year 2027, can occur anytime subsequent to the addition of a pump at the Yankee Hill Pumping Station. If condition allows, this pumping station can continue to be used until the end of its useful life so a Year 2027 date could be deferred if efficient operations of the Station are still being recognized at that time.

6.8.18 Yankee Hill Pumping Station - Add 6 mgd Pump

Discussion of the addition of a 6 mgd pump at the Yankee Hill Pumping Station was provided in the pumping capacity desktop evaluations.

6.8.19 PRV Southeast SL to High SL - Vault near Southeast PS

This PRV station, discussed in the desktop storage section, will allow the transfer of water from the Southeast Service Level directly into the Southeast Storage Reservoir. This allows for the direct transfer of water from Vine Street Pumping Station East into the High Service Level at the Southeast Reservoir.

6.8.20 Innovation Campus - Phase 2 - 12-inch Main

This Phase 2 main is a second reliable feed to the Innovation Campus and will provide redundant fire protection as well as reliability in every day operations to supply the Innovation Campus.

6.8.21 Adams Street Reservoir and Pipelines for HSL (5 MG)

The Adams Street Reservoir and pipelines were discussed in the desktop storage evaluations and confirmed through the Year 2032 EPS modeling.

6.8.22 54-inch Main from Northeast PS to 88th and Holdrege

The existing transmission system has sufficient capacity to meet maximum day Year 2032 demands. However, the completion of this main will provide additional flexibility during all demand conditions, but especially during the bypass of Northeast for Winter Operations. To quantify the benefit that this main will have during maximum day conditions, the same control set was modeled for scenarios both with and without this improvement to make an apples-to-apples comparison of the benefit. Figure 6-28 provides this comparison. The difference between the operating water levels in the two scenarios shows a daily difference of about 8 feet in the levels of Vine Street. This roughly relates to a volume of 5 MG over the course of a day, or 5 mgd as a rate. This essentially means that for the same energy cost for the Northeast Pumping Station, with the 54-main completed, LWS can transfer an additional 5 mgd. During an average day or Winter Operations when Northeast Reservoir is being bypassed, the energy cost savings could be even greater. This improvement should be constructed towards the end of the 12-year CIP to provide operational flexibility and energy management benefits.

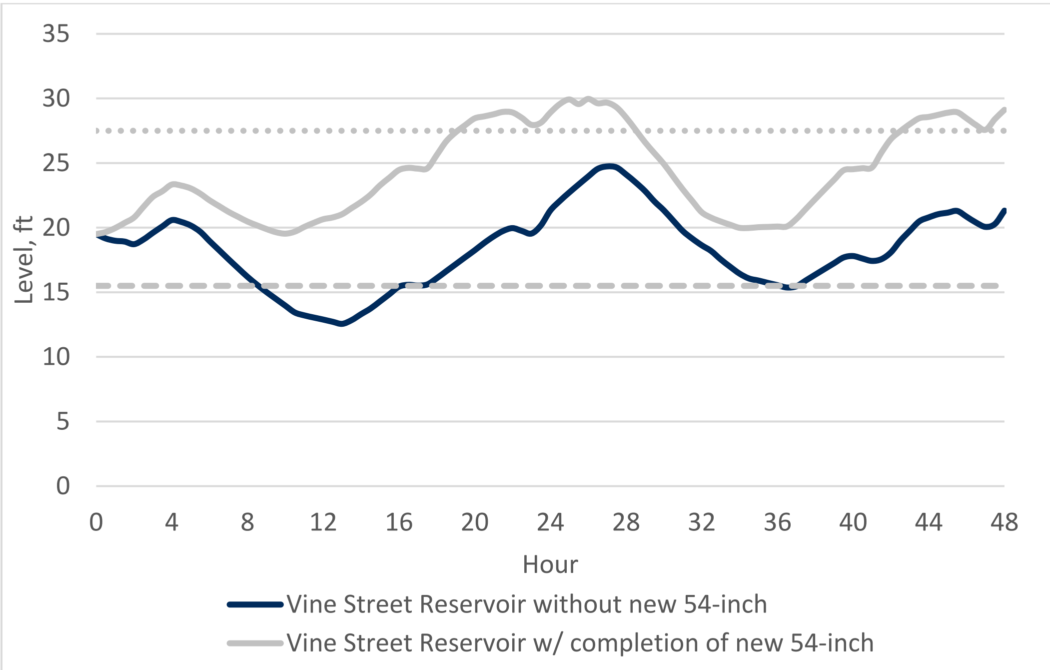


Figure 6-28 **54-inch Main from Northeast PS to 88th and Holdrege Modeling Evaluation (Control Set Held Constant)**

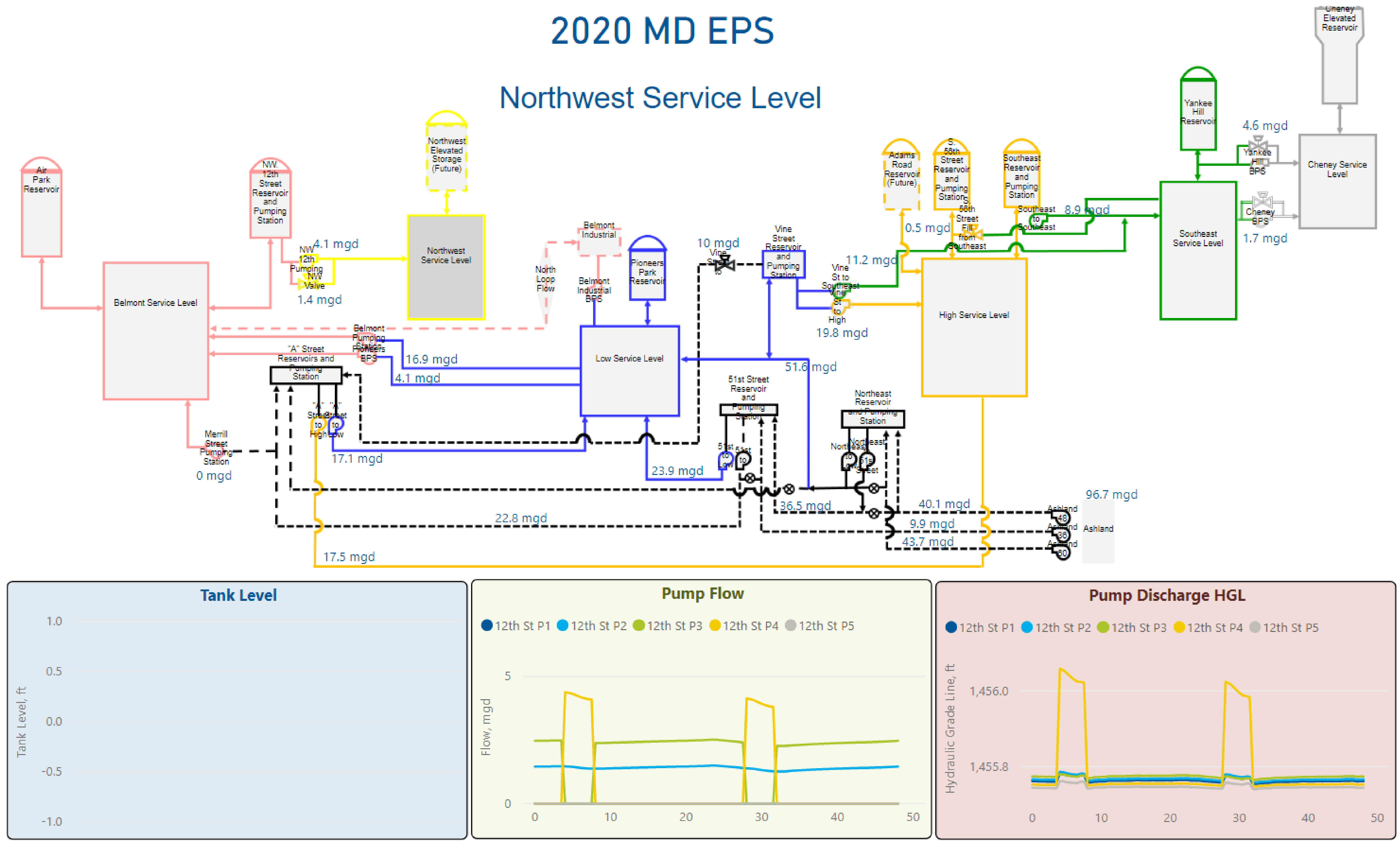


Figure 6-29 2020 MD EPS Model Results – Northwest Service Level

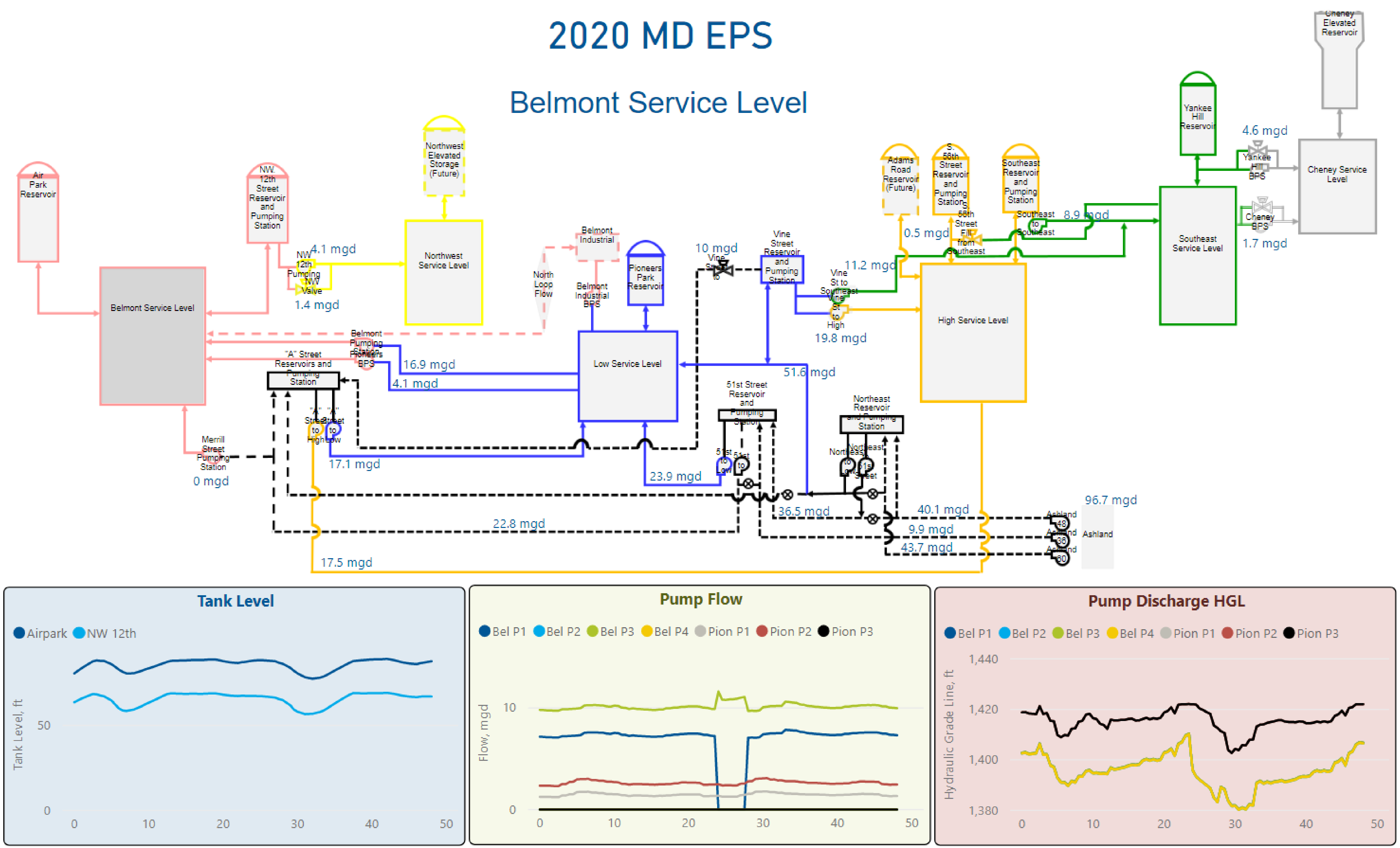


Figure 6-30 2020 MD EPS Model Results – Belmont Service Level

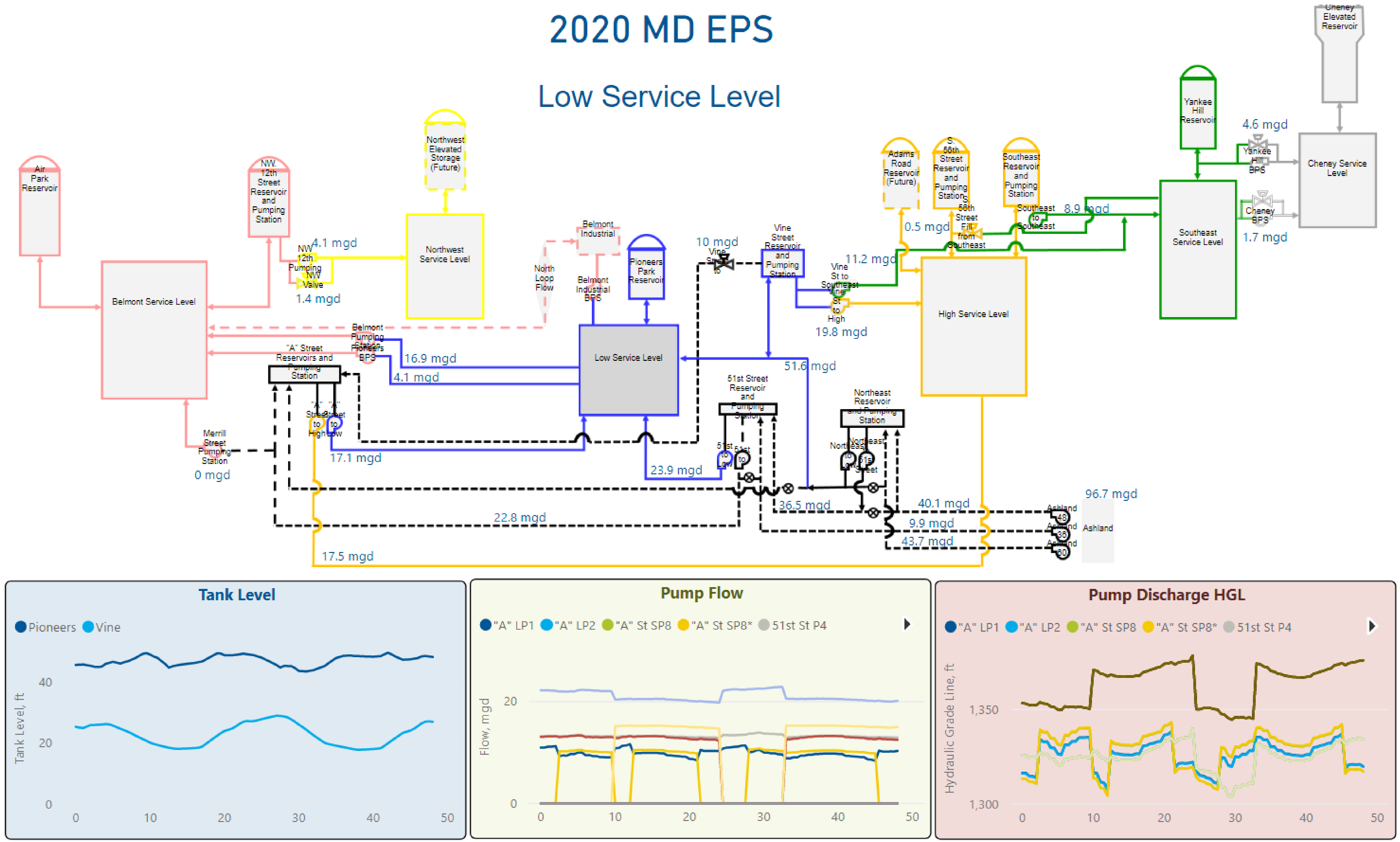


Figure 6-31 **2020 MD EPS Model Results – Low Service Level**

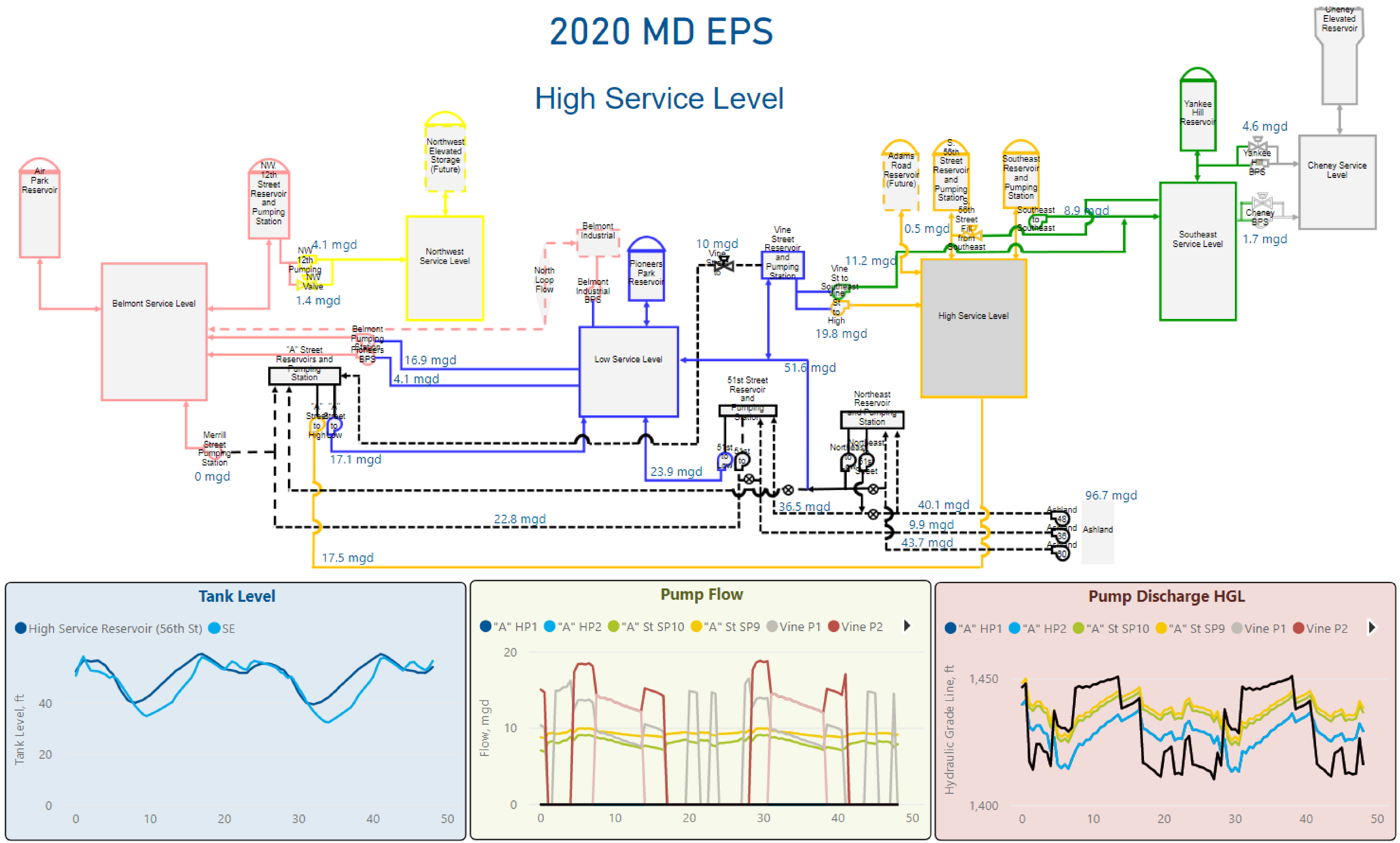


Figure 6-32 2020 MD EPS Model Results – High Service Level

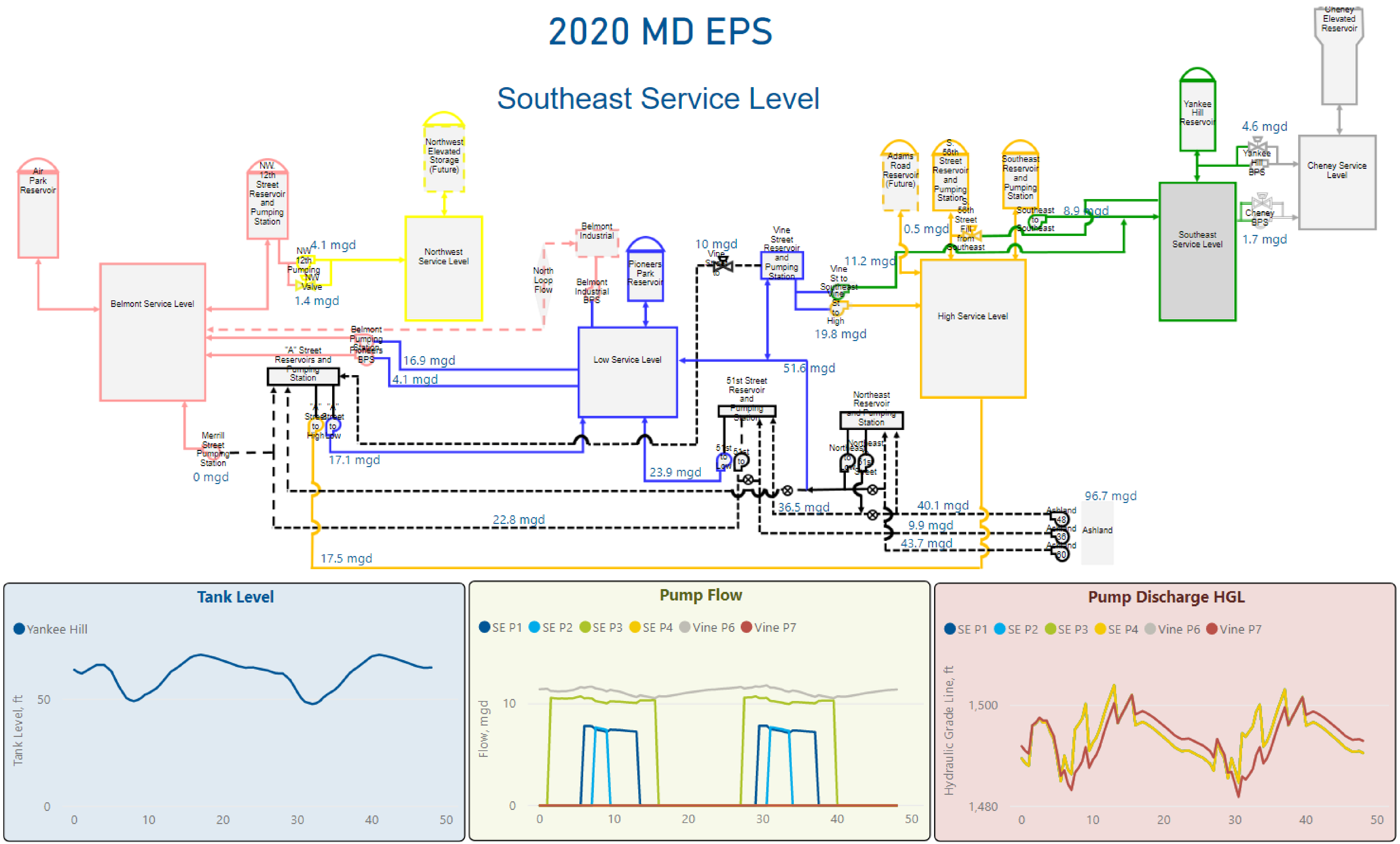


Figure 6-33 2020 MD EPS Model Results – Southeast Service Level

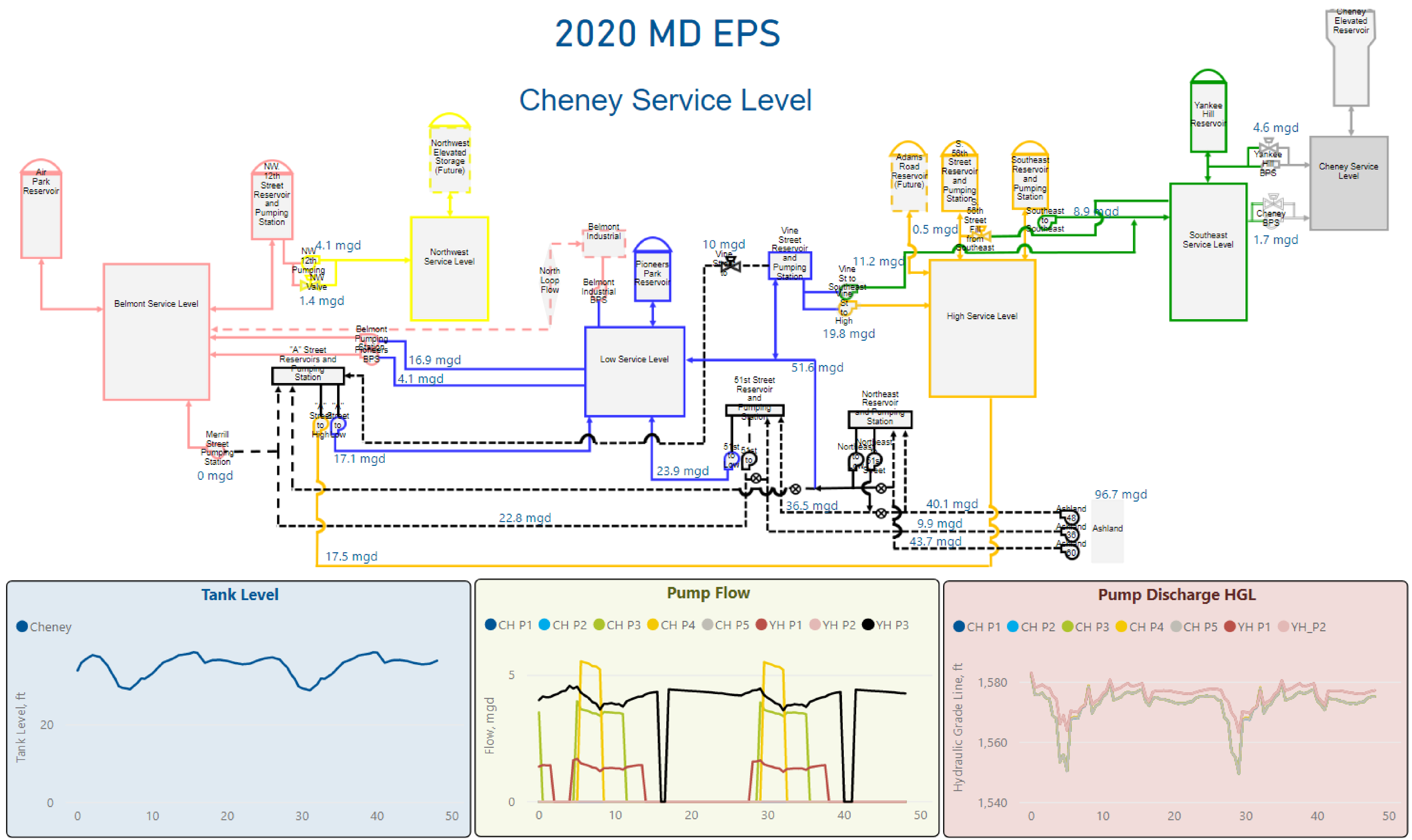


Figure 6-34 **2020 MD EPS Model Results - Cheney Service Level**

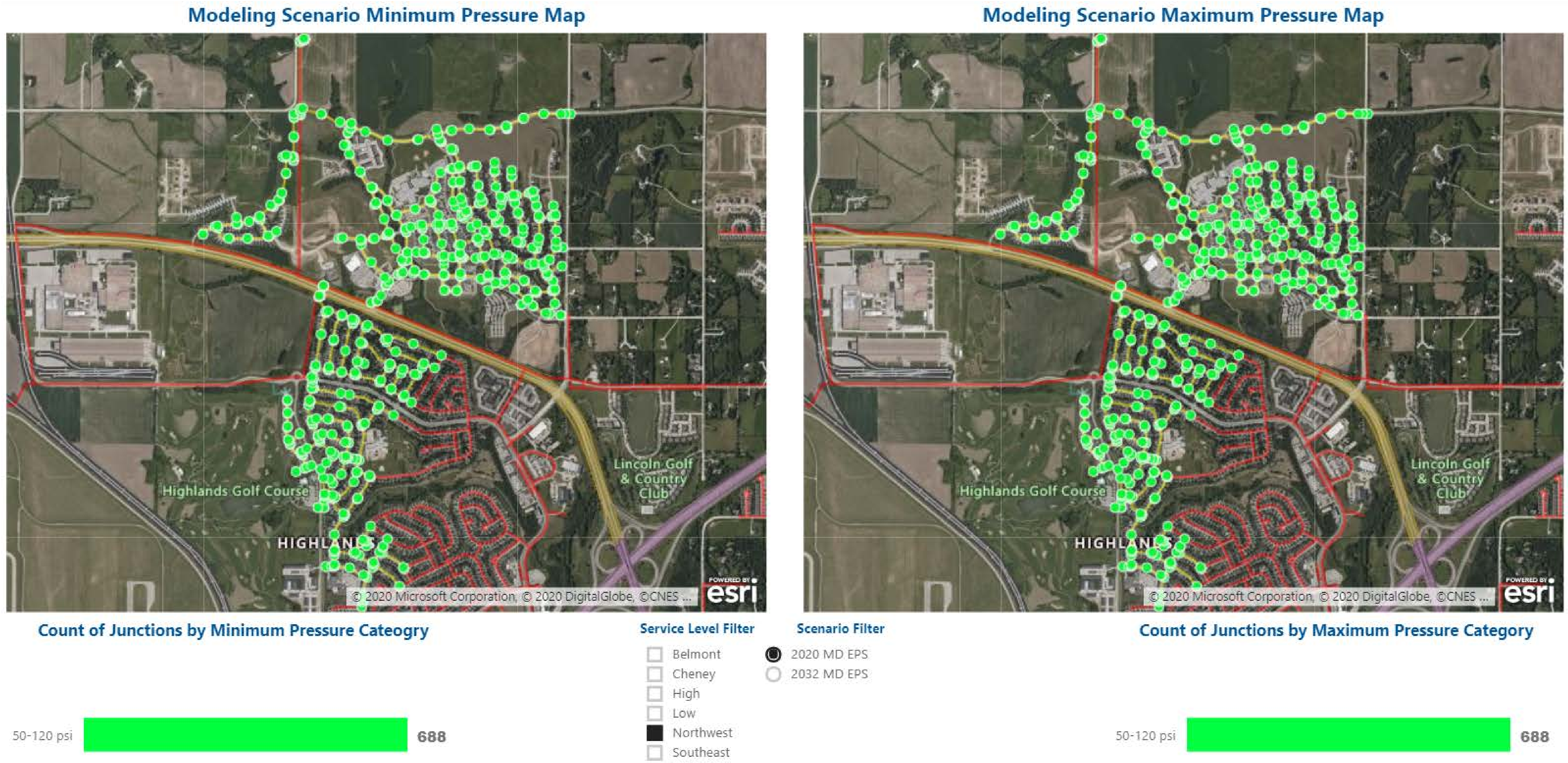


Figure 6-35 2020 MD EPS Model Pressures – Northwest Service Level

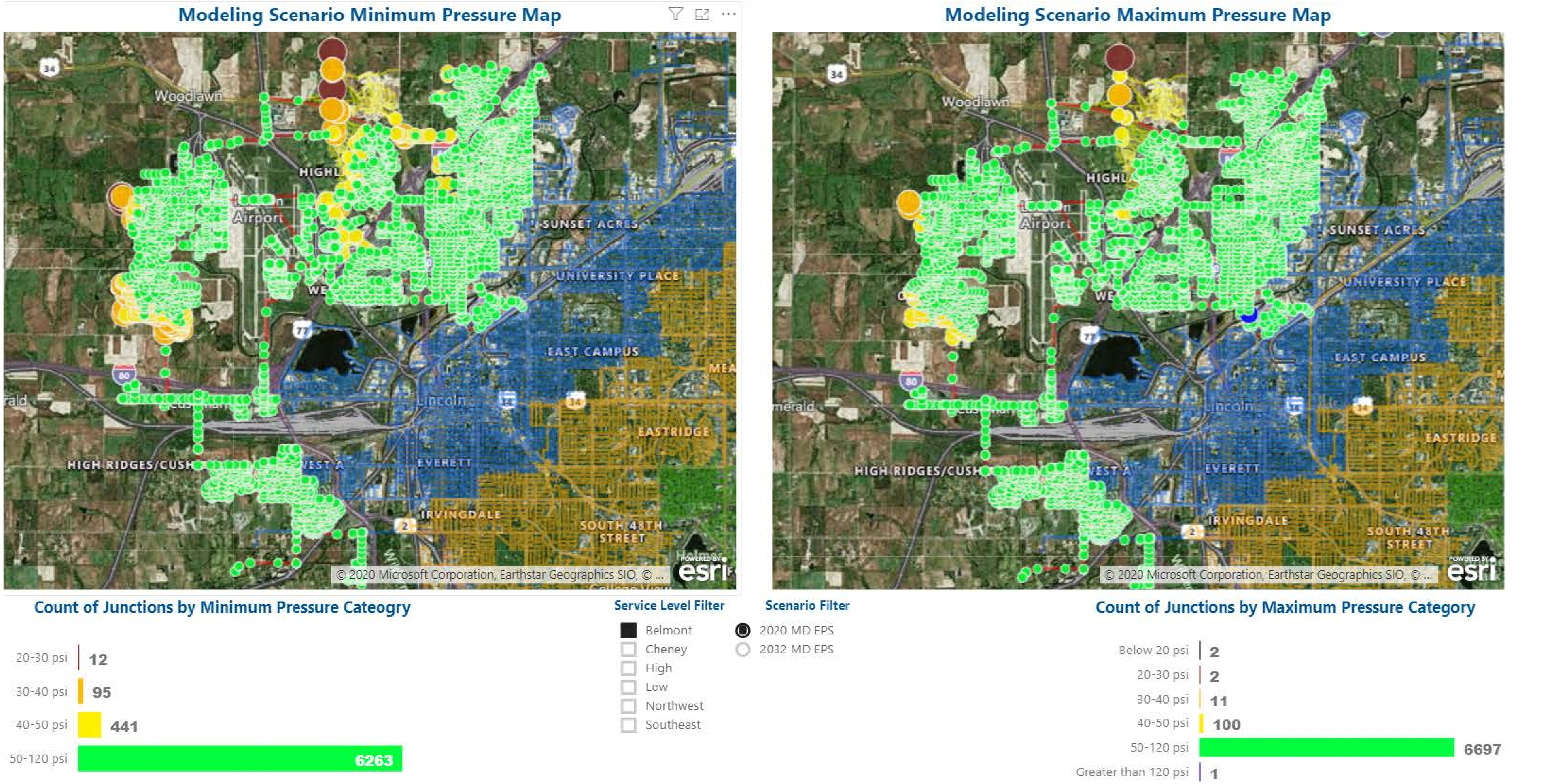
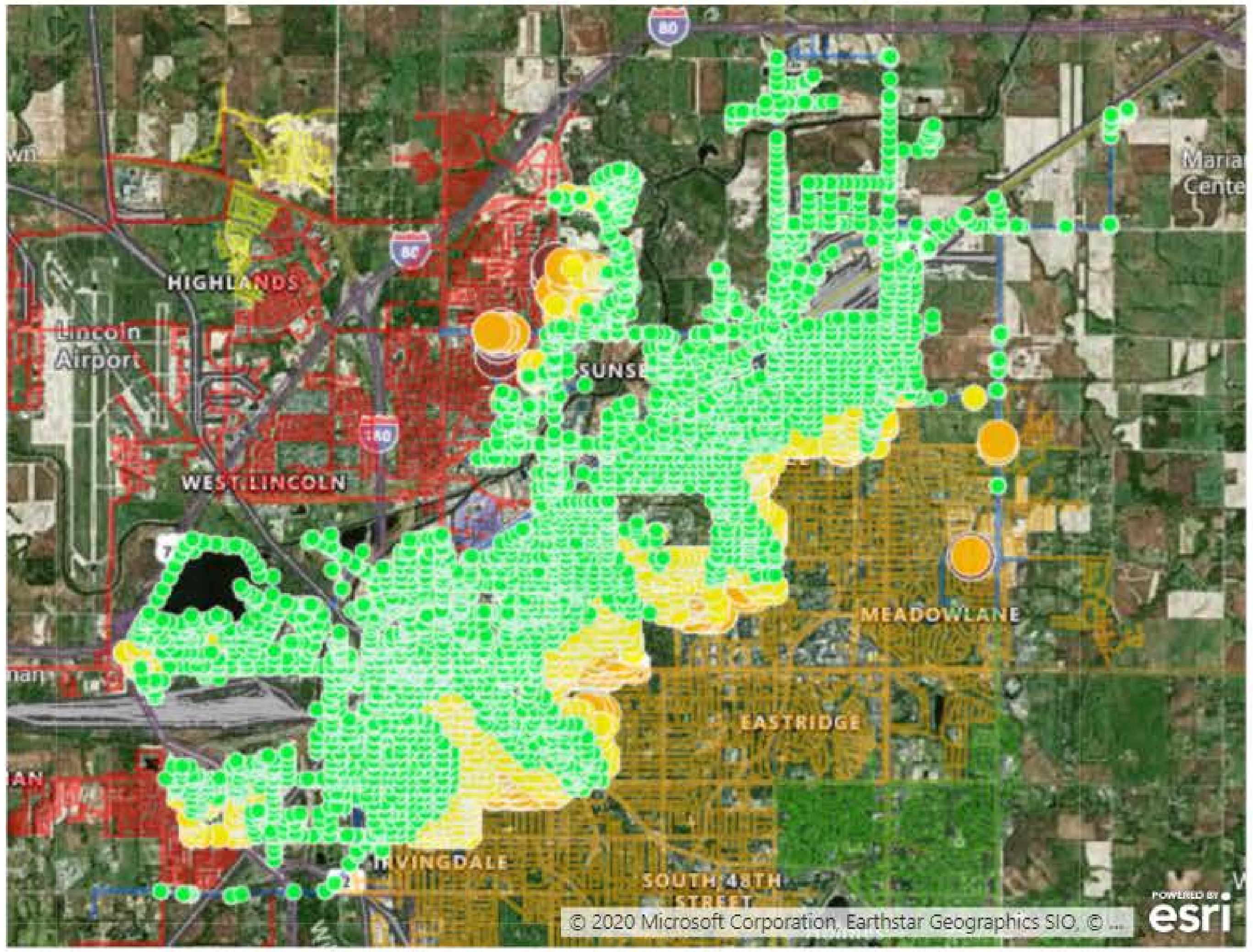
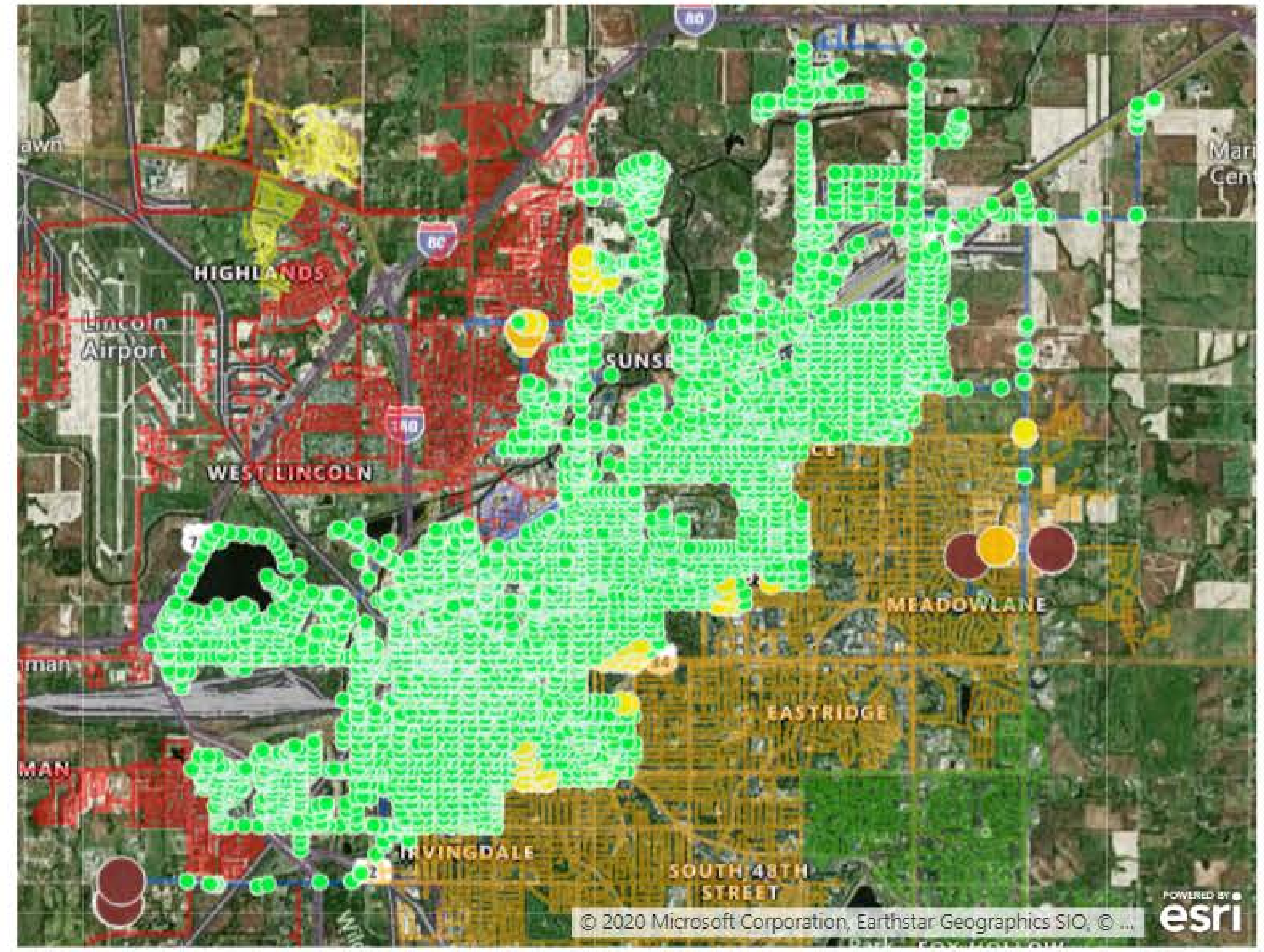


Figure 6-36 2020 MD EPS Model Pressures – Belmont Service Level

Modeling Scenario Minimum Pressure Map



Modeling Scenario Maximum Pressure Map



Count of Junctions by Minimum Pressure Cateogry

Service Level Filter

- ☐ Belmont
- ☐ Cheney
- ☐ High
- ■ Low
- ☐ Northwest
- ☐ Southeast

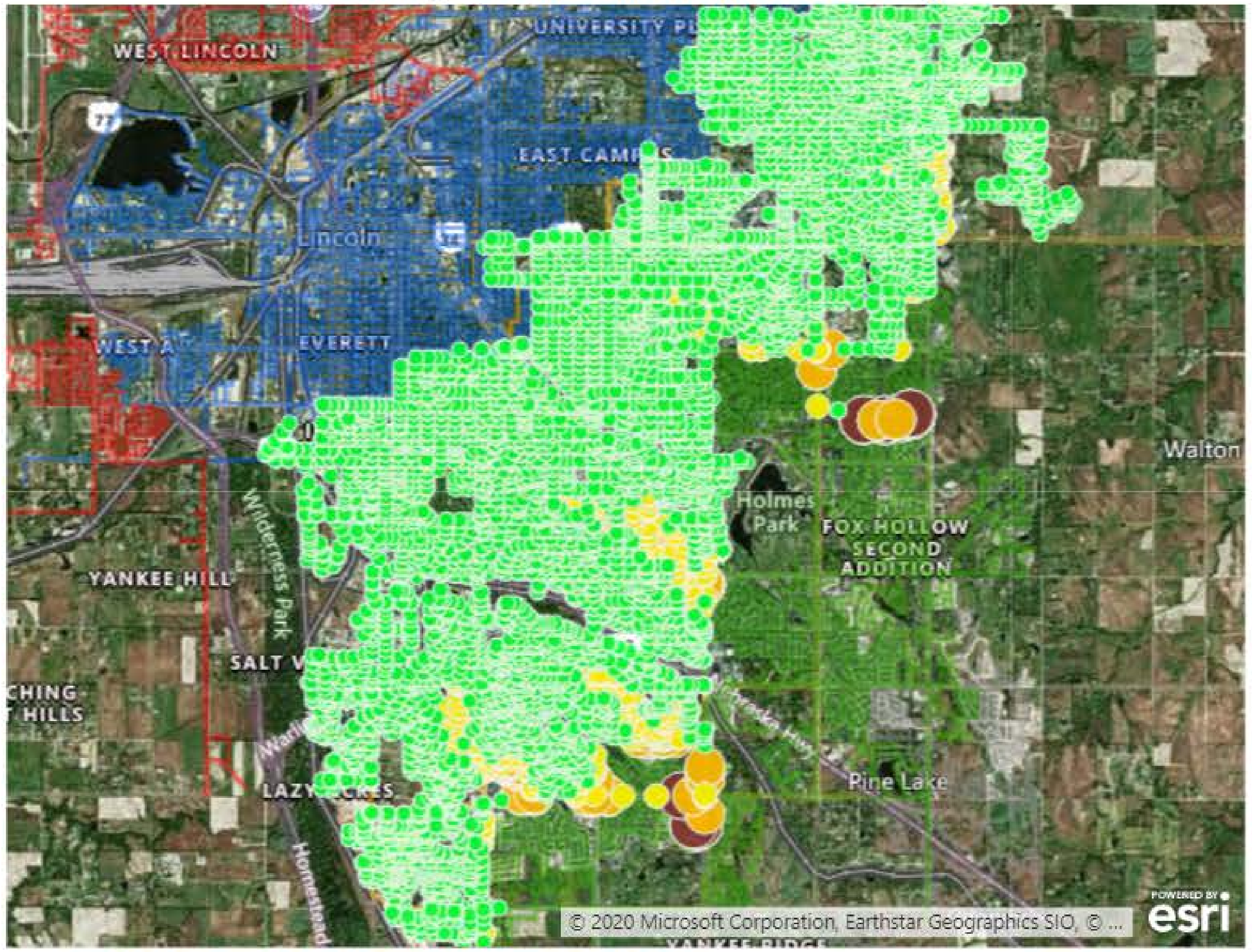
Scenario Filter

- ◉ 2020 MD EPS
- ○ 2032 MD EPS

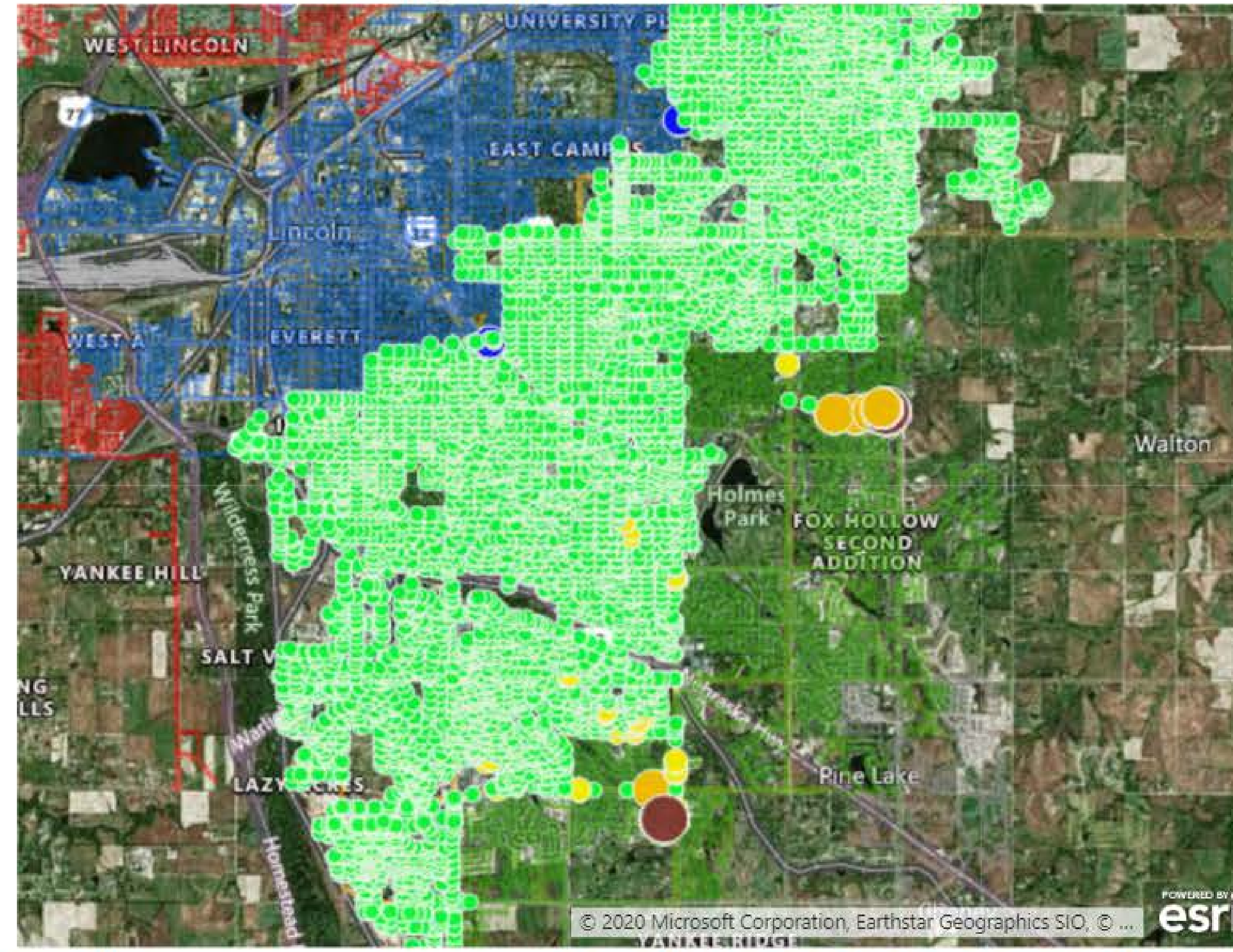
Count of Junctions by Maximum Pressure Category

Figure 6-37 2020 MD EPS Model Pressures – Low Service Level

Modeling Scenario Minimum Pressure Map



Modeling Scenario Maximum Pressure Map



Count of Junctions by Minimum Pressure Cateogry

Service Level Filter

- Belmont
- Cheney
- High (selected)
- Low
- Northwest
- Southeast

Scenario Filter

- 2020 MD EPS (selected)
- 2032 MD EPS

Count of Junctions by Maximum Pressure Category

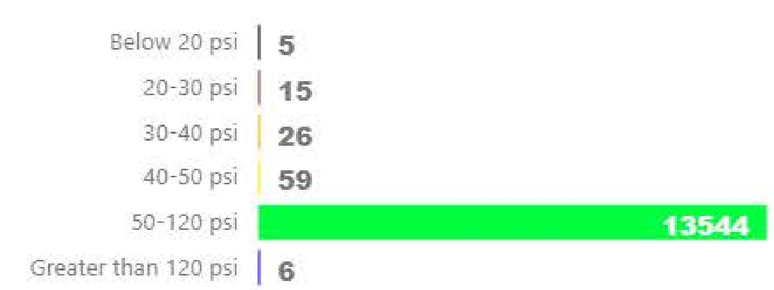
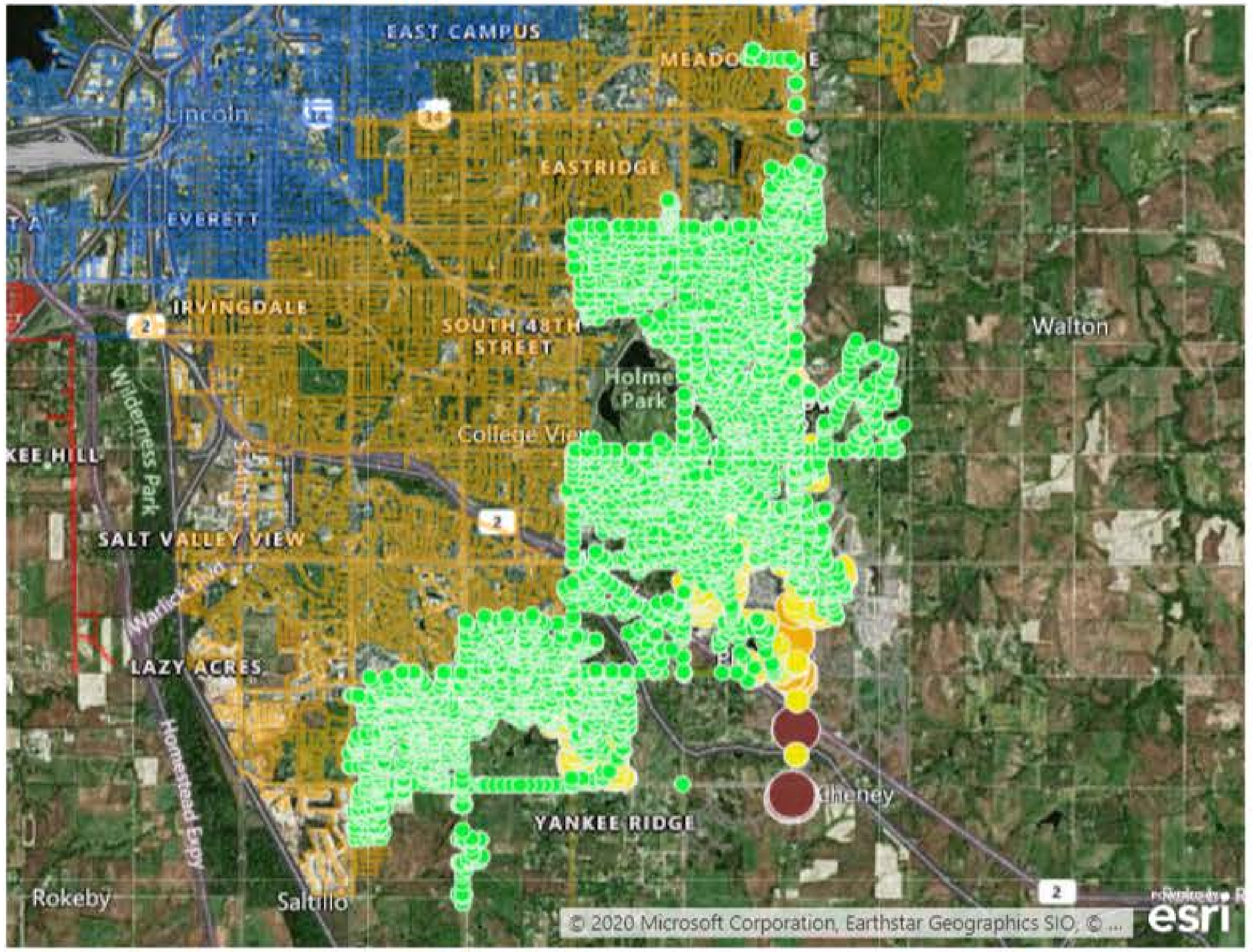
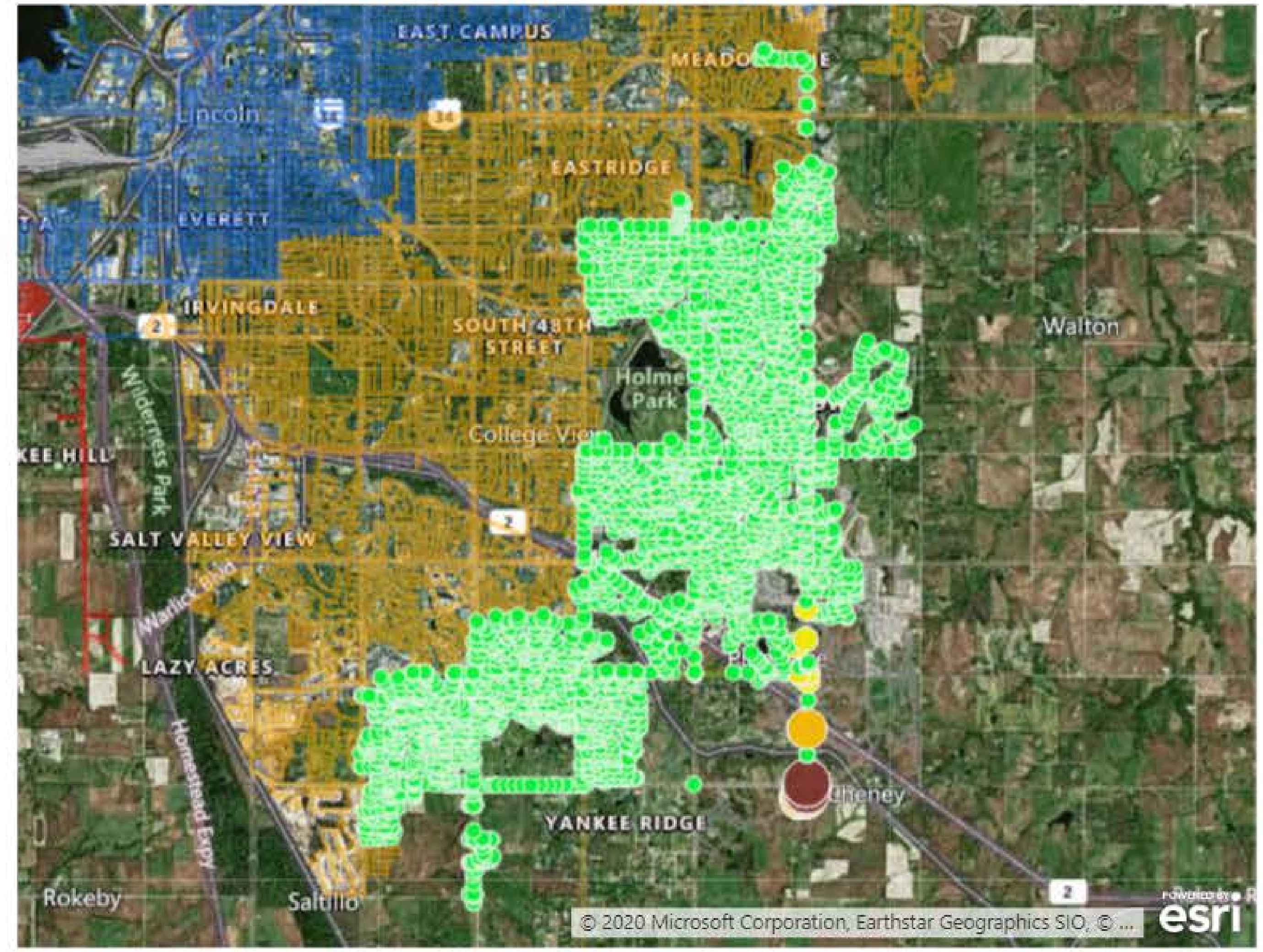


Figure 6-38 2020 MD EPS Model Pressures – High Service Level

Modeling Scenario Minimum Pressure Map



Modeling Scenario Maximum Pressure Map



Count of Junctions by Minimum Pressure Cateogry

Service Level Filter

- Belmont
- Cheney
- High
- Low
- Northwest
- Southeast

Scenario Filter

- 2020 MD EPS
- 2032 MD EPS

Count of Junctions by Maximum Pressure Category

Figure 6-39 2020 MD EPS Model Pressures – Southeast Service Level

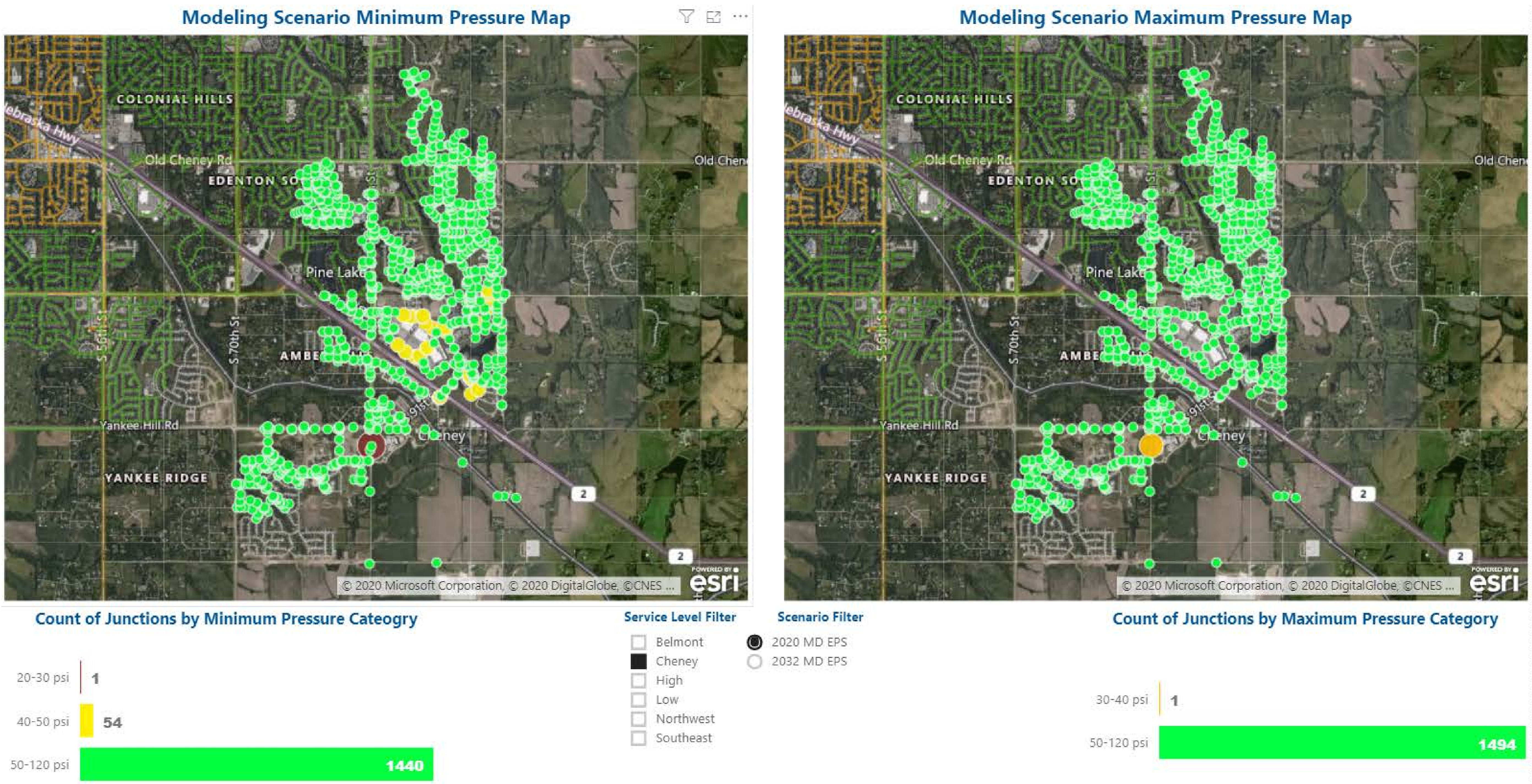


Figure 6-40 2020 MD EPS Model Pressures – Cheney Service Level

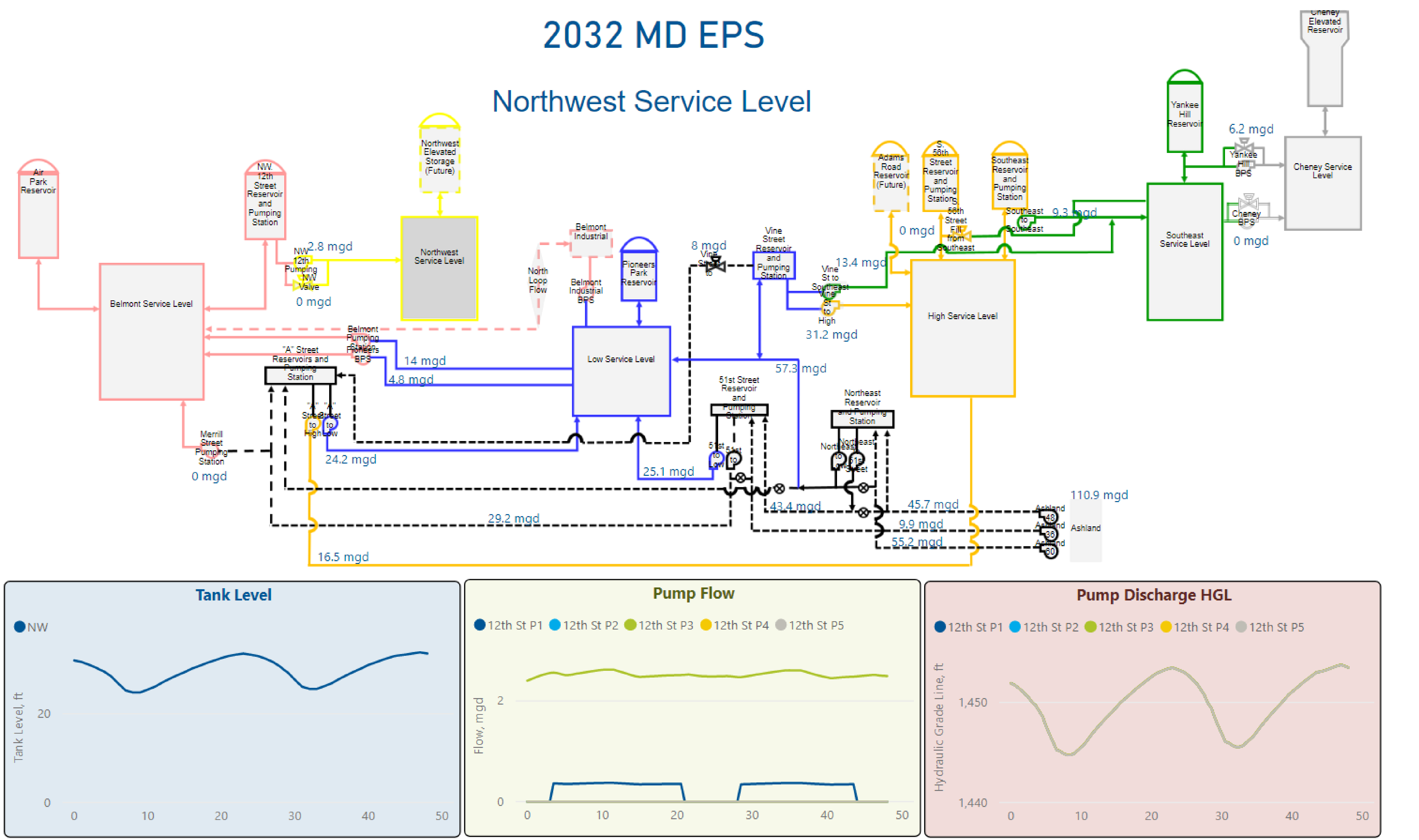


Figure 6-41 **2032 MD EPS Model Results – Northwest Service Level**

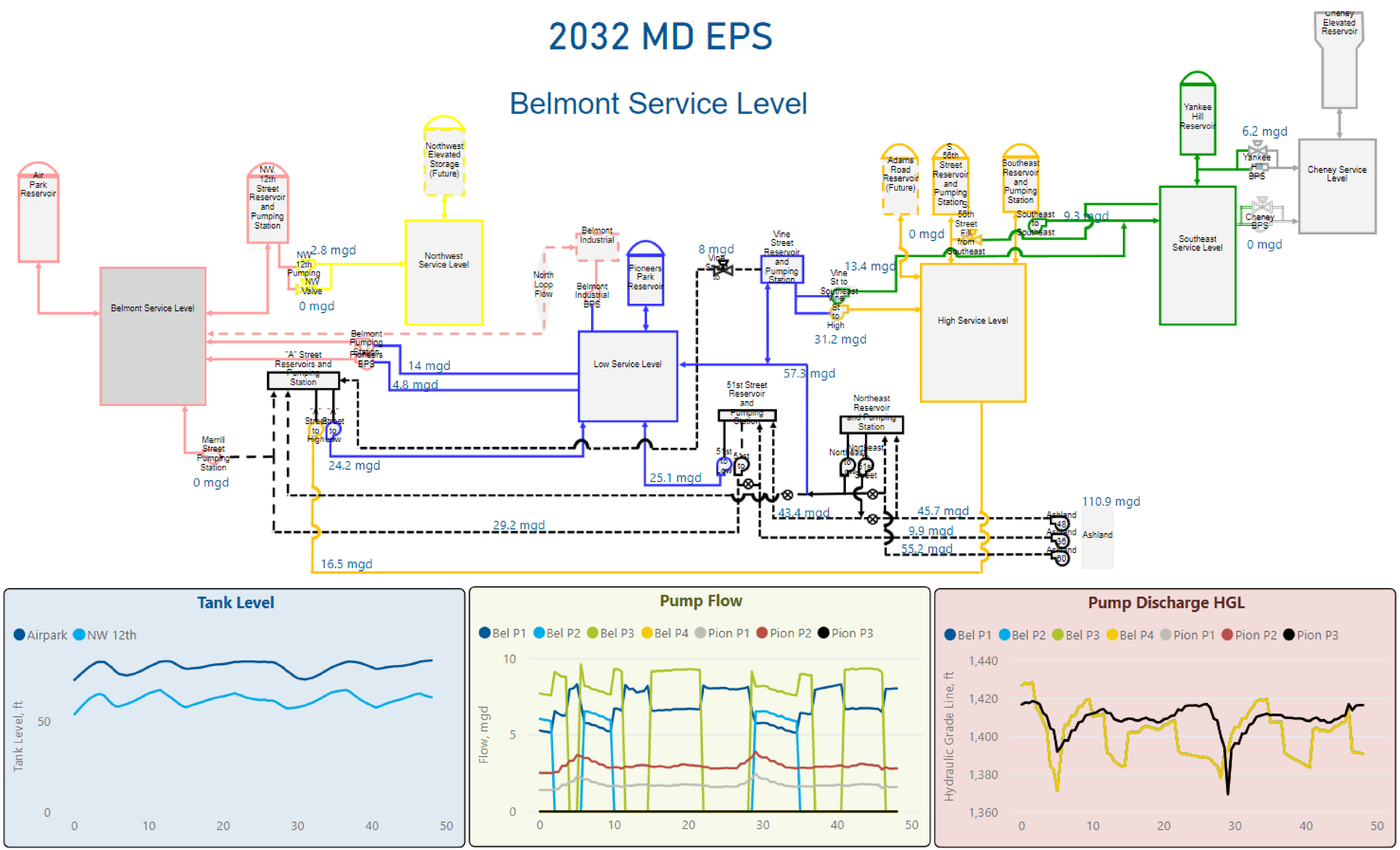


Figure 6-42 **2032 MD EPS Model Results – Belmont Service Level**

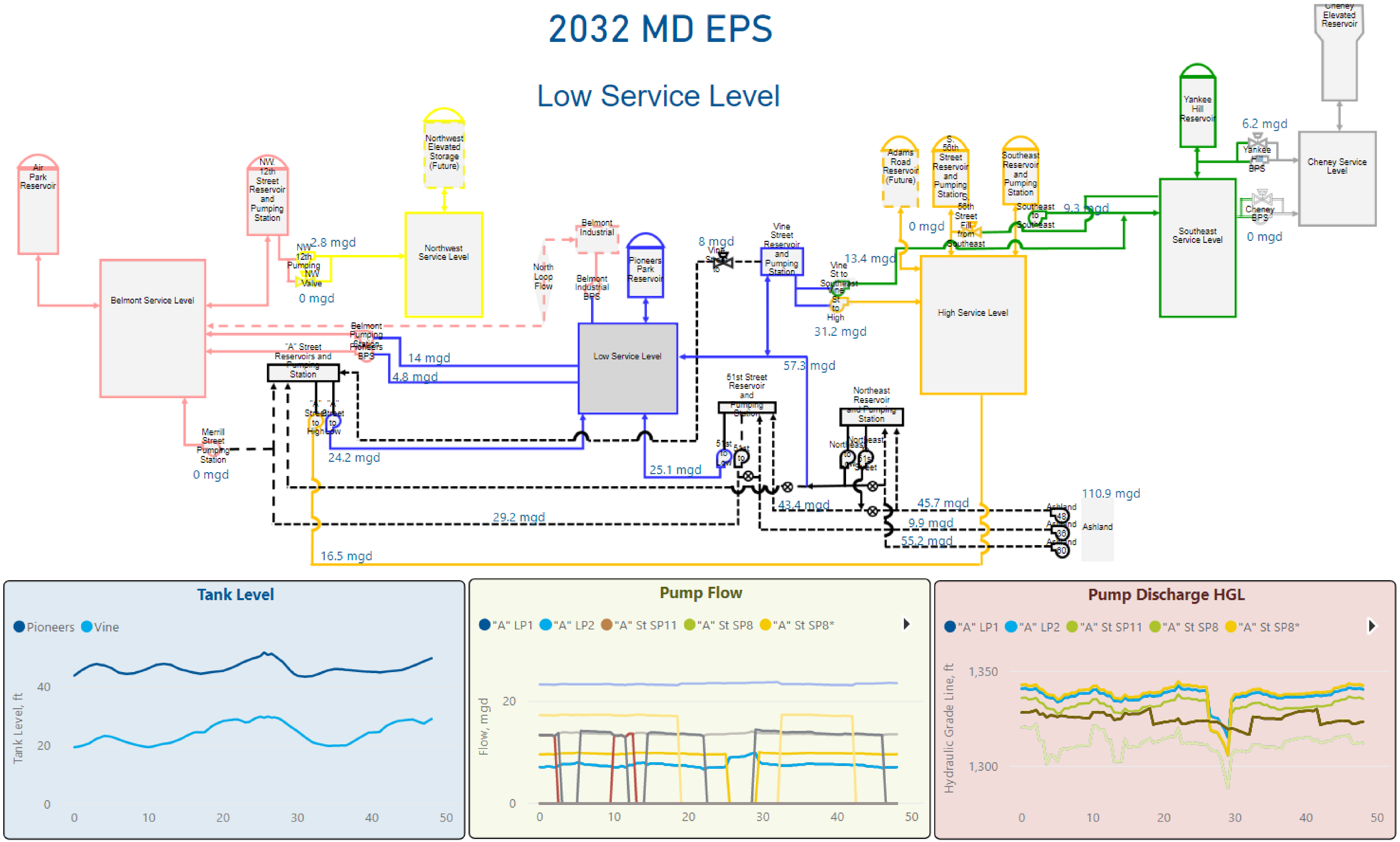


Figure 6-43 2032 MD EPS Model Results – Low Service Level

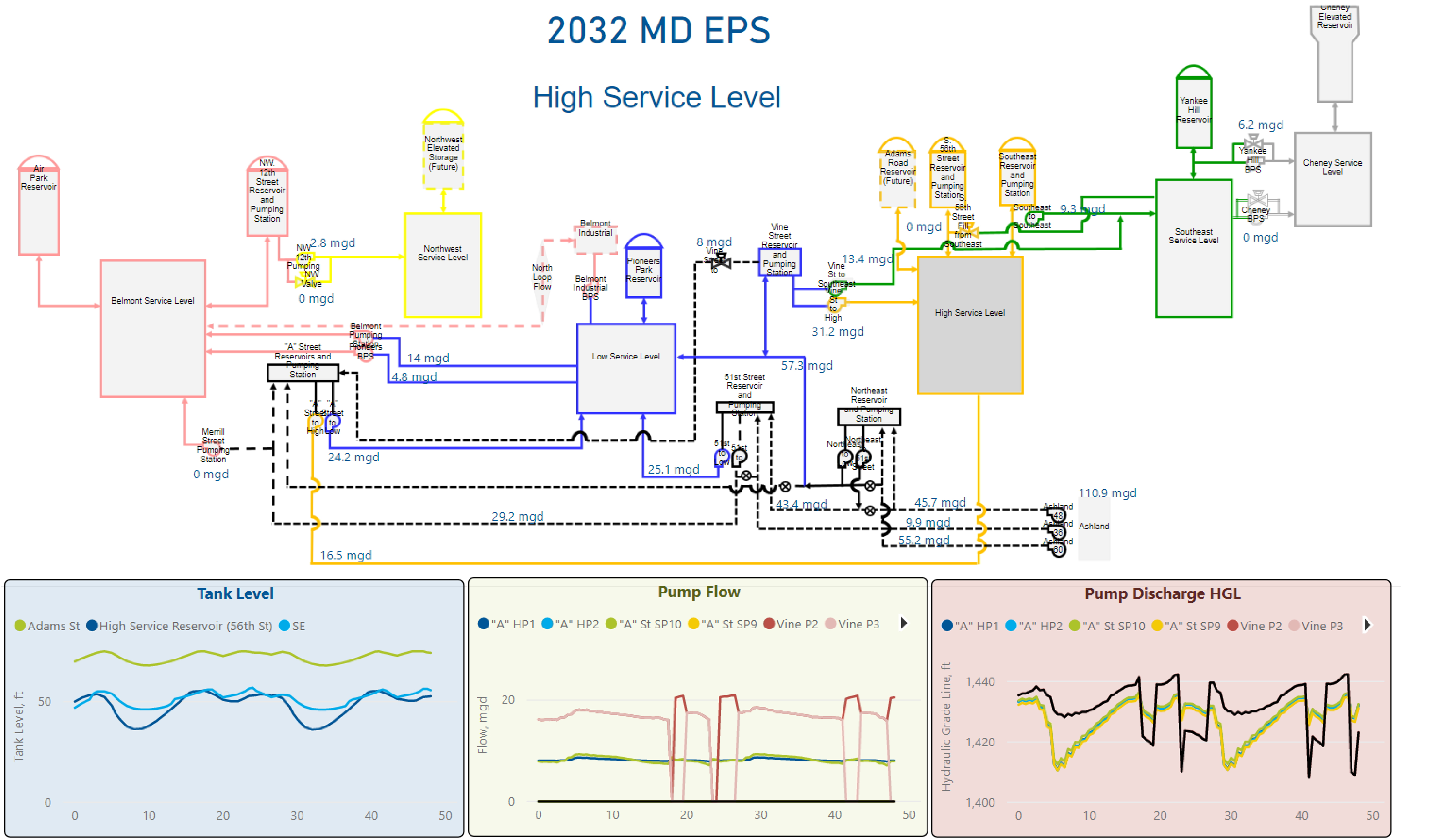


Figure 6-44 **2032 MD EPS Model Results – High Service Level**

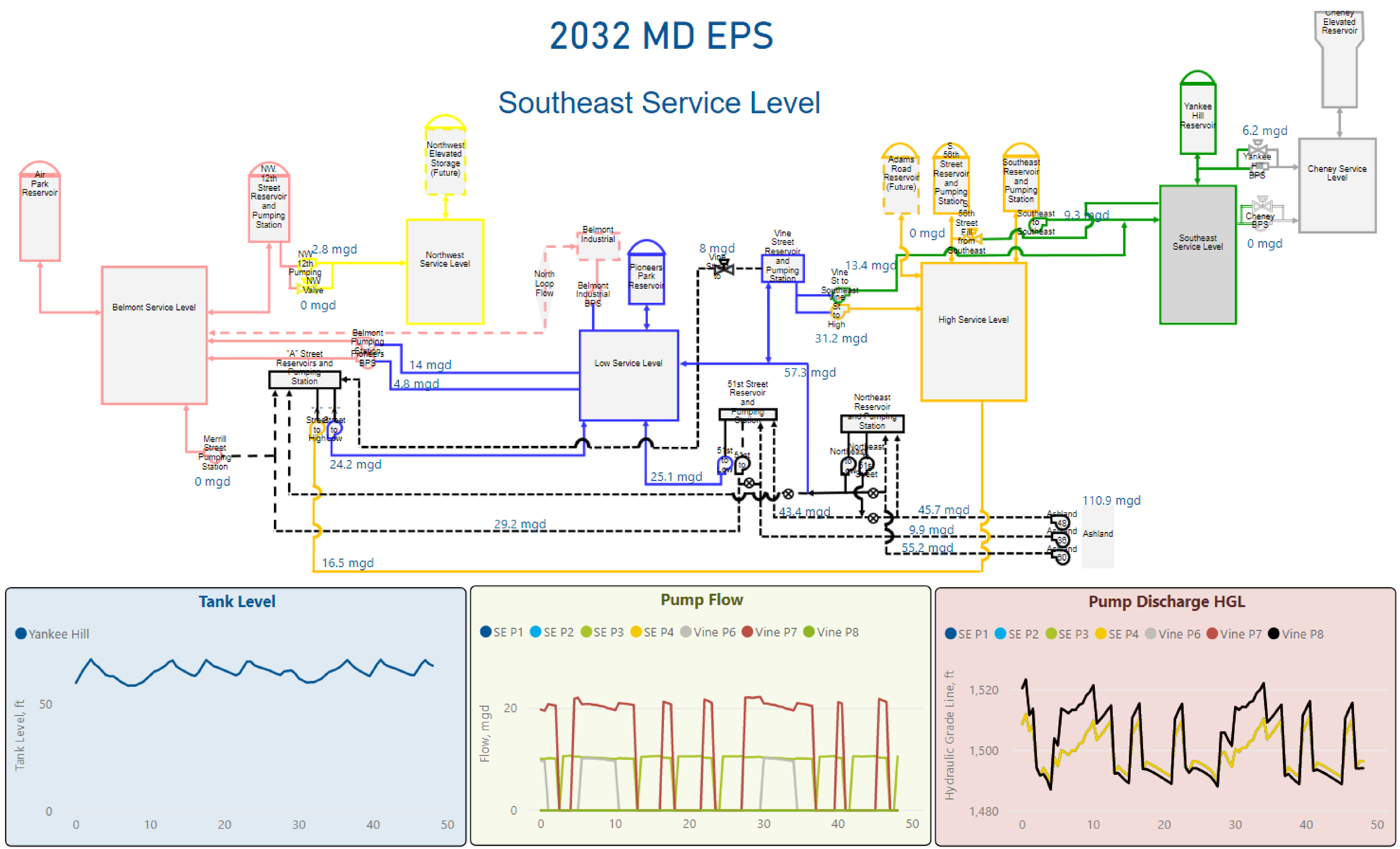


Figure 6-45 2032 MD EPS Model Results – Southeast Service Level

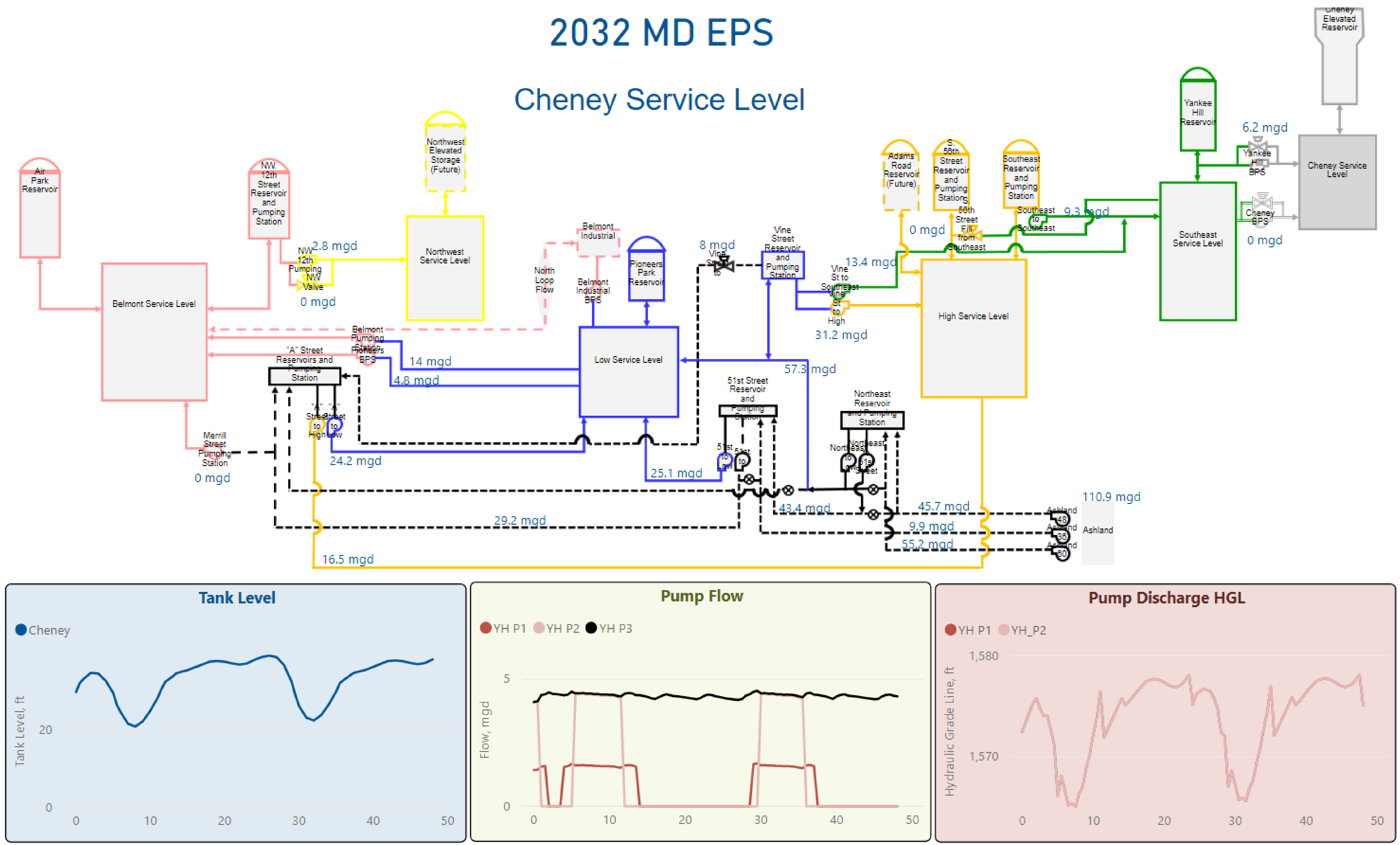


Figure 6-46 **2032 MD EPS Model Results – Cheney Service Level**

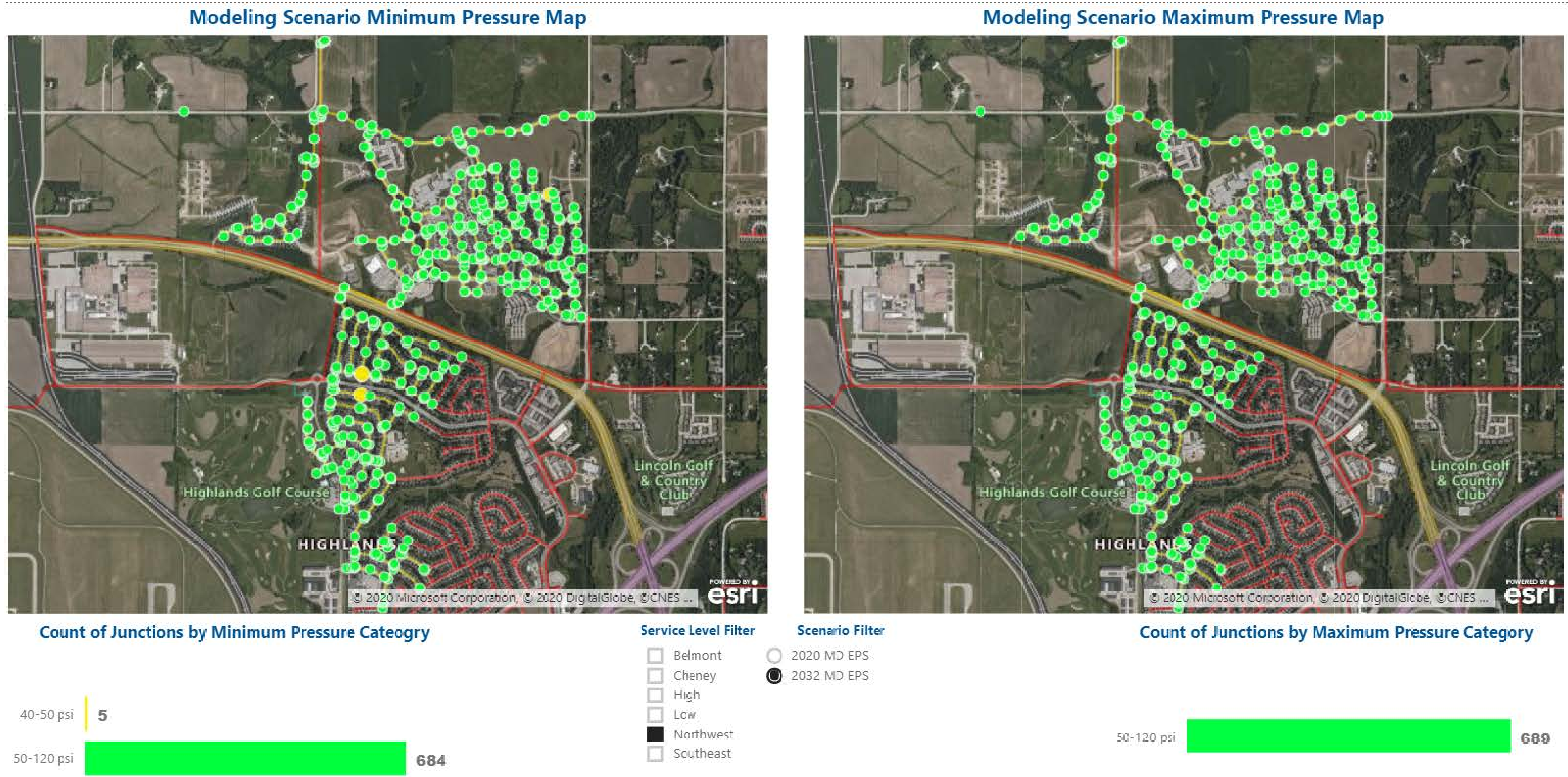
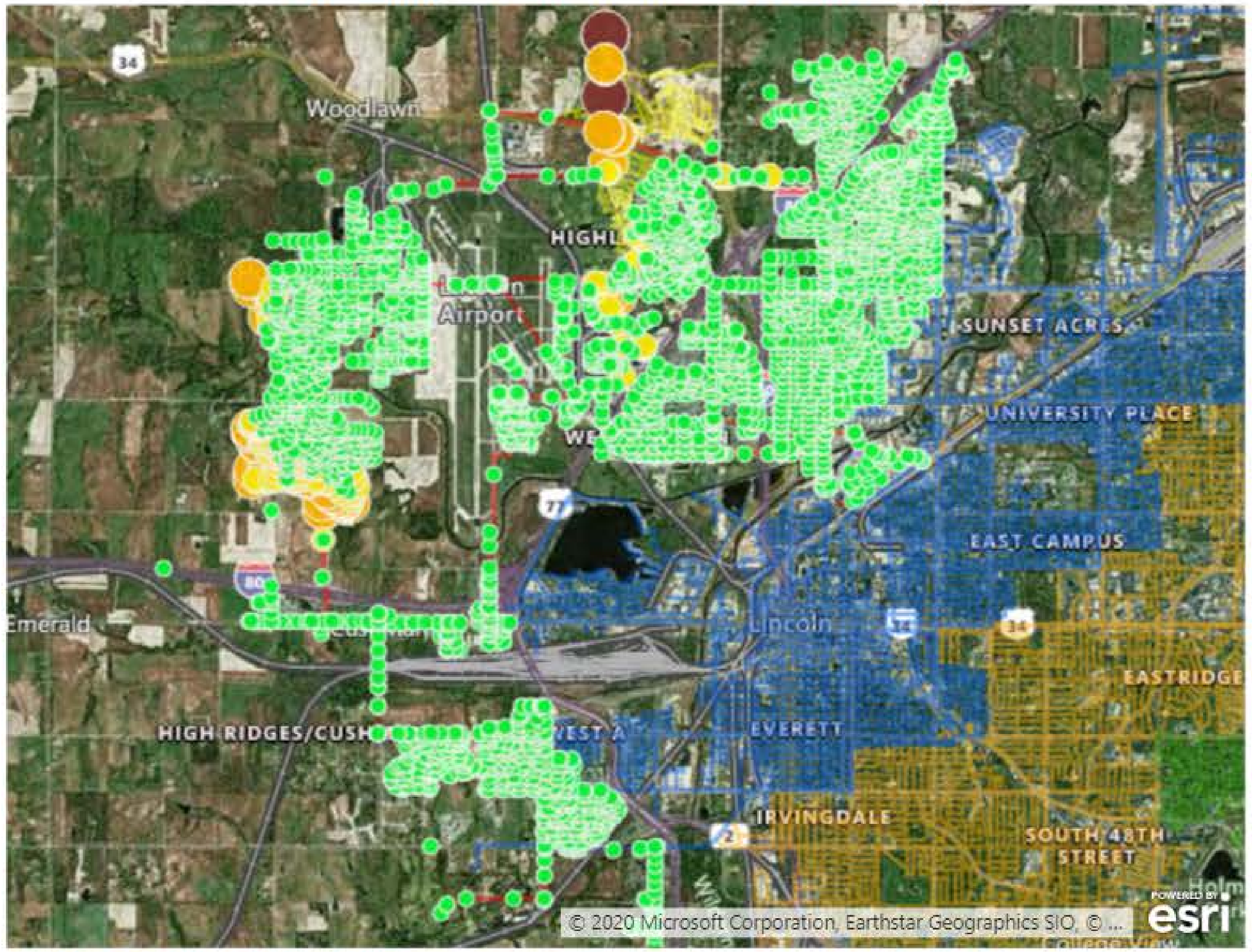
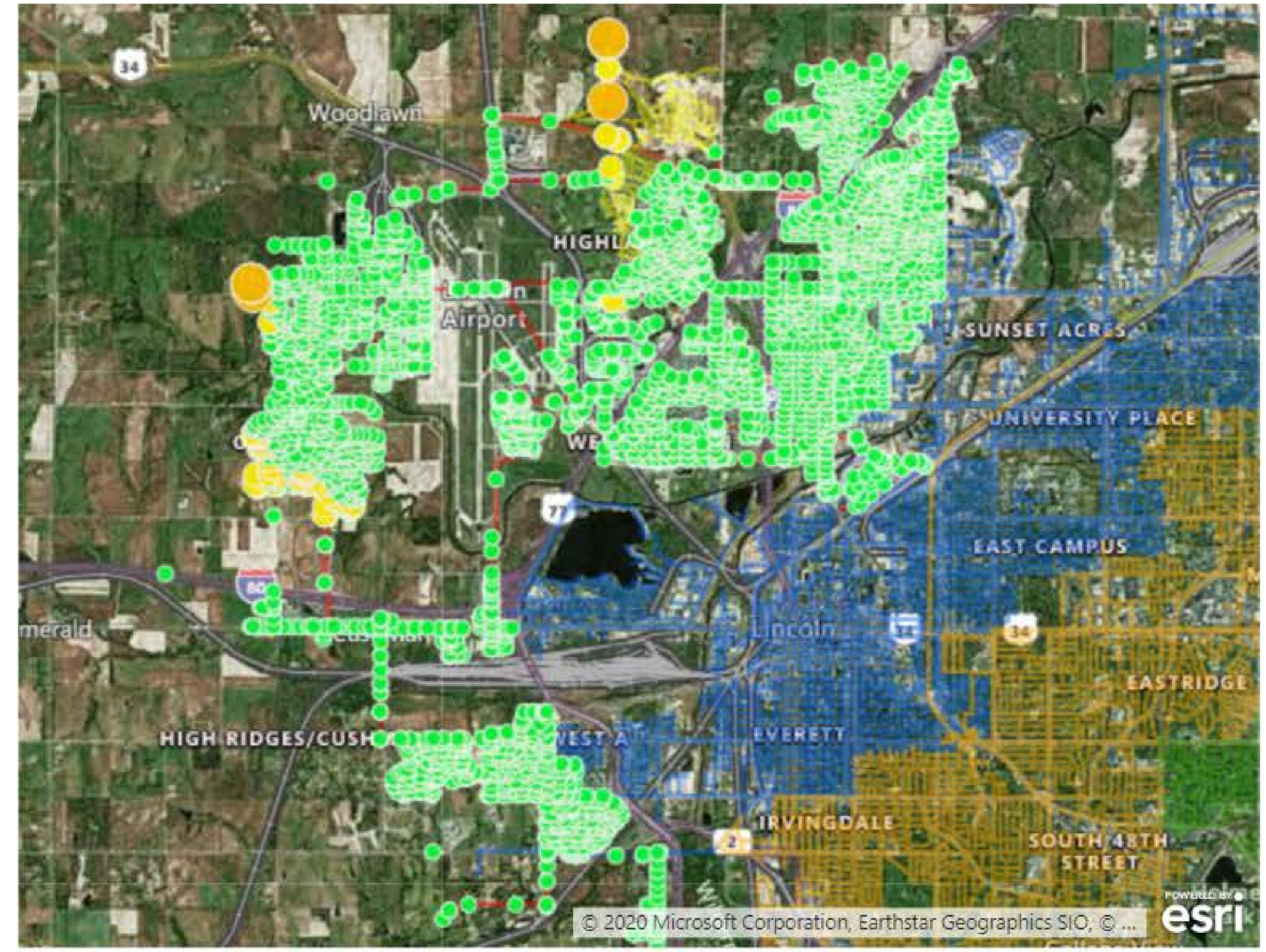


Figure 6-47 2032 MD EPS Model Pressures – Northwest Service Level

Modeling Scenario Minimum Pressure Map



Modeling Scenario Maximum Pressure Map



Count of Junctions by Minimum Pressure Cateogry

Service Level Filter

- [x] Belmont
- [ ] Cheney
- [ ] High
- [ ] Low
- [ ] Northwest
- [ ] Southeast

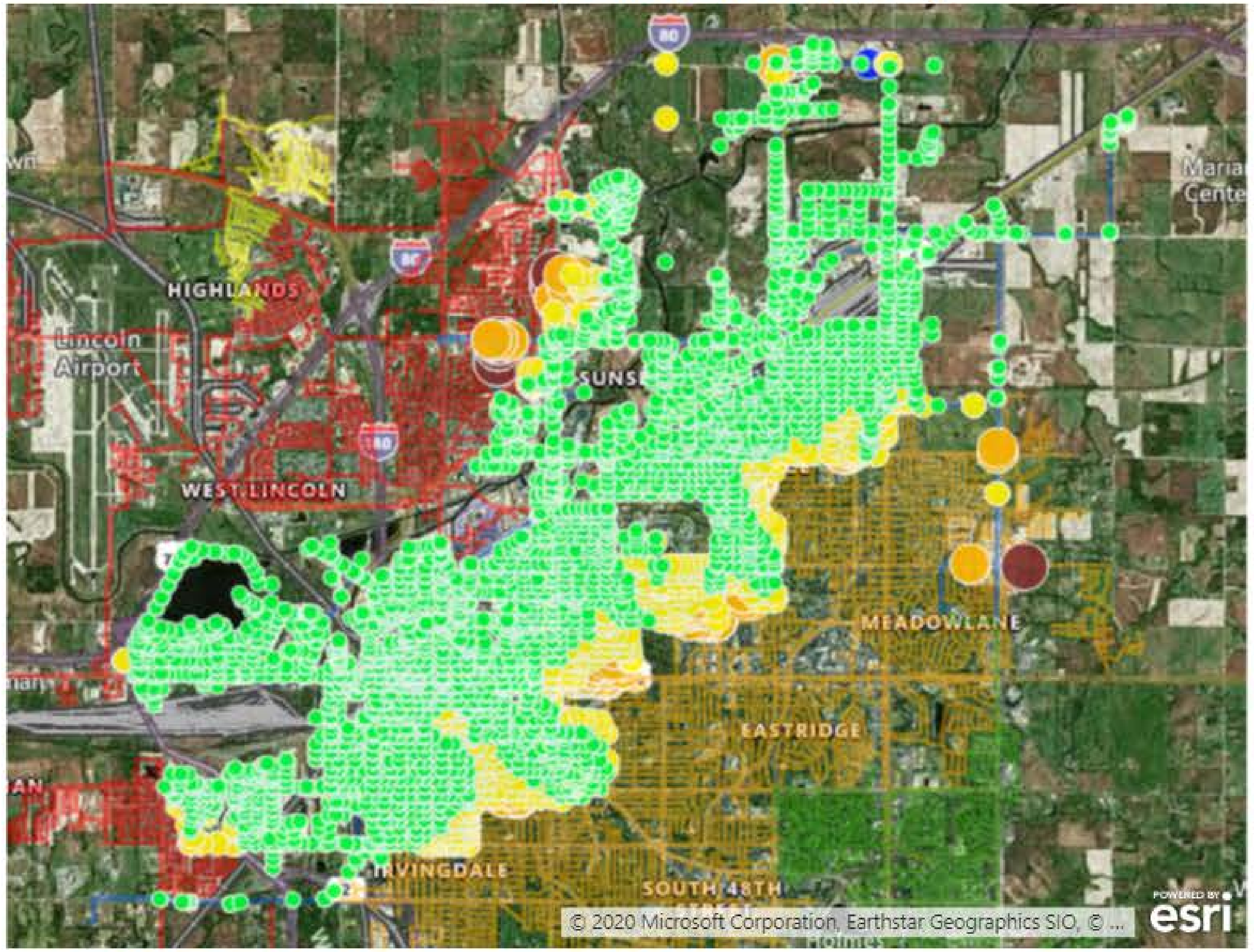
Scenario Filter

- ○ 2020 MD EPS
- ● 2032 MD EPS

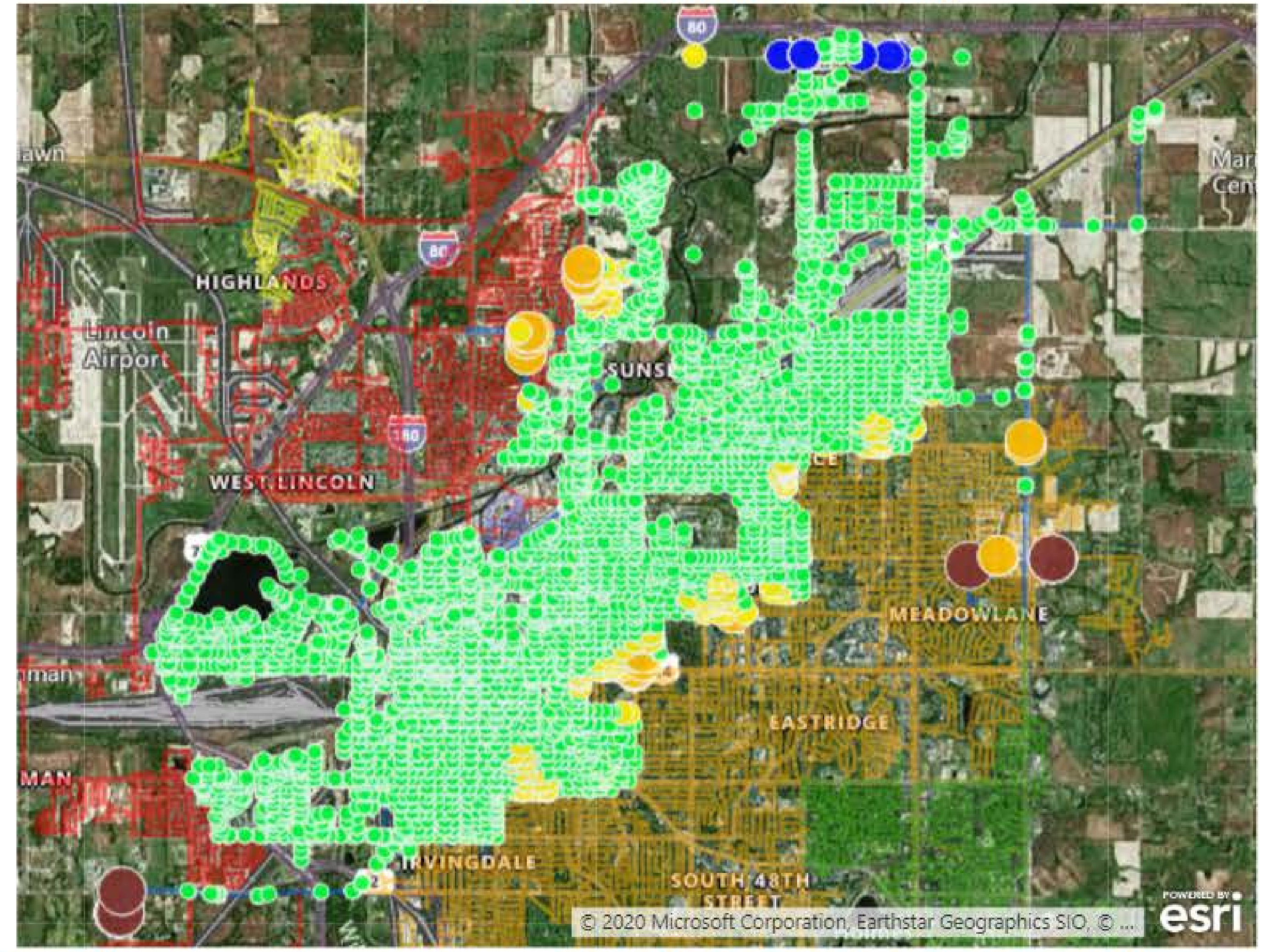
Count of Junctions by Maximum Pressure Category

Figure 6-48 2032 MD EPS Model Pressures – Belmont Service Level

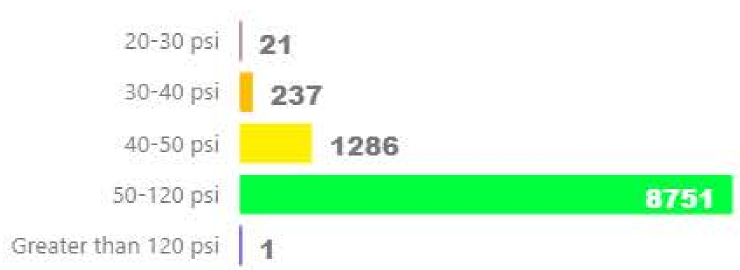
Modeling Scenario Minimum Pressure Map



Modeling Scenario Maximum Pressure Map



Count of Junctions by Minimum Pressure Cateogry



Service Level Filter

- ☐ Belmont
- ☐ Cheney
- ☐ High
- ■ Low
- ☐ Northwest
- ☐ Southeast

Scenario Filter

- ○ 2020 MD EPS
- ● 2032 MD EPS

Count of Junctions by Maximum Pressure Category

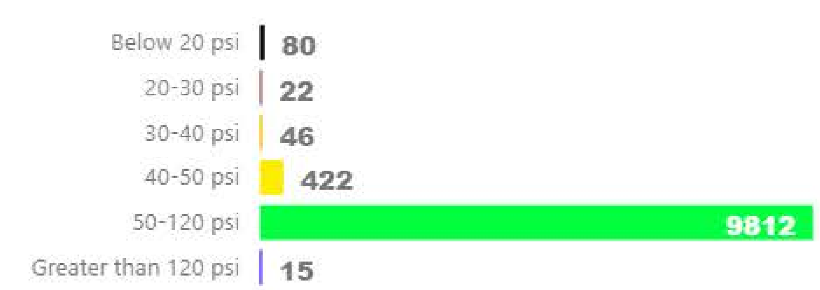
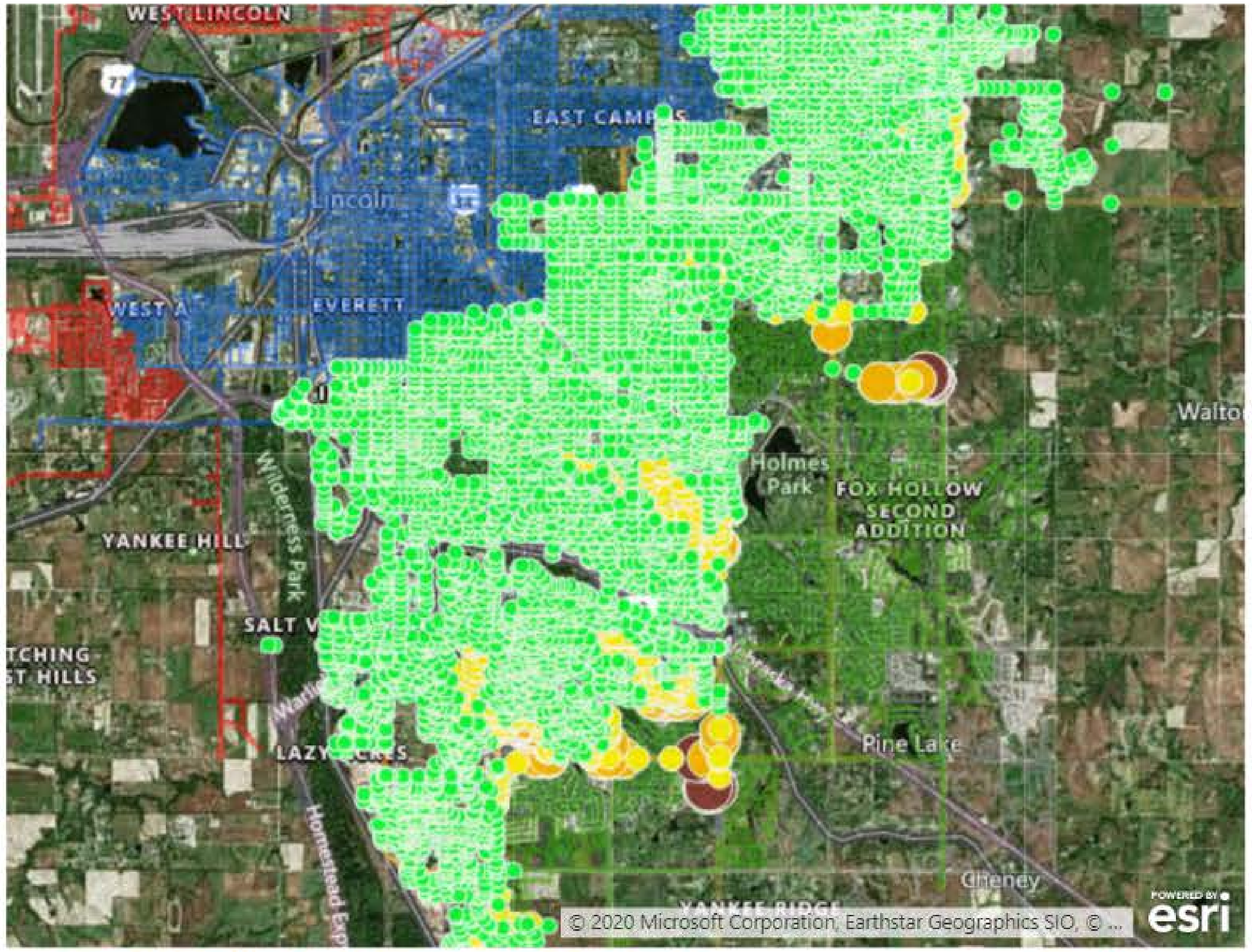
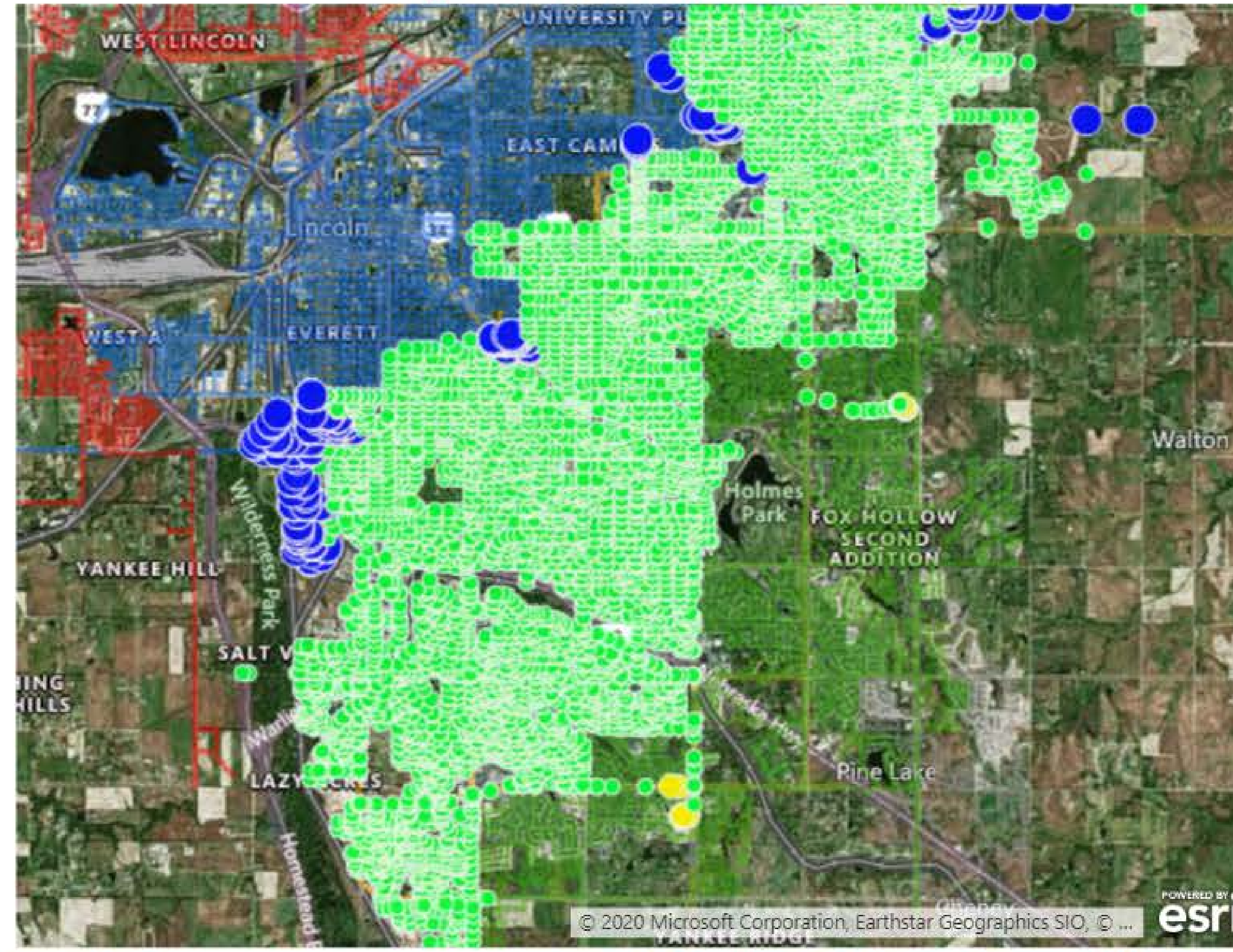


Figure 6-49 2032 MD EPS Model Pressures – Low Service Level

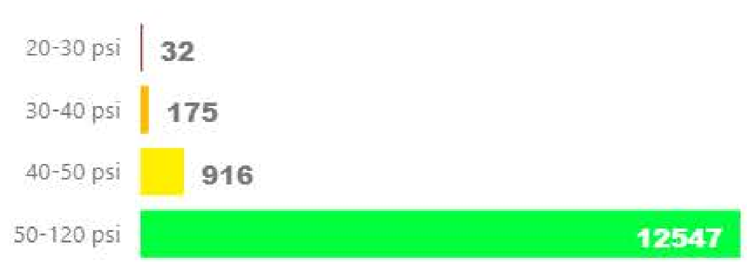
Modeling Scenario Minimum Pressure Map



Modeling Scenario Maximum Pressure Map



Count of Junctions by Minimum Pressure Category



Service Level Filter

- Belmont
- Cheney
- High
- Low
- Northwest
- Southeast

Scenario Filter

- 2020 MD EPS
- 2032 MD EPS

Count of Junctions by Maximum Pressure Category

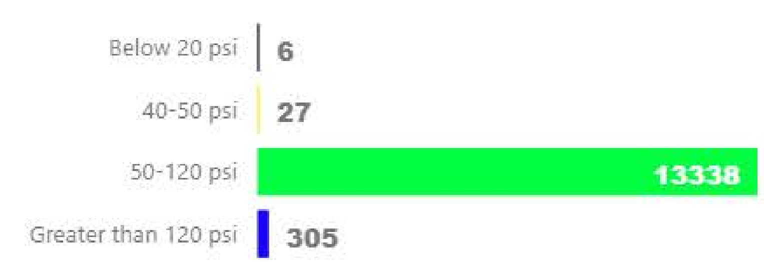
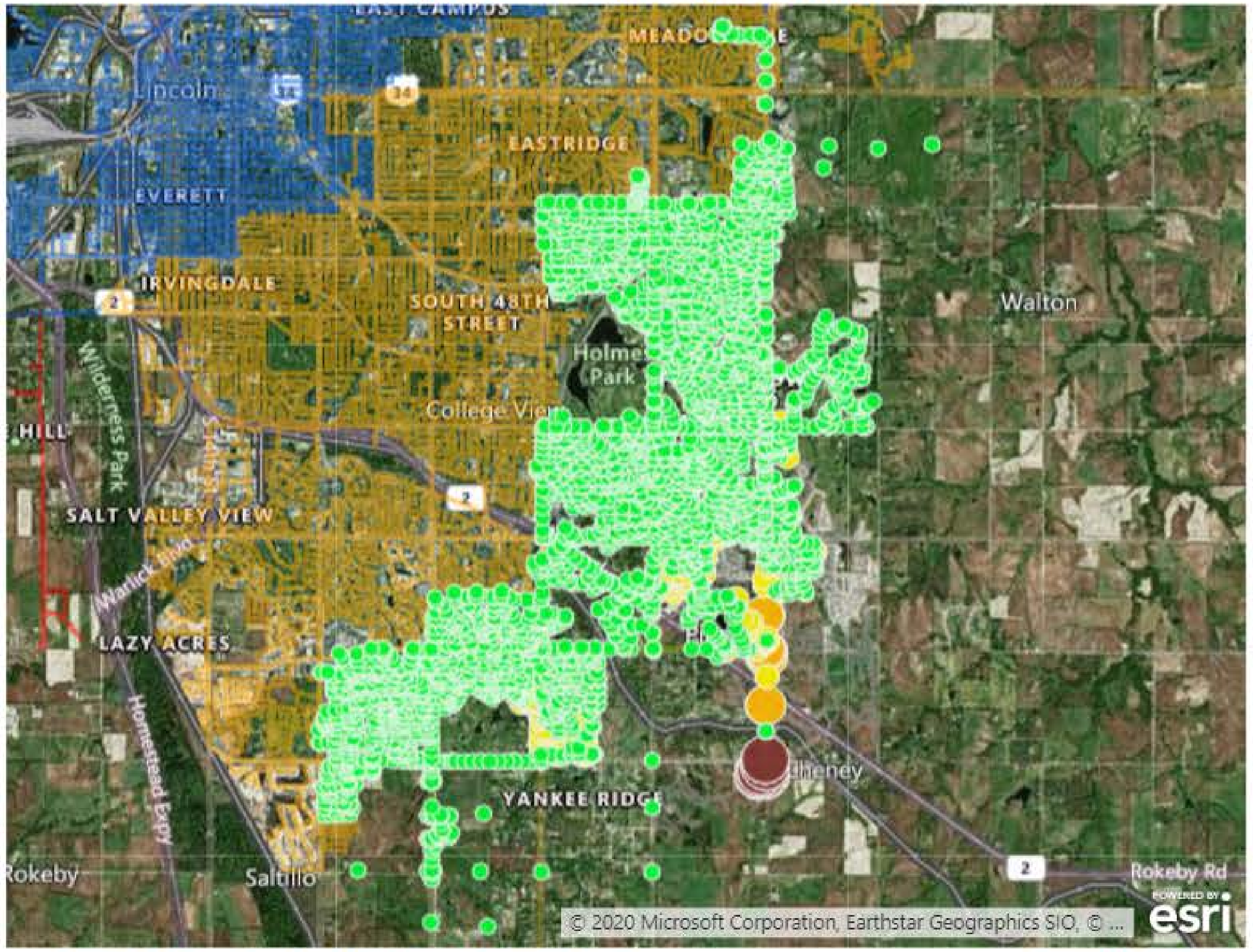
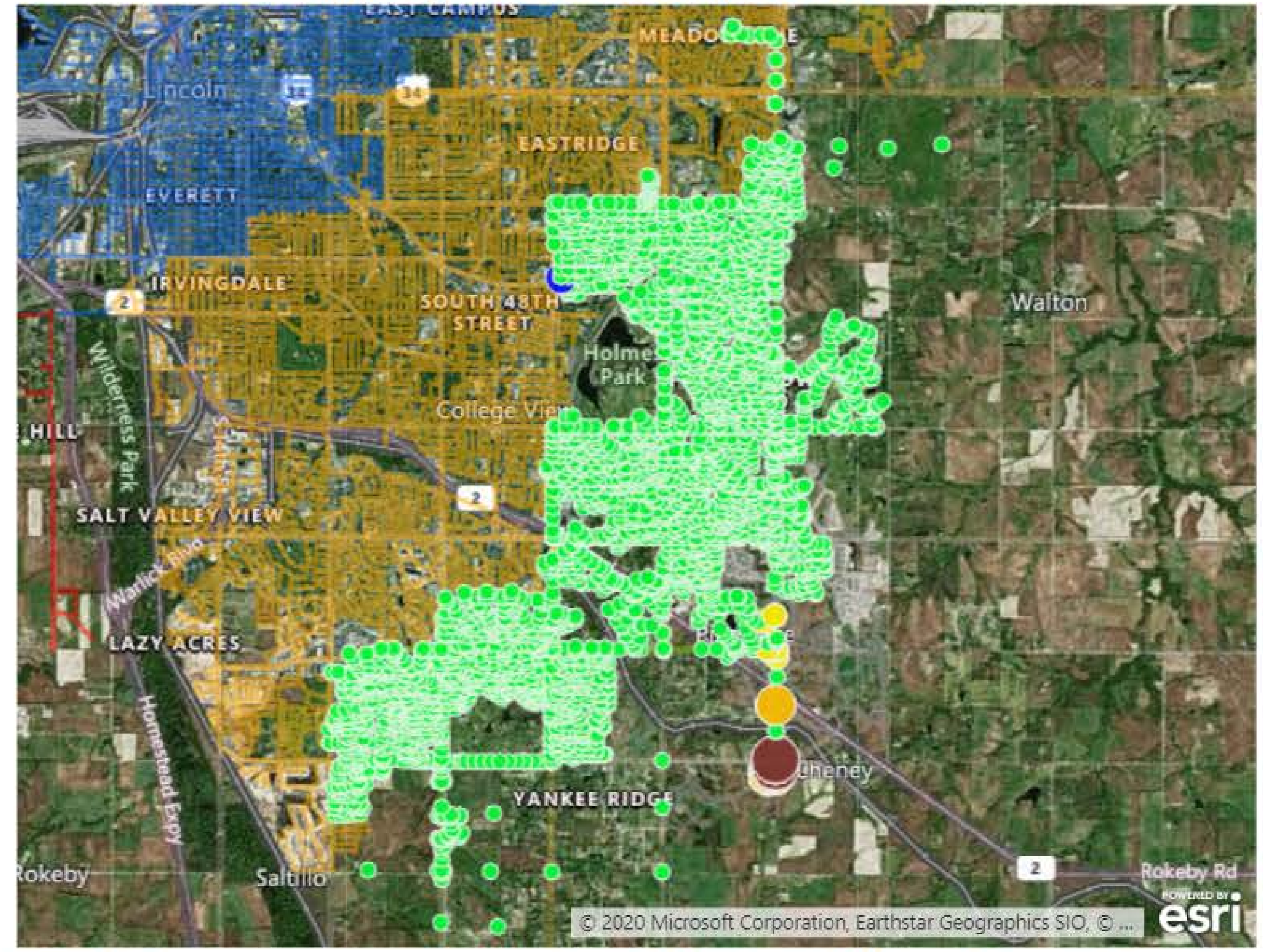


Figure 6-50 2032 MD EPS Model Pressures – High Service Level

Modeling Scenario Minimum Pressure Map



Modeling Scenario Maximum Pressure Map



Count of Junctions by Minimum Pressure Cateogry

Service Level Filter

- Belmont
- Cheney
- High
- Low
- Northwest
- Southeast

Scenario Filter

- 2020 MD EPS
- 2032 MD EPS

Count of Junctions by Maximum Pressure Category

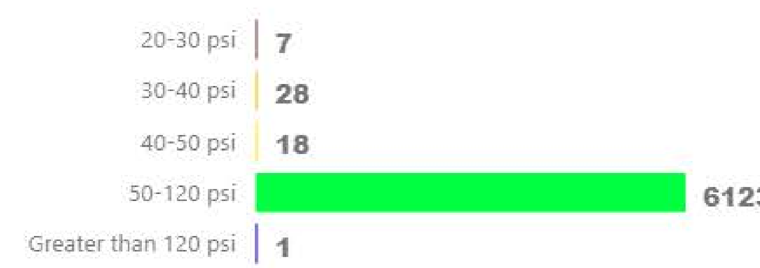
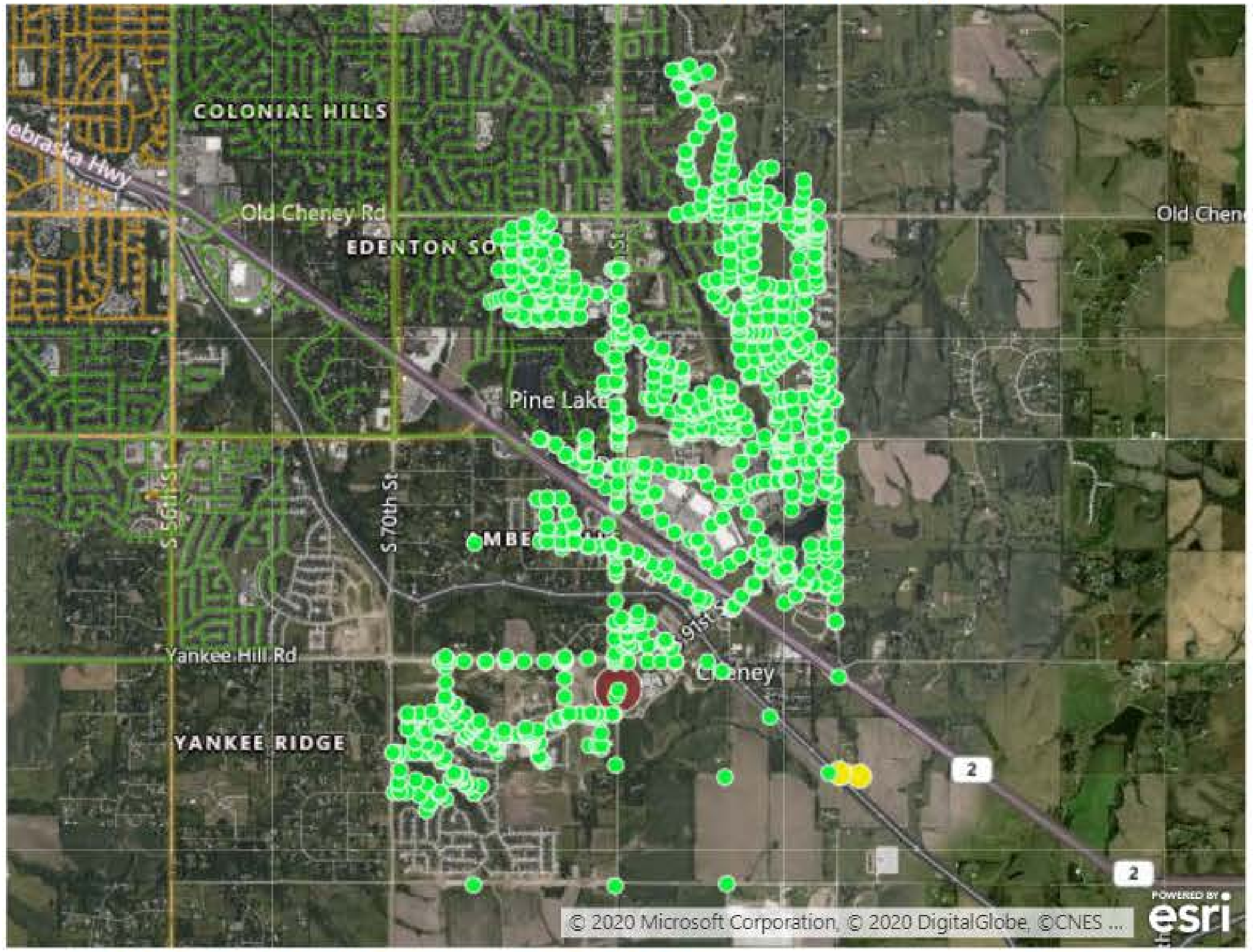
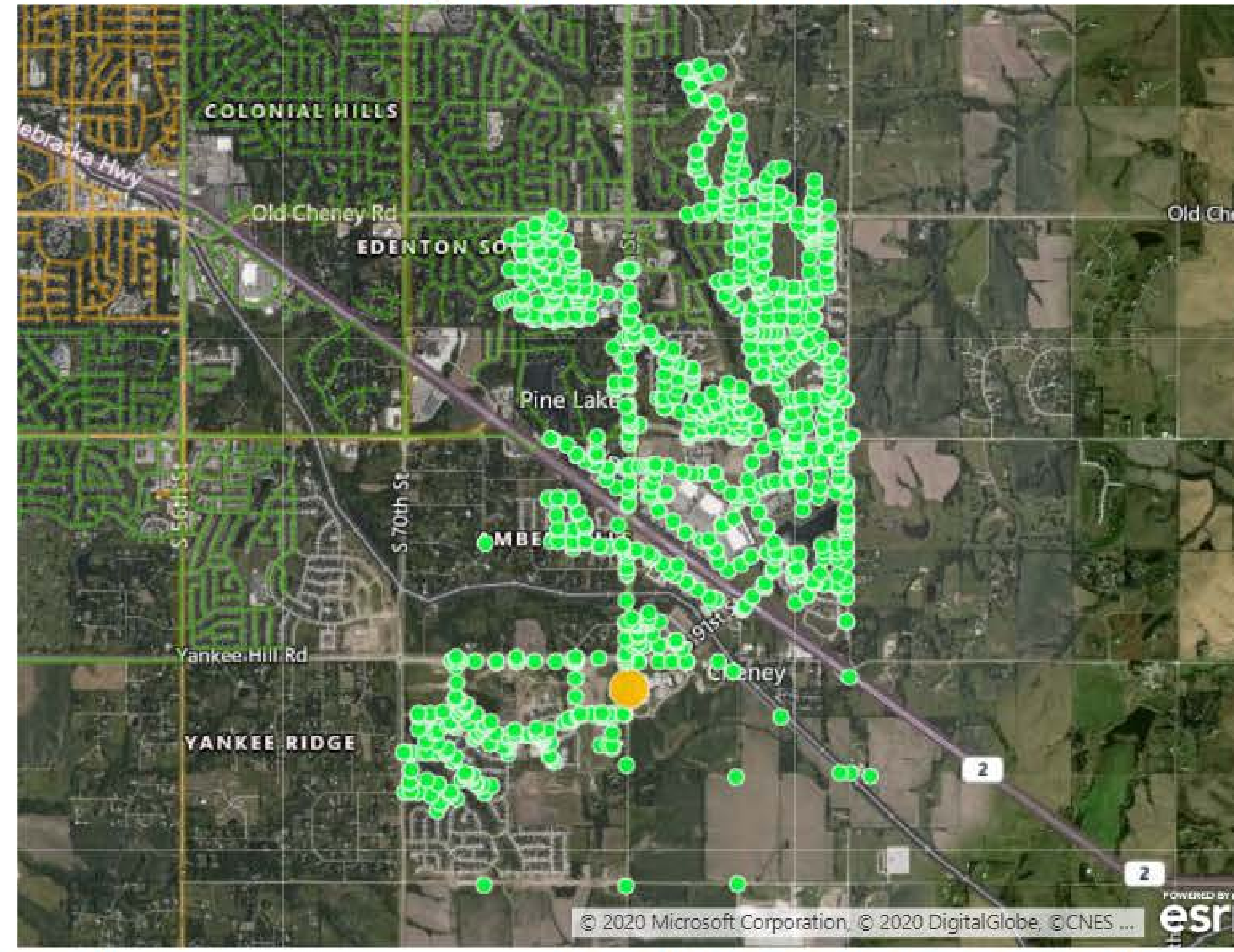


Figure 6-51 2032 MD EPS Model Pressures – Southeast Service Level

Modeling Scenario Minimum Pressure Map



Modeling Scenario Maximum Pressure Map



Count of Junctions by Minimum Pressure Cateogry

Service Level Filter

- Belmont
- Cheney
- High
- Low
- Northwest
- Southeast

Scenario Filter

- 2020 MD EPS
- 2032 MD EPS

Count of Junctions by Maximum Pressure Category

Figure 6-52 2032 MD EPS Model Pressures – Cheney Service Level

7.0 Distribution System Water Quality

This chapter describes the results of water quality monitoring for nitrification and compliance assessments for Stage 2 DBPR and LCR. A detailed analysis of distribution system water quality as it pertains to nitrification monitoring and control was conducted based on distribution system water quality data provided by LWS. LWS collects samples for distribution system water quality analysis from nearly 160 monitoring sites located throughout the distribution system. Approximately 120 of the sample locations are for compliance with the Total Coliform Rule (TCR), and another 25 sample locations are for general distribution system water quality monitoring for operational purposes. Additionally, LWS collects samples from a minimum of 50 sites for Lead and Copper Rule (LCR) compliance monitoring and 7 sites for Stage 2 Disinfectants and Disinfection Byproducts Rule (Stage 2 DBPR) compliance monitoring.

This chapter also provides a summary of distribution system water quality modeling to characterize relationships between water age and degradation of chlorine residual. As part of the distribution system evaluation, alternatives such as implementation of chloramine booster stations and installation of PRVs, were modeled to identify viable solutions for distribution system water quality improvements. Pilot and full-scale testing procedures to evaluate the effectiveness of recommended distribution system water quality improvements are also included herein.

7.1 Disinfection Byproducts

As noted in Chapter 5, LWS must maintain compliance with all regulated disinfection byproducts (DBPs) summarized in Table 7-1. Stage 1 Disinfectant and Disinfection Byproduct Rule (Stage 1 DBPR) defined maximum contaminant limits (MCLs) for total trihalomethanes (TTHM), the five regulated haloacetic acids (HAA5), chlorite and bromate. Subsequently, the Stage 2 DBPR revised compliance with the MCLs for TTHMs and HAA5s to be based on a locational running annual average (LRAA) of individual DBP monitoring sites, whereas compliance with chlorite and bromate MCLs is based on the running annual average (RAA) at the point of entry (POE).

Table 7-1 Maximum Contaminant Levels for Disinfection Byproducts

Disinfection Byproducts	MCL (mg/L)
Total trihalomethanes (TTHM)	0.080
Haloacetic acids (HAA5)	0.060
Chlorite	1.0
Bromate	0.010

7.1.1 Bromate

Since the East Plant includes ozonation, bromate monitoring is required at the South Pump Station POE. As specified under Stage 1 DBPR, the MCL for bromate is 10 µg/L and compliance is monitored based on the RAA of monthly measurements or quarterly measurements for systems on reduced monitoring. Reduced monitoring can be obtained if the raw water bromide RAA is less than 0.05 mg/L or if the bromate RAA is less than 2.5 µg/L at the POE. LWS has been on reduced quarterly monitoring for bromate since the third quarter of Year 2013 based on their ability to maintain a bromate RAA of less than 2.5 µg/L at the POE.

Figure 7-1 provides the individual bromate measurements and associated RAA from November 2014 to August 2018. As demonstrated by the figure, the bromate RAA has consistently been less than or equal to 2.5 μg/L with all individual measurements less than 4 μg/L.

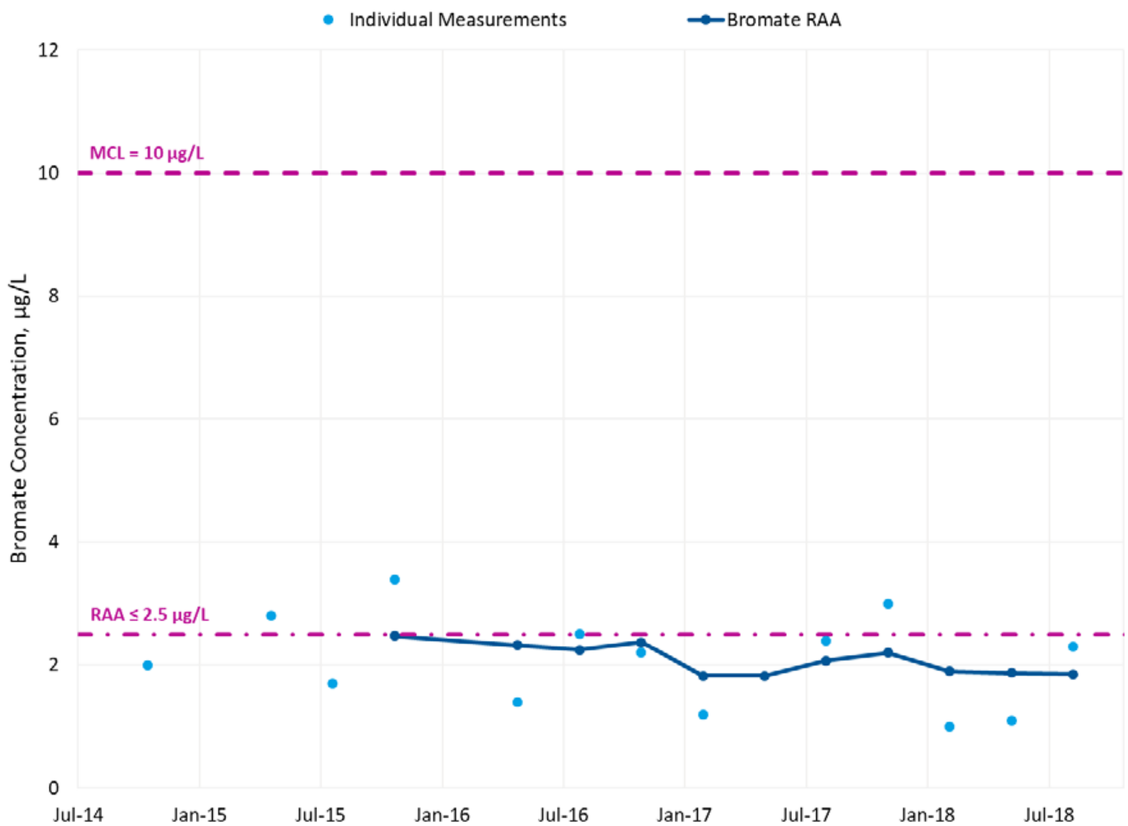


Figure 7-1 Bromate Concentration and RAA at the East Plant Point of Entry from November 2014 to August 2018

7.1.2 TTHMs and HAA5s

Based on the population served, routine monitoring normally consists of quarterly sampling from 12 monitoring sites. However, LWS is on reduced monitoring since TTHMs and HAA5s have been maintained at less than 50 percent of the MCL. Therefore, compliance with the MCL is based on the LRAA of quarterly measurements at the monitoring sites identified as 12-2H, 4-3J, and 7-4J for TTHMs and 11-5B, 9-8B and 9-9D for HAA5s. While separate sites are used for compliance monitoring of TTHMs and HAA5s, LWS collects information on both parameters at each location. Figure 7-2 and Figure 7-3 provide the LRAA from October 2015 to October 2018 for TTHMs and HAA5s, respectively, at all monitoring sites. As demonstrated in Figure 7-2, the LRAA for TTHMs has consistently been less than 40 μg/L (50 percent of the MCL). Similarly, the LRAA for HAA5s has been maintained at less than 20 μg/L (33 percent of the MCL).

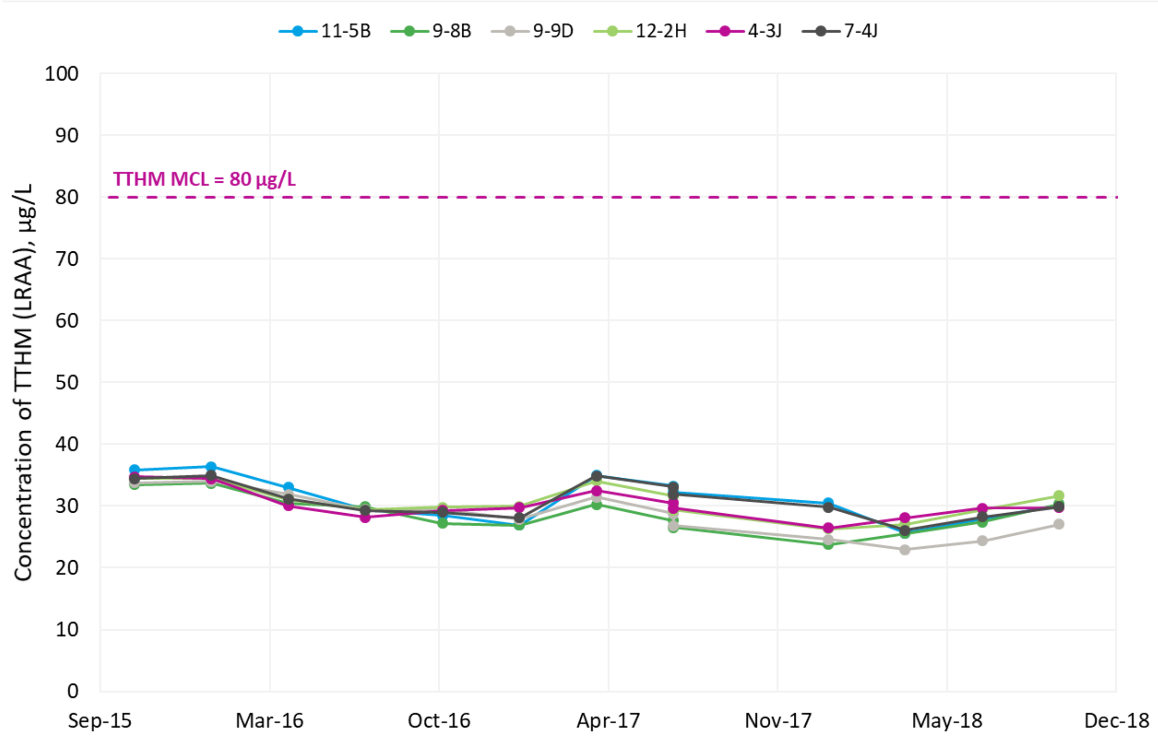


Figure 7-2 **TTHM Locational Running Annual Average from 2015 to 2018**

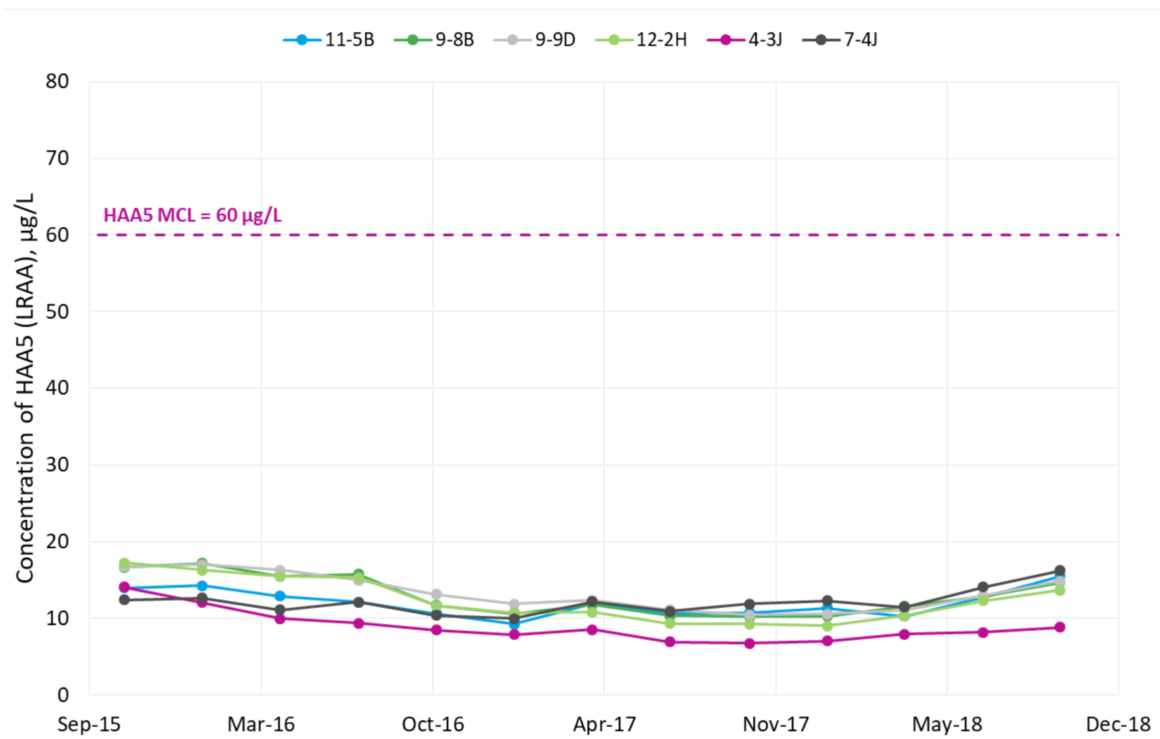


Figure 7-3 **HAA5 Locational Running Annual Average from 2015 to 2018**

7.2 Lead and Copper

7.2.1 LCR Monitoring and Compliance

LWS is currently on reduced monitoring for lead and copper, which requires LWS to monitor for LCR compliance data every three years. LWS's historical LCR compliance monitoring results for lead are shown in Figure 7-4, where the minimum value, 90th percentile compliance value and maximum value are indicated for each monitoring event. The minimum values of lead detected for each LCR monitoring event have been below detection levels and are shown as zero on Figure 7-4. The 90th percentile lead levels have always been below the lead action level of 15 µg/L, which explains how LWS is on reduced monitoring. The maximum detected lead levels have historically been less than the lead action level since 1998, but in 2016 there was a lead level measured at 403 µg/L. Due to the elevated lead level measured during the 2016 sampling event a closer evaluation was conducted for the three most recent sampling events (i.e., 2013, 2016 and 2019).

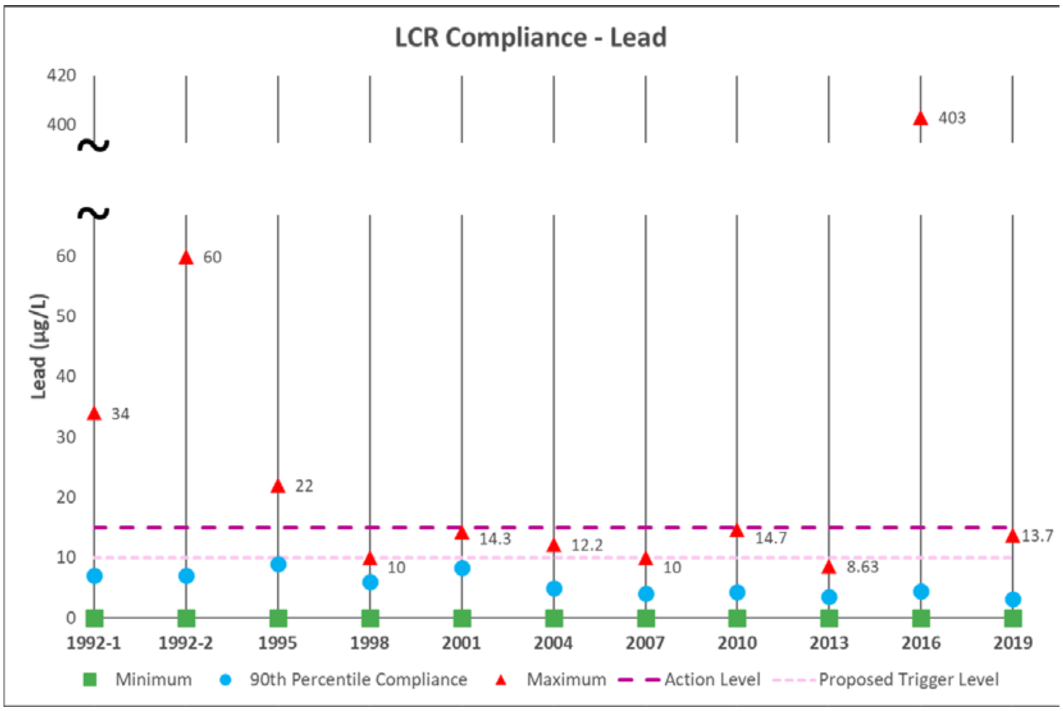


Figure 7-4 Historical LCR Compliance Monitoring for Lead

As noted in Chapter 5, the action levels for lead (15 µg/L Pb) and copper (1,300 µg/L Cu) are based on the 90th percentile ranking of the sample result data set for any particular sampling event. The three most recent lead and copper compliance results are shown in Figure 7-5 and Figure 7-6 respectively. The 90th percentile results for lead and copper have been below the action levels for each of the sampling events in 2013, 2016 and 2019.

During the 2016 sampling event, 2 of the 57 samples had lead concentrations greater than the lead action level, with results of 60.6 µg/L Pb and 403 µg/L Pb. Both locations with elevated lead levels were resampled by LWS and the results were 0.73 µg/L Pb and 55.5 µg/L Pb, respectively. The one location that still showed elevated lead levels was resampled by DHHS and the lead result was 20.2 µg/L Pb. The location with repeat levels of elevated lead during 2016 sampling was a house built in

1903 that has a lead service line, but its LCR result from 2013 only showed 3.5 µg/L Pb and in 2019 the lead result was 5.09 µg/L Pb.

Of the 57 samples analyzed during the 2016 sampling event, 32 were from houses served by lead service lines, and only one of these 32 samples had lead concentrations greater than 7 µg/L Pb. The 2019 LCR results at this location returned to low levels indicating that the spike in lead was a short-term occurrence at one location.

The 90th percentile for both lead and copper increased slightly during the Year 2016 sampling event when compared to the Year 2013 sampling event, but then the results decreased slightly during the Year 2019 sampling event. In terms of compliance with the lead and copper action levels, LWS is still well below the regulatory limits.

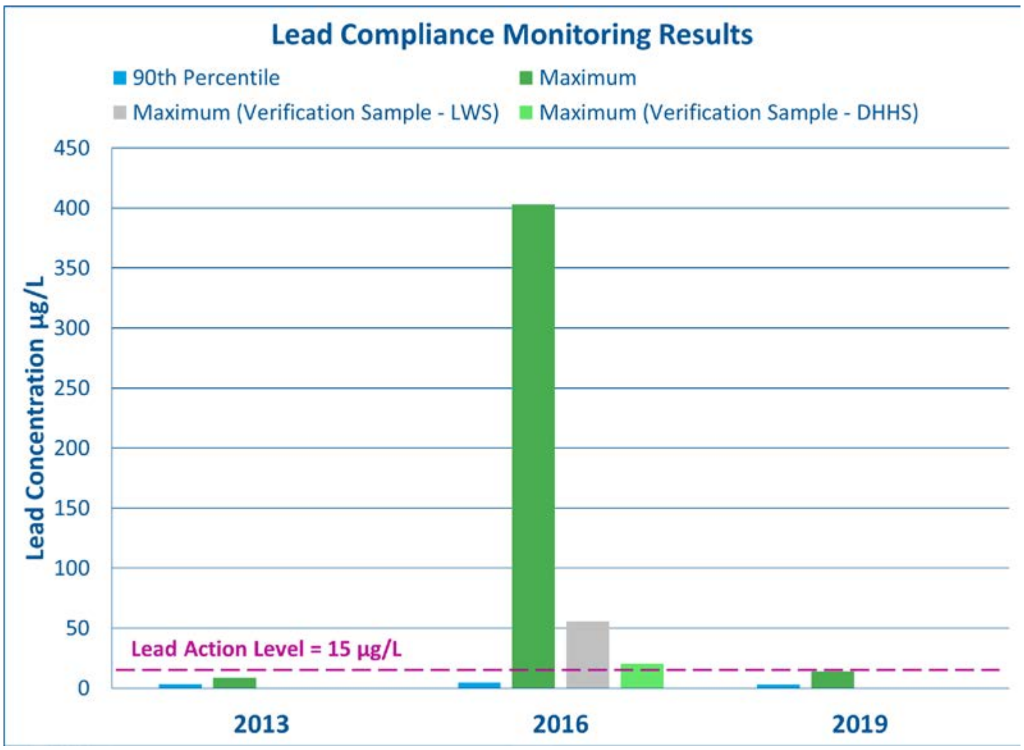


Figure 7-5 Lead LCR Compliance Data for 2013, 2016 and 2019

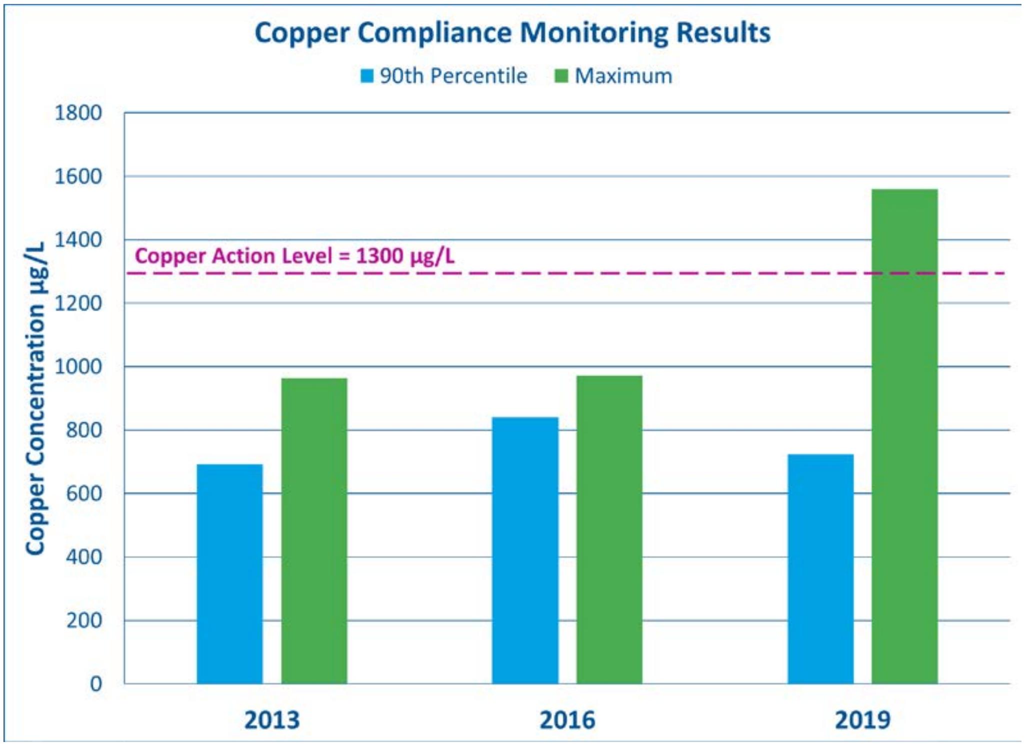


Figure 7-6 Copper LCR Compliance Data for 2013, 2016, and 2019

7.2.2 Lead Service Line Replacement Strategy

When treated drinking water enters the Lincoln distribution system, lead is not detectable. However, the presence of materials containing lead in the private service lines and premise plumbing present the opportunity for leaching of lead. There are two alternatives to limit the potential for lead to leach into drinking water: 1) remove sources of lead and 2) optimize water chemistry and corrosion control treatment to limit the solubility of lead. LWS's compliance monitoring for the Lead and Copper Rule (LCR) indicates that the system is optimized to limit the aggressiveness of the finished water toward pipe materials, as both lead and copper levels are well below respective action levels. However, there are still samples with detectable levels of lead and thus removing lead materials will benefit the finished water quality at customers' taps.

7.2.3 Identifying Lead Service Lines

Most lead pipes were installed prior to 1950. Removing lead materials such as lead service lines (LSLs) or lead goosenecks (pigtails, swings) is a difficult undertaking, as records identifying these materials are rare and difficult to locate. Typically, utilities begin the process of identifying lead pipes by reviewing the following:

- Tap cards from the initial service connection that might include the pipe material or date to confirm if lead was used at that time.
- Historic maintenance records that could explain if a repair was made to a lead pipe or if the lead pipe was removed either as a standalone project or as a result of main repairs.
- Tax records to determine the date when a building or residence was constructed.
- Plumbing permits for when buildings were renovated to determine whether a service line was replaced.

- Historic plumbing codes or ordinances to identify when specific materials were allowed for service lines.
- Discussions with personnel that have worked with the utility for an extended period to learn the typical practice for noting the replacement or repair on an LSL.

It is important to note that galvanized iron pipe downstream of lead materials should be removed when lead pipes/materials are removed as the iron can act like a sponge for dislodged lead particulate. Disturbances from stopping flow, removing lead materials and re-starting the flow of water through the service line can release lead particulate from the iron pipe and create a health risk.

7.2.4 LWS LSL Identification Program

LWS reviewed available information to identify locations with LSLs, galvanized iron service lines, and service lines of unknown material that require further investigation. LWS conducted their LSL identification by searching the following datasets:

- Scanned Water & Sewer Tap Record Image Files located on LWS's Website.
- Extracted Hansen CMMS Service Line Asset Data.
- GIS feature classes for Mains and Service Lines.
- Historical Records spreadsheets in EXCEL that include records of all Water Replacement Projects since 1975.

After reviewing the records, the data was sorted to identify potential LSLs based on the following criteria:

- Date of installation (e.g. before 1950 or blank).
- Service line pipe diameter (e.g. greater than ½-inch or blank).
- Service line status (e.g. active service line, not expired, or blank).

The compilation of these records identified approximately 4,000 potential service lines for replacement based on the review completed in October 27, 2016 as shown in Figure 7-7.

LWS's records focus on the service line material from the main to the stop box. There is a potential that the portion of the service line from the stop box to the premise plumbing could be a different material. As such, LWS is now incorporating an additional field to their dataset to try and categorize the service line material for this portion of pipe. Once the inventory is completed, the next step is to develop a plan to verify the records and begin the process of removing LSLs and downstream galvanized iron pipe, where applicable. It should be noted that in Lincoln, NE, the customer owns the entire service line from the water main to the connection with interior plumbing, as indicated in Figure 7-8.

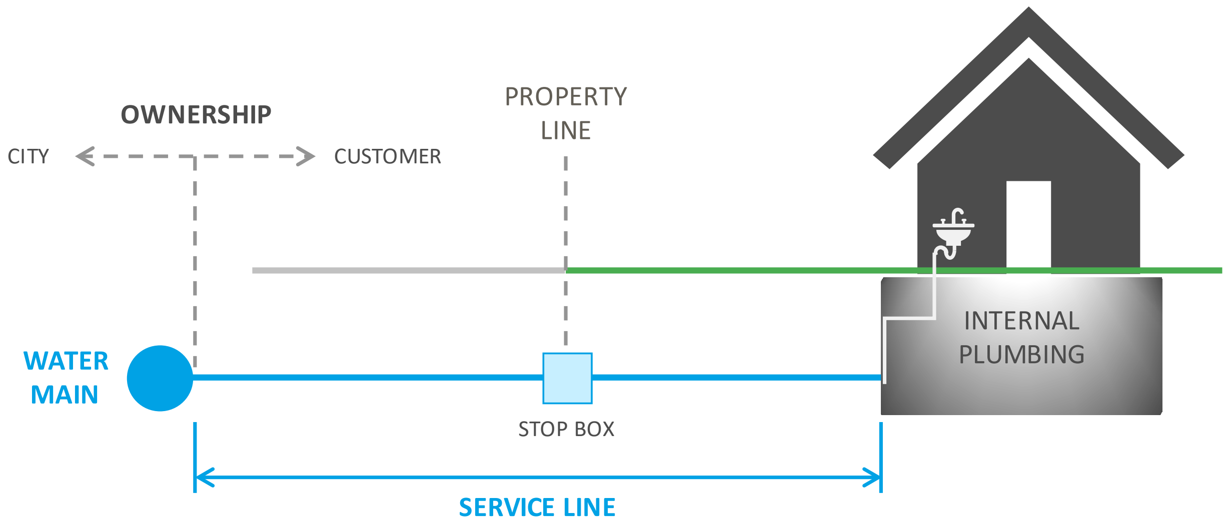


Figure 7-7 Diagram of LWS Service Line Connection to a Residence

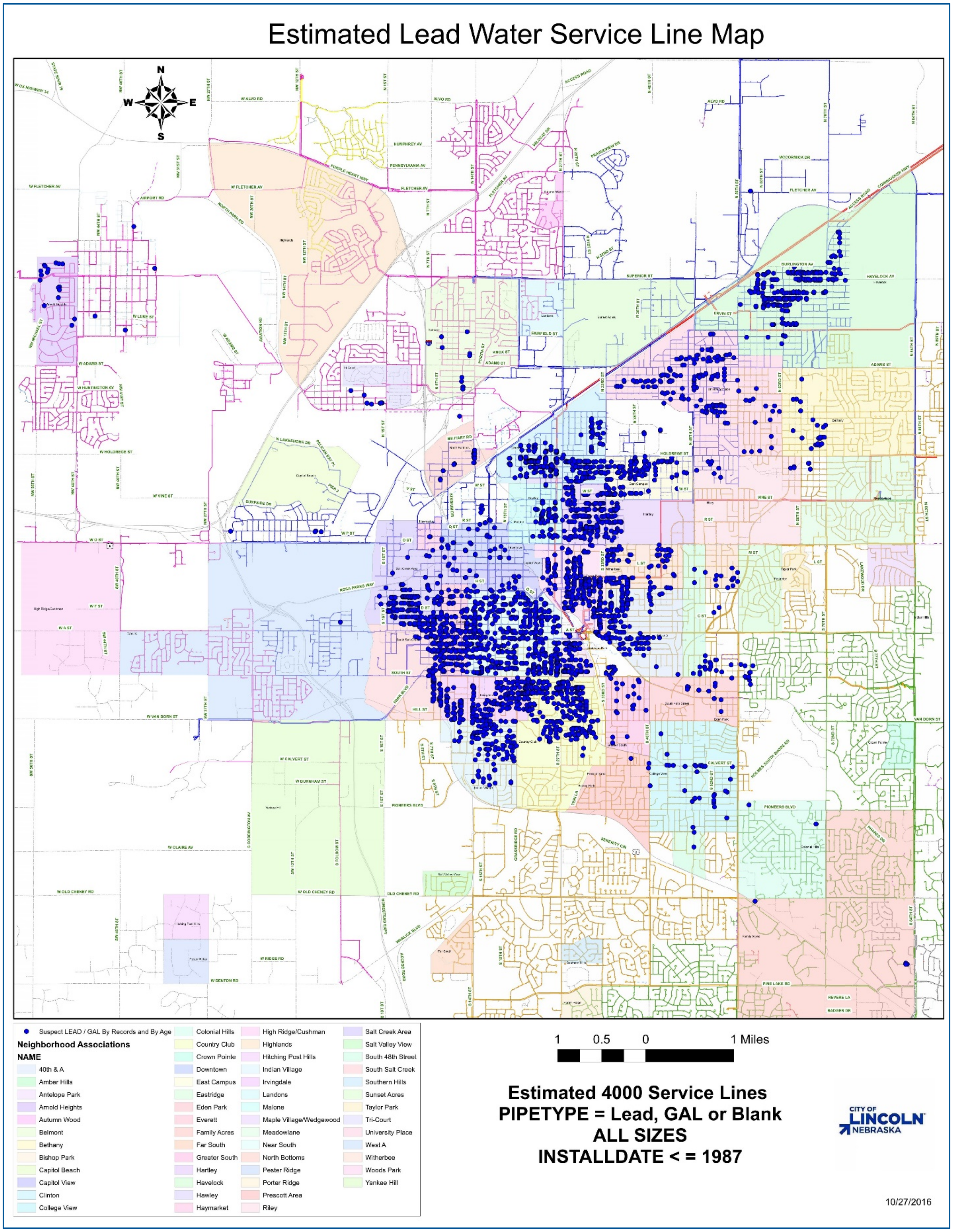


Figure 7-8 Estimated LSLs or Galvanized Iron Service Lines in Lincoln's Distribution System (10/27/2016)

7.2.5 Proposed Lead and Copper Rule Revisions

On October 10, 2019, the United States Environmental Protection Agency (USEPA) released proposed Lead and Copper Rule (LCR) Revisions that were published in the federal register on November 13, 2019. Major changes in the proposed LCR include:

- Implementation of publicly available LSL inventory.
- Proactive LSL replacement program.
- Requirement for full LSL replacement, as opposed to partial LSL replacement.
- Public outreach and educational programs.

The proposed LCR revisions include a requirement for public water systems (PWSs) to develop a publicly available LSL inventory. LWS had already begun this process prior to the release of proposed regulations and is well positioned to meet or exceed any proposed timelines established for LSL inventory development and replacement plans. The proposed LCR revisions detailed a proactive full LSL replacement program, regardless of whether the AL has been exceeded. The proposed LCR revisions also require utilities to focus on public education and engage with customers on LSL replacement plans. A distinction was made between full and partial LSL replacements, as research has shown that a partial replacement can increase the release of lead due to the disturbance of particulate lead during partial replacement activities. Full lead service line replacement includes replacing any lead pipe or downstream galvanized iron pipe between the water main and the connection to the interior plumbing of a residence or building as shown in Figure 7-7.

One uncertainty surrounding the requirement for full LSL replacement is the cost of replacement for customer-owned service lines. Subsequent sections describe funding strategies available for LSL replacement and case studies of funding options that other PWSs have utilized.

7.2.6 Funding for LSL Replacements

The State of Michigan revised its LCR in 2018, which requires PWSs to locate and remove LSLs, including the portion owned by the homeowner at the PWS's cost. The USEPA's proposed LCR revisions do not require that the PWS pay for the replacement of the LSL portion owned by the homeowner. Since the LSLs in Lincoln, NE are completely owned by the customer, LWS will have to determine if the cost of the full LSL replacement is paid by the customer, subsidized by LWS or fully paid by LWS. These funding considerations are important and there are several federally available funding programs to assist PWSs with LSL replacement programs.

The following funding options are available for LSL replacement projects:

- USEPA's Drinking Water State Revolving Fund (DWSRF) – the DWSRF has provided $1.126 billion for infrastructure improvement projects including LSL replacements in the 2019 fiscal year.
- Clean Water State Revolving Fund (CWSRF) – states have the option of transferring funds from their CWSRF to their DWSRF to address lead-related projects through October 4, 2020. The State of Nebraska might have available funding sources, which could be transferred to DWSRF for LSL replacement projects.
- Community Development Block Grant (CDBG) – the US Department of Housing and Urban Development developed the CDBG to provide communities with resources to address a wide range of projects including LSL replacement programs.

- USEPA's Water Infrastructure Improvements for the Nation (WIIN) Act – this act provides federal funding to address LSL replacement projects.
- Water Infrastructure Finance and Innovation Act (WIFIA) – utilities can pursue a low interest rate federal loan through the WIFIA program administered by the USEPA. WIFIA loans provide funding for water infrastructure related projects and improvements.

With the proposed LCR revisions focusing on identifying the sources of lead and removing them from the distribution system, it is anticipated that more federally-available funds will be allocated for LSL replacement projects when the final rule is promulgated. It is anticipated that the final LCR revisions will be published in 2020 and that the rule will allow PWSs to develop their distribution system material inventories and LSL replacement programs over a three-year period. All these steps are expected to take a few years to implement, which would provide Federal and State governments the opportunity to set aside more funding to assist with LSL replacement projects.

7.2.7 Examples of LSL Replacement Programs

Some PWSs have proactively started to replace LSLs and pay or provide financing options for the replacement on the homeowner's side to ensure that all parts of the community receive the highest quality water regardless of economic status. Below are a few examples of approaches that PWSs have taken to address LSL replacement programs and funding options for customers:

- Milwaukee Water Works began a program in 2017 to remove LSLs and they developed a special financing option to help the homeowner's pay for their portion of the LSL over 10 years.
- Philadelphia Water created a Homeowner's Emergency Loan Program (HELP) to provide customers a zero-interest loan to be paid back over a 60-month period.
- The City of Madison Wisconsin chose to replace LSLs rather than change chemical treatment that would have dramatically increased capital and operating costs, and now the City is reimbursing customers a portion of what they paid to replace their portion of the LSLs.
- The Boston Water and Sewer Commission created an incentive program to offer its customers a credit of up to $2,000 to allow the utility to replace the full LSL at one time, and the customer can finance the remainder of the cost interest free over 48 months.

7.2.8 LSL Replacement Plan Development

The development of an LSL replacement plan with the appropriate prioritization is key to limiting disturbances to the infrastructure, minimizing inconveniences for the community and maximizing the funds available.

Replacing an LSL involves coordination between the utility, the homeowner, potentially the current tenant if the residence is a rental property, and the contractor who will be replacing the LSL. When designing an LSL replacement plan LWS should coordinate with the department of transportation and City officials to overlap activities so that when a road is being repaired or replaced that any lead service lines can be replaced while already under construction to limit the disturbances to pipes, roads, and homeowner's property. Prioritization of LSL replacements should be based on a combination of both the health risk for vulnerable populations and the cost-effectiveness of replacement.

If there are no main replacement projects or street improvement projects scheduled in areas with LSLs, then prioritization should be given to areas with vulnerable populations such as the following:

- Registered childcare facilities or areas with high populations of children.
- Areas with longer water age or lower disinfectant residuals.
- Older areas with a higher likelihood of premise plumbing containing lead pipe, copper pipe with lead solder, galvanized pipe, or older brass fittings and fixtures with higher levels of lead.
- LCR monitoring locations with elevated levels of lead (i.e., "find-and-fix" description in the proposed LCR).

The proposed LCR describes that a PWS would be required to replace the water system-owned portion of the LSL within 45 days if the homeowner chooses to replace their portion of the LSL. It would be beneficial for the customer to have the entire LSL replaced at one time to avoid a partial LSL replacement. LWS could develop a list of customers that would like to pay to replace their portion of the LSL and coordinate with contractors to limit the effort and have the entire LSL replaced at one time. The LSL replacement program should take into consideration that lower income households may not have financial ability to take part in full LSL replacement. The program needs to account for all considerations and design a plan that will help the funds to go the furthest by combining LSL replacements with other infrastructure improvement projects while also focusing on areas with vulnerable populations.

7.2.9 LSL Replacement Activities

Prior to conducting an LSL replacement, the contractors must be trained to understand the importance of delicately removing lead or galvanized piping to avoid pipe scale disturbance that could dislodge metal particulate. The contractors should also be provided with door hangers or flyers with information about lead risks and the LSL replacement program with contact numbers so that the contractors are not acting as the spokesperson for LWS if the homeowners have questions, comments, or complaints.

After an LSL is replaced, there are additional steps to ensure that a customer's water quality is not compromised (LSLR Collaborative, 2019). These steps involve whole house flushing. LWS will need to determine if the contractors replacing the LSLs will be responsible for this task or if there will be a separate crew dedicated to whole house flushing. Whole house flushing is critical to remove particulate that enters a customer's home after the new service line is installed and the water service is turned back on. The flushing process involves removing aerators from faucets throughout the house to allow the particulate to pass though the lines. A water heater, water softener, or filtration device (either for the entire house or at specific faucets) should be bypassed during the flush so that metal particulate does not collect in these devices.

The flushing begins by fully opening the hose bib (typically on the outside of the house or in the basement at the point of connection) and allowing water to flow continuously throughout the flush. Then the faucets throughout the house are opened one by one starting on the lowest level (i.e., basement if available) and then moving up a level until all the faucets with a drain are open. This flushing involves turning on faucets in laundry rooms, bathtubs, showers, sinks, etc. Once all the faucets are open, they are left on for 30 minutes to allow any released particulate to find its way out of the premise plumbing. The faucets are closed in the opposite order that they were turned on, meaning that the top floor faucets are turned off first. Additional water quality sampling should be collected to quantify the concentration of metals. If lead levels are elevated, then additional flushing might be necessary along with follow up monitoring. Filters and extra filter cartridges should be supplied when elevated levels of lead are detected after the removal of an LSL and could be used as a standard practice with three months of replacement filters for all LSL replacements.

7.3 Nitrification Water Quality Monitoring

7.3.1 TCR Monitoring Sites

The TCR monitoring sites are a set of approximately 120 locations, which are sampled every two to four weeks for the following water quality parameters:

- Total chlorine residual.
- Monochloramine residual.
- Free ammonia.
- Total coliform and e-coli.
- Nitrite.

7.3.2 Distribution Monitoring Sites

The distribution monitoring sites include 25 locations, which are sampled once a month for the following water quality parameters:

- pH and temperature.
- Total chlorine residual.
- Nitrite.
- Nitrate.
- Total coliform and e-coli.
- Heterotrophic plate counts.
- Conductivity / total dissolved solids.
- Fluoride.
- Turbidity.
- TOC.
- Iron and manganese.
- Hardness and alkalinity
- Metals analysis (ICP-MS)
- Phosphate

In February 2019, LWS added alkalinity and hardness to the water quality monitoring conducted at the distribution monitoring sites. In September 2019, additional monitoring of free and total ammonia was incorporated at the distribution monitoring sites.

7.3.3 Distribution Water Quality Monitoring Map

Figure 7-9 provides a map of all the distribution system water quality monitoring sites. All of the water quality monitoring sites are designated with alphanumeric codes, where the TCR monitoring sites lead with a number (e.g. 7-6E), and the distribution monitoring sites lead with a letter (e.g. D7). Additional monitoring is conducted at the pump stations, which are labeled according to location (e.g. Belmont).

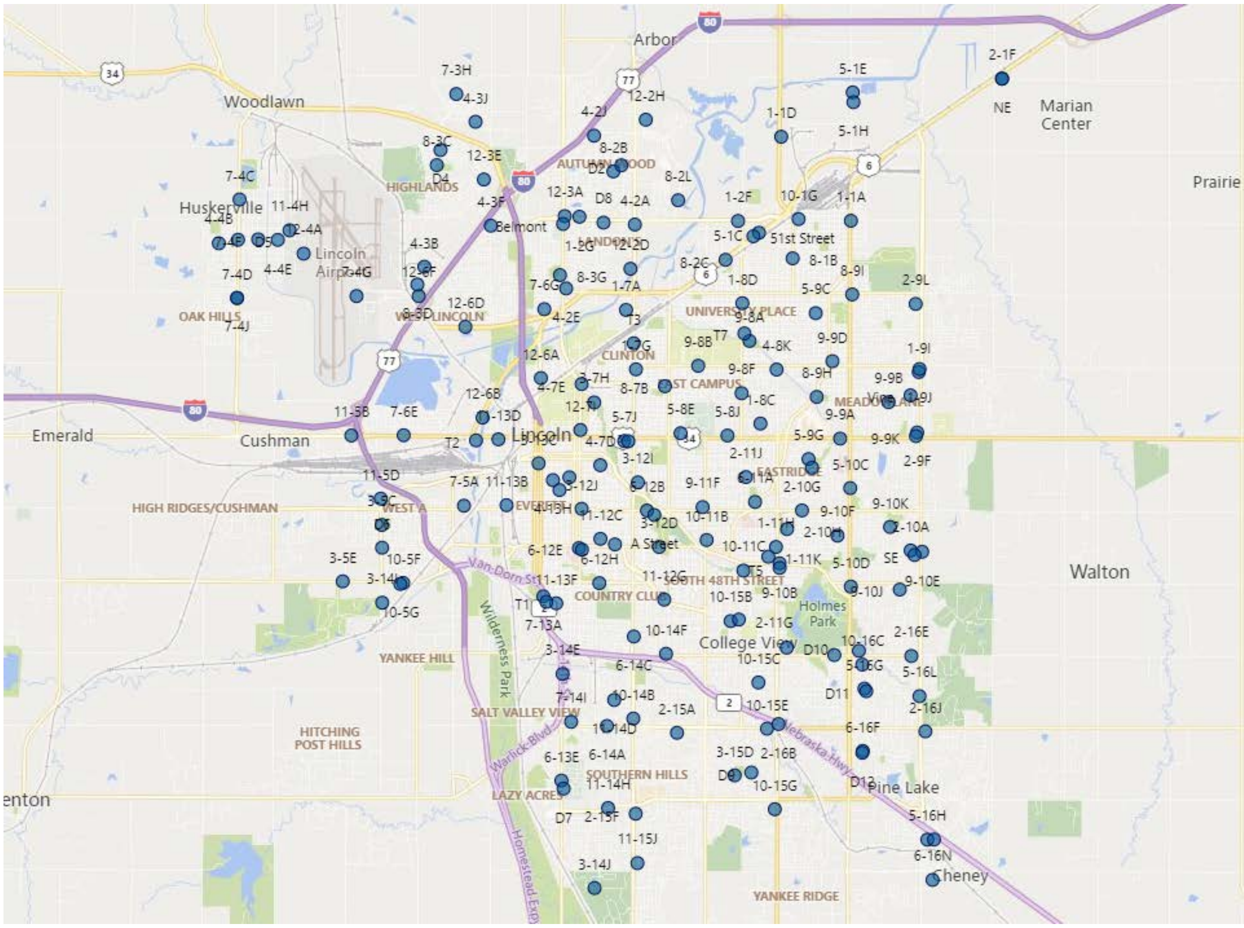


Figure 7-9 Map of Distribution System Water Quality Monitoring Sites

7.4 Nitrification Overview

Nitrification in the distribution system is typically caused by two bacteria groups: ammonia oxidizing bacteria (AOBs) and nitrite oxidizing bacteria (NOBs). AOBs consist of *Nitrosomonas* bacteria, which utilize ammonia (NH_3) as a substrate, converting the NH_3 to nitrite (NO_2). As the total chlorine residual decays, free ammonia becomes available to microorganisms in the distribution system allowing for this process to occur. Similarly, NOBs consist of *Nitrospina* and *Nitrobacter*, which utilize nitrite as a substrate to produce nitrate (NO_3). The rate of nitrification can slow down if either substrate or product concentration becomes too high or too low.

American Water Works Research Foundation (AwwaRF) Report No. 900669 – Nitrification Occurrence and Control in Chloraminated Water Systems identifies significant levels of nitrification as occurring when an increase in nitrite concentration of 50 μg/L or greater is observed. However, initial signs of nitrification may be observed earlier due to loss of total chlorine residual and smaller incremental changes in nitrite (i.e. increase in nitrite of 20 µg/L).

Other conditions, such as temperature, pH and disinfectant residual, can impact the extent to which nitrification may occur. Nitrification is more prevalent when water temperature ranges from 25°C to 30°C but can occur at temperatures as low as 15°C. Additionally, total chlorine residuals of less than 1.5 mg/L can support the growth of nitrifying bacteria, so maintaining a chlorine residual in the range of 2.5 mg/L or higher is generally recommended for controlling nitrification. Additionally, most bacteria groups are sensitive to high or low pH conditions. Previous studies have observed that high pH conditions generally deter biological growth.

Nitrification is typically characterized by:

- Reduction in total chlorine residual
- Decrease in free/total ammonia
- Increase in nitrite and/or nitrate
- Increase in HPCs

Indicators of nitrification may also include reduction in alkalinity, dissolved oxygen and pH.

7.5 Nitrification Occurrence

Distribution system water quality data collected from 2014 through 2018 demonstrates a consistent pattern of chlorine residual decay with corresponding increases in nitrite concentration occurring between August and December of each year. This timeframe overlaps with relatively warm water temperatures of 20°C to 25°C in water supplied by horizontal collector wells and 18°C to 23°C in water supplied by the vertical wells. With water demands dropping in late summer/early fall, the increased water age in the distribution system and elevated water temperatures provide an environment conducive for bacterial regrowth and nitrification. As climate change continues to impact ambient air temperatures, it can be expected that over time, the water temperature will rise as well, likely at a slower rate and lag relative to the rise in ambient air temperature creating more challenging conditions for nitrification control.

Figure 7-11 shows the total chlorine residual in the East and West WTP finished water, Belmont Pump Station, D2 and D5 monitoring sites located within the Belmont Service Level from January 2015 to January 2019. The figure demonstrates a trend of chlorine residual decay between the months of August and December, which is highlighted by the gray bands. Figure 7-10 identifies the D2, D5 and Belmont monitoring sites on a map for context.

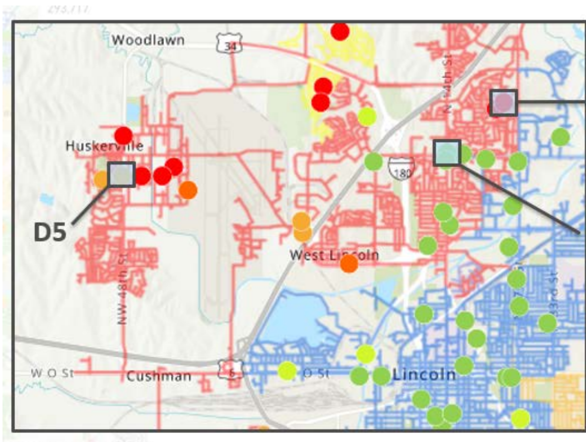


Figure 7-10 Locations of D2, D5 and Belmont Water Quality Monitoring Sites

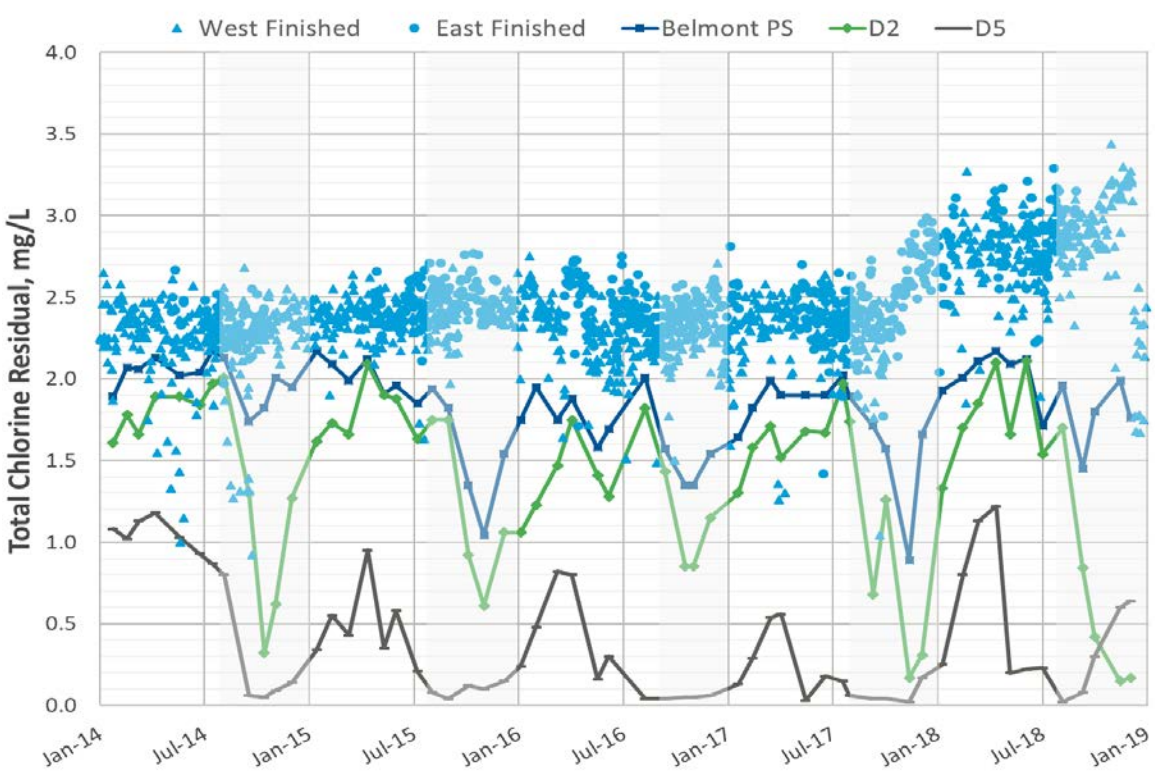


Figure 7-11 Total Chlorine Residual at Distribution System Monitoring Sites in the Belmont Service Level from January 2014 to January 2019

Figure 7-12 shows the resulting increase in nitrite concentration at the same monitoring sites, as excess ammonia from chloramine residual decay is converted into nitrite by AOBs. While major spikes in nitrite concentration typically occur between September and January, increases in the nitrite concentration exceed 50 µg/L at the D2 monitoring site nearly year-round.

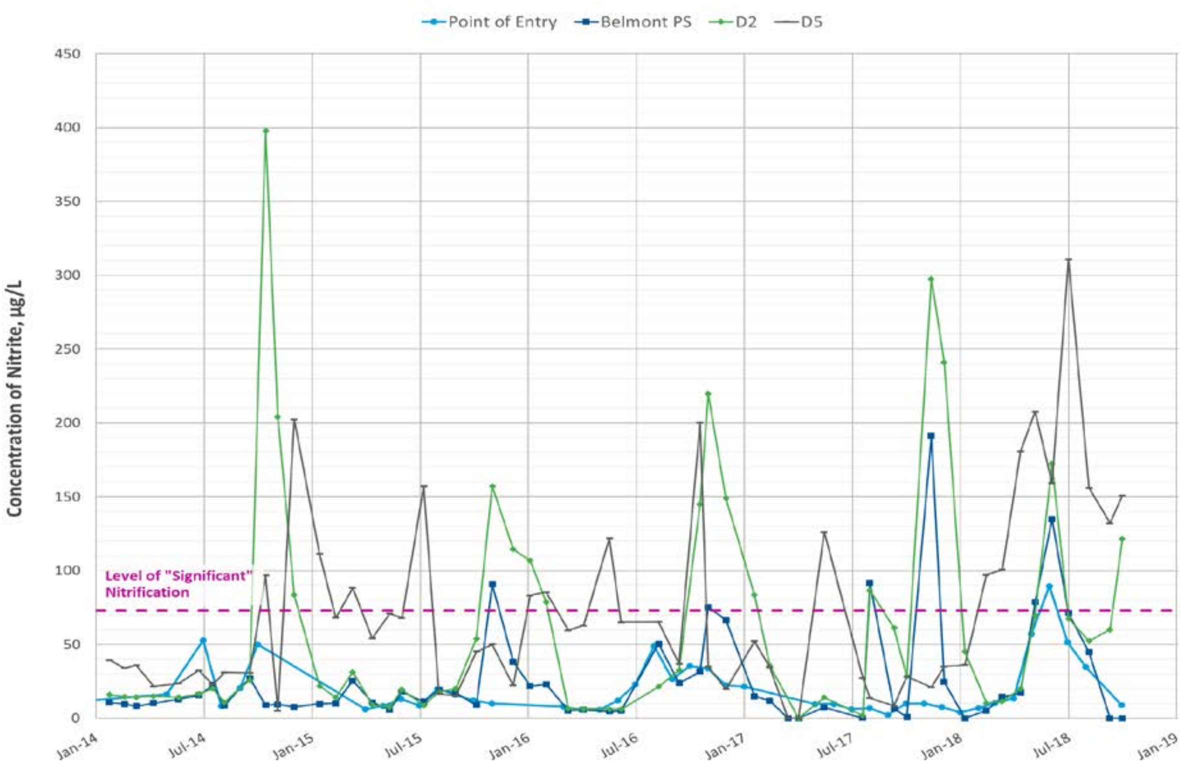


Figure 7-12 Nitrite Concentration at Distribution System Monitoring Sites in the Belmont Service Level from January 2014 to January 2019

The impacts of nitrification have historically been more significant in areas with high water age, such as Air Park, Northwest, Cheney, and Pioneers. Figure 7-13 and Figure 7-14 provide heat maps of the average monthly chlorine residuals monitored throughout the distribution system from April to November of 2016 and 2017, respectively.

From 2014 to 2017, the total chlorine residual leaving the WTP was maintained at approximately 2.5 mg/L as Cl_2. However, the chlorine residual tends to degrade as water moves south and west across the distribution system. From February to July, the total chlorine residual in most of the Low and High Service Levels remained above 1.5 mg/L as Cl_2. However, during the peak nitrification season between September and December, chlorine residuals dropped to less than 0.25 mg/L as Cl_2 in 20 percent of the distribution system monitoring sites in 2016 and 26 percent of the distribution system monitoring sites in 2017. Nitrification was at its peak in November 2017, with chlorine residuals of less than 0.25 mg/L as Cl_2 in as much as 36.7 percent of the distribution system monitoring sites.

As noted previously, it is desirable to maintain chlorine residuals greater than 2.5 mg/L as Cl_2 since values less than 1.5 mg/L can support the growth of nitrifying bacteria. Between the months of September and December 2017, the total chlorine residual was greater 1.5 mg/L as Cl_2 in only 18 percent of the distribution system monitoring sites.

While the impacts of nitrification are more significant between the months of September and December, the Belmont, Northwest, and Cheney Service Levels experience significant degradation of chlorine residual year-round due to the long water age and relatively low demands. Specifically, in Air Park, Industrial Zone and the immediately surrounding areas, the total chlorine residual is typically below 0.5 mg/L as Cl_2 year-round, and during the nitrification season the chlorine residual is less than 0.25 mg/L as Cl_2.

Figure 7-15 and Figure 7-16 provide heat maps of the average monthly total chlorine between the peak nitrification seasons from September to December in 2016 and 2017, respectively. Figure 7-17 and Figure 7-18 provide heat maps for nitrite and HPCs over the same timeframe in 2017.

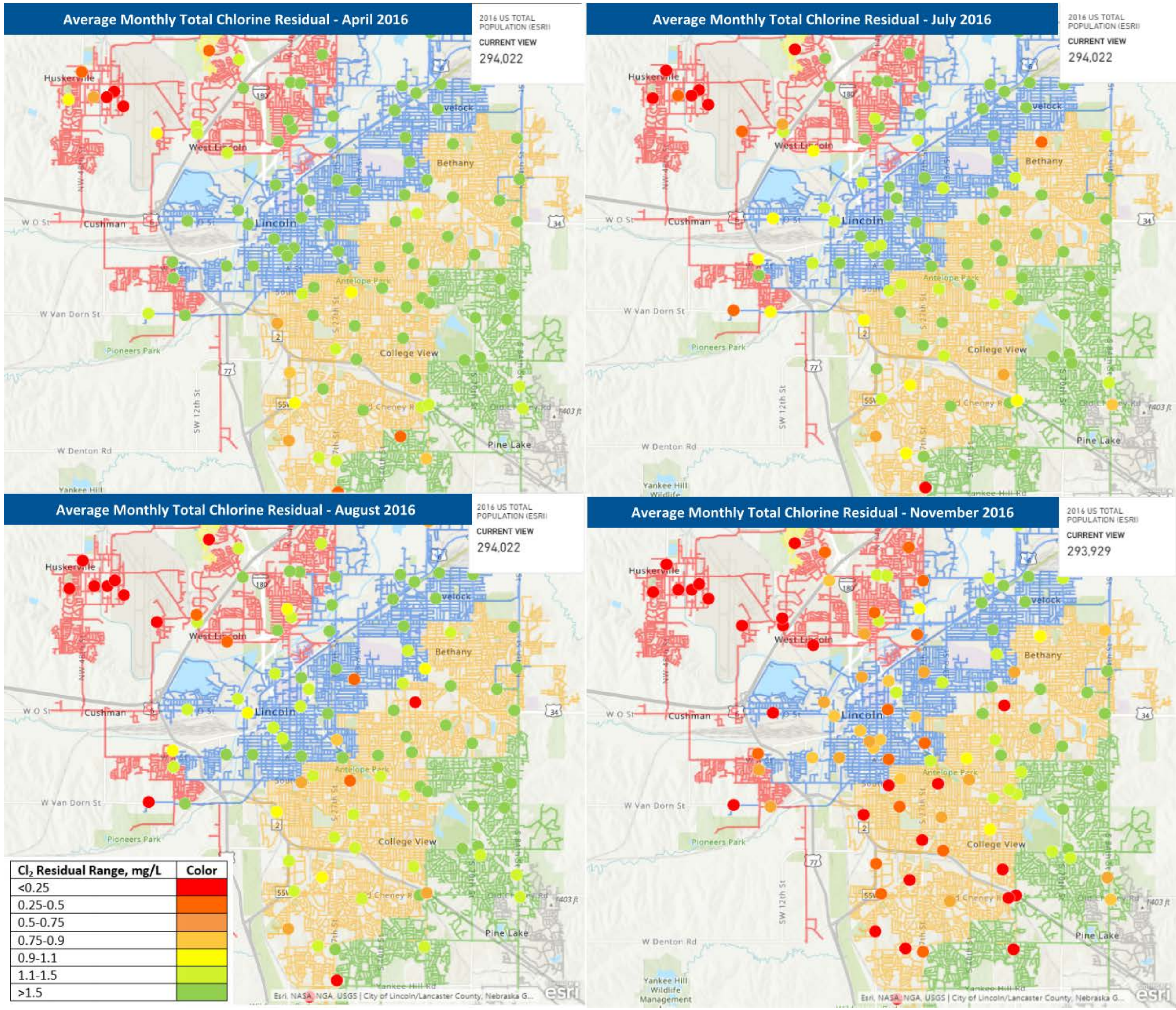


Figure 7-13 Total Chlorine Residual Heat Map from April to November 2016

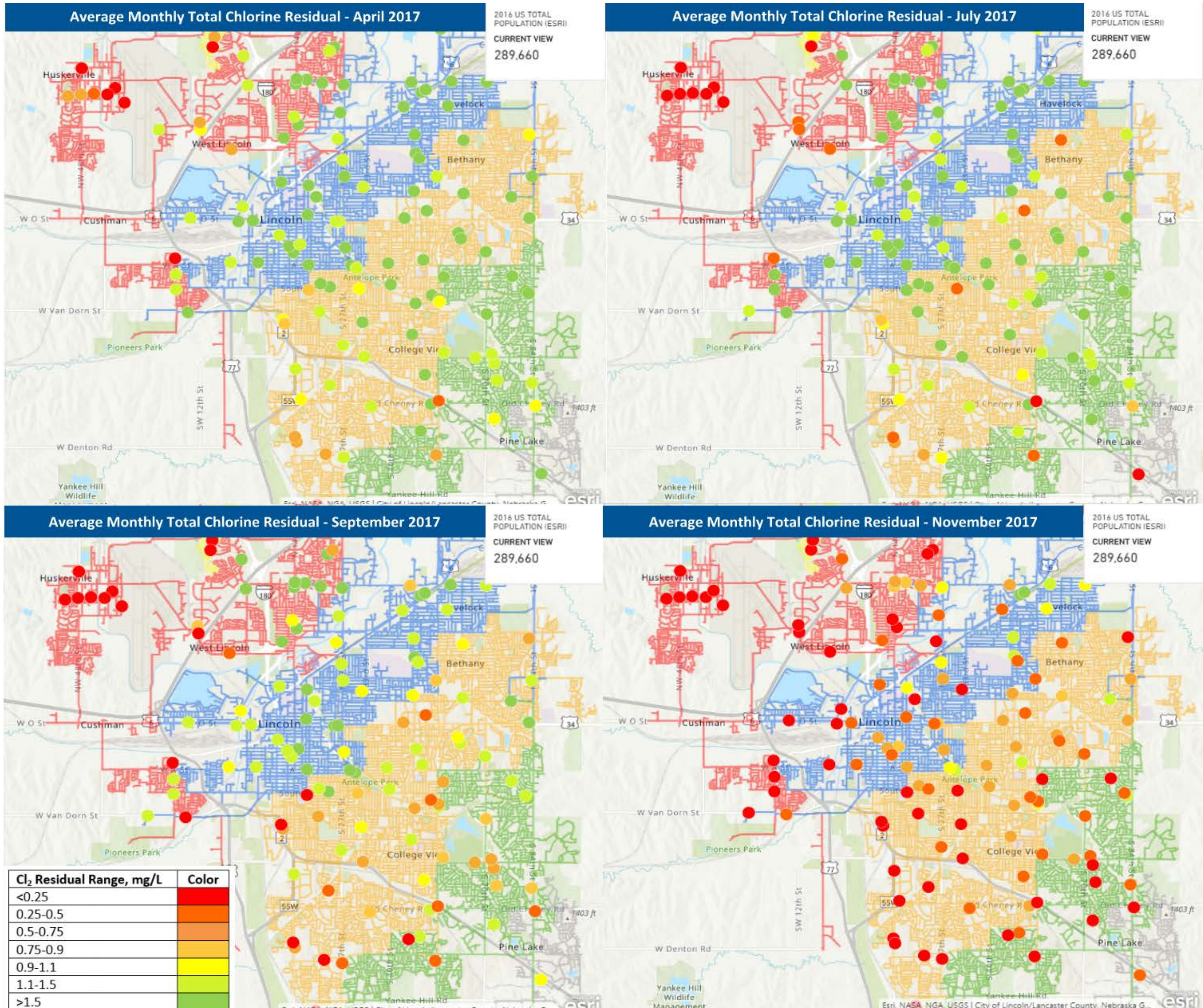


Figure 7-14 **Total Chlorine Residual Heat Map from April to November 2017**

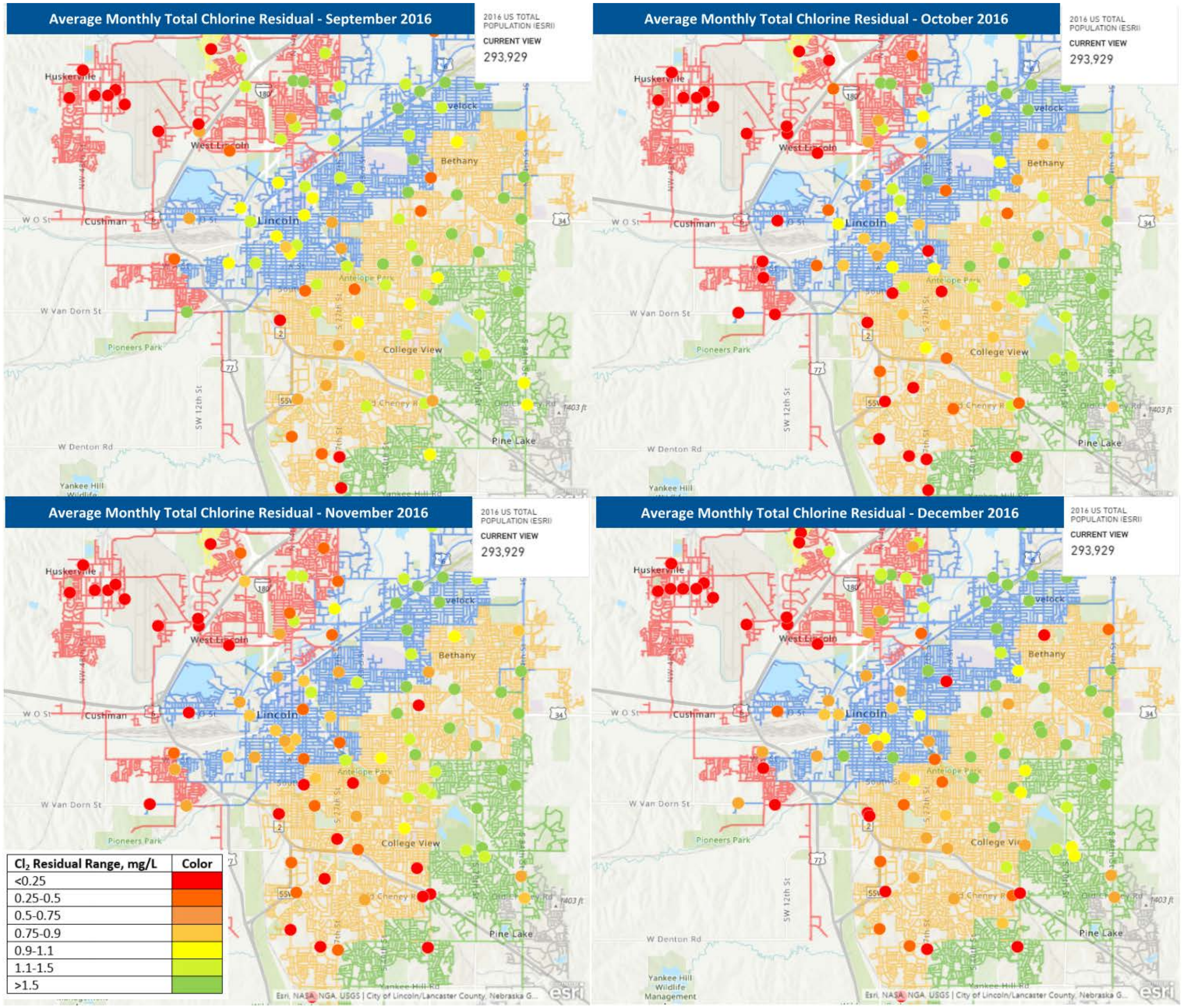


Figure 7-15 **Total Chlorine Residual Heat Map During Peak Nitrification Season from September to December 2016**

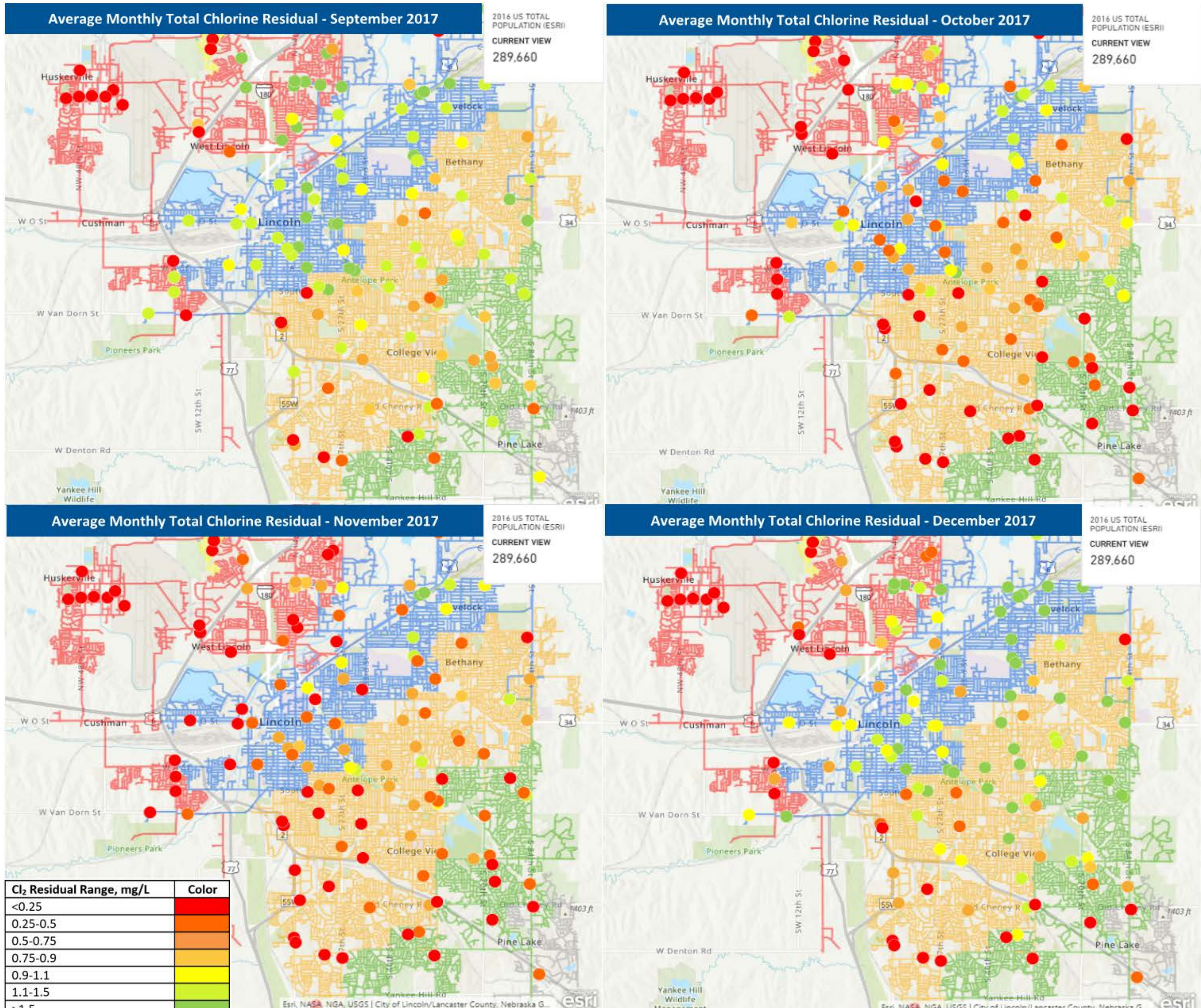


Figure 7-16 Total Chlorine Residual Heat Map During Peak Nitrification Season from September to December 2017

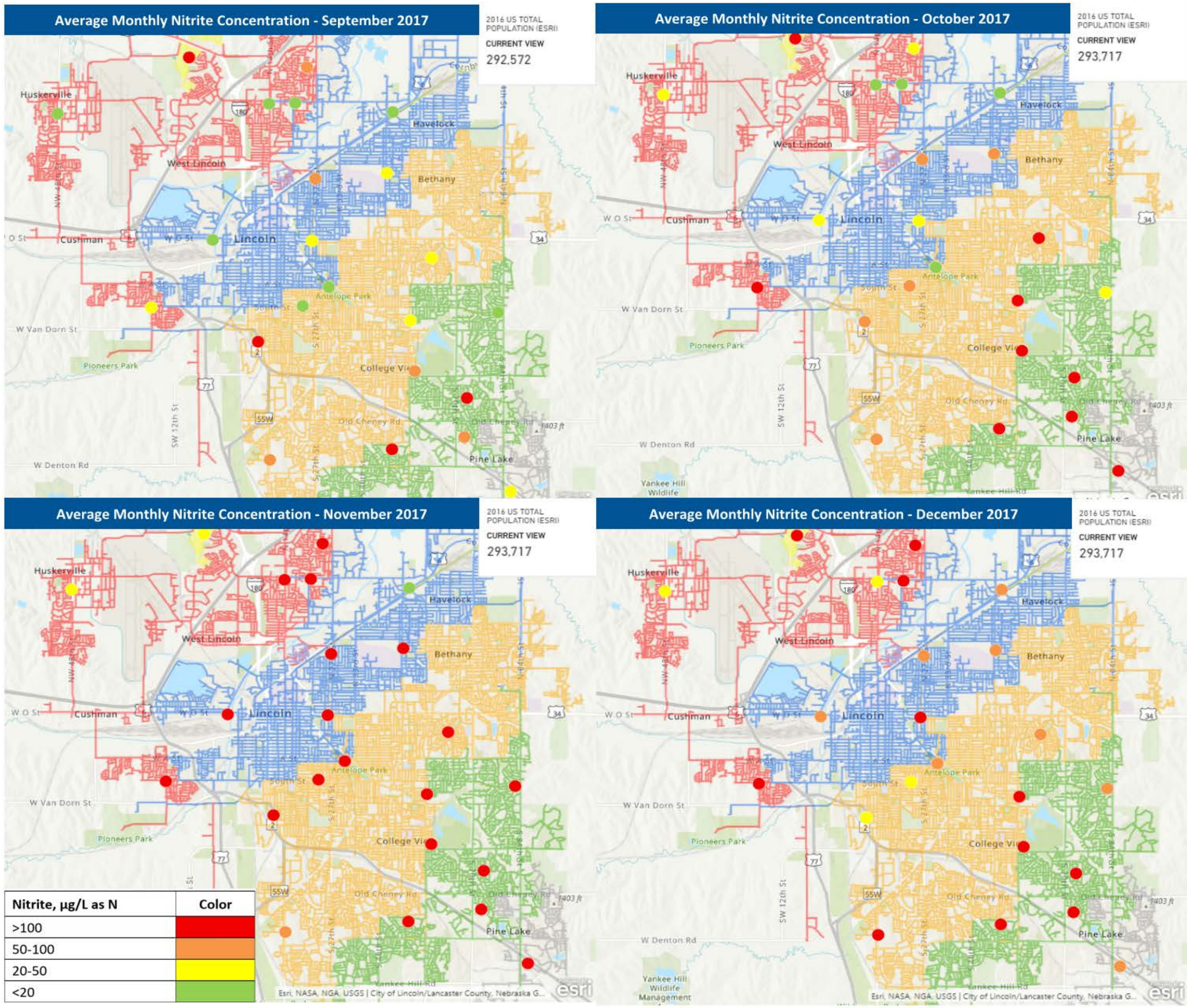


Figure 7-17 **Heat Map Illustrating the Change in Nitrite Concentration Throughout the Distribution System During Peak Nitrification Season from September to December 2017**

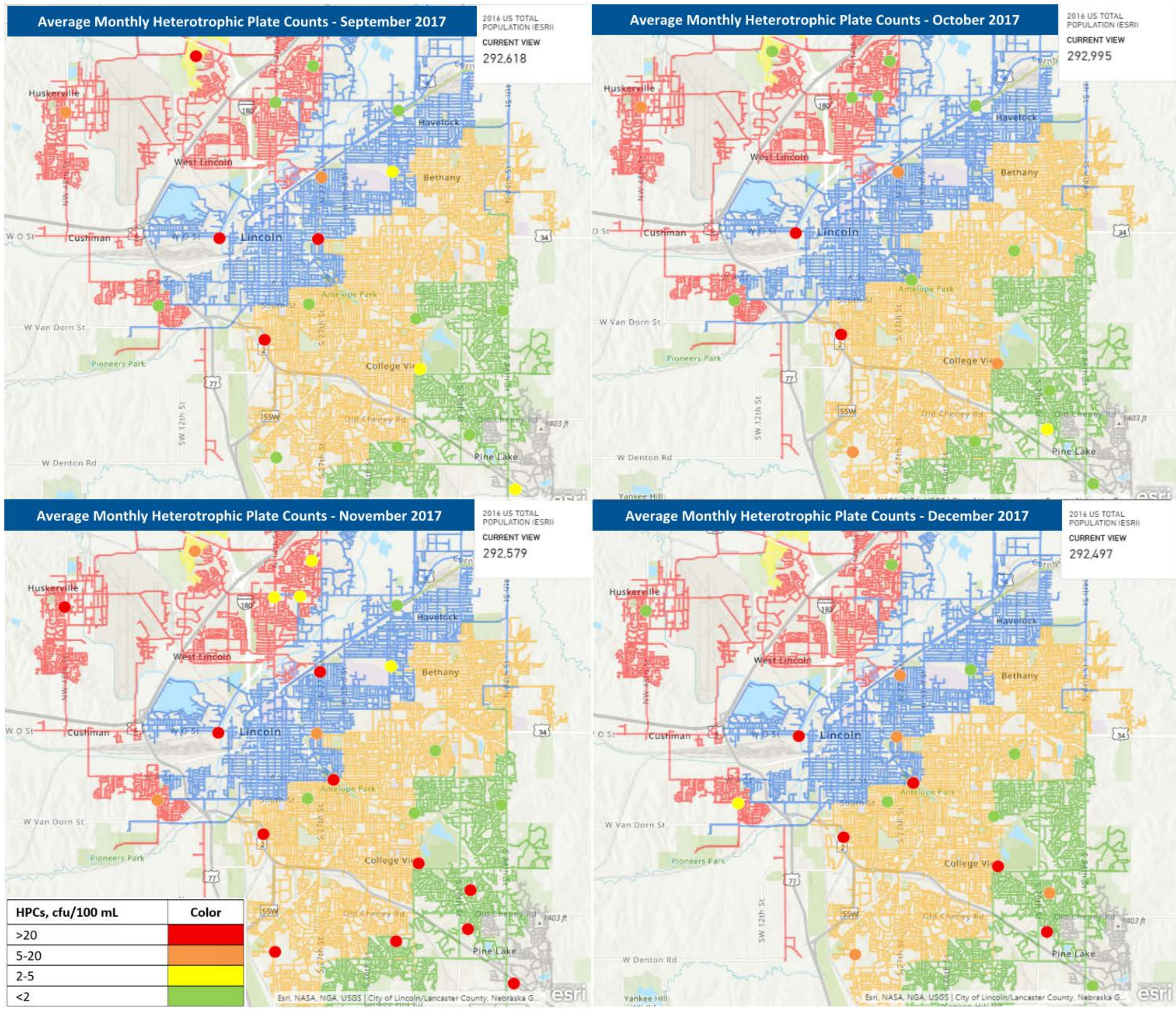


Figure 7-18 Heat Map Illustrating the Change in Heterotrophic Plate Counts Throughout the Distribution System During Peak Nitrification Season from September to December 2017

7.5.1 Nitrification Management (2018 to present)

In Year 2018, LWS implemented operational changes to control nitrification through management of delivered water quality and water age. Winter operations that contributed to nitrification control included the following measures:

- Increase total chlorine residual beginning in December 2017.
- Take East Plant out of service in the month of September (reduced water temperature, TOC and AOC).
- Isolate reservoirs (Vine Street, Air Park) for maintenance activities (reduced water age).
- Deep cycling of above ground storage reservoirs (improved turnover and reduces potential for stagnant water).
- Reduce operating volumes in below ground storage reservoirs (reduced water age).

These operational changes considerably improved the widespread impacts of nitrification that were observed in Year 2017. Figure 7-19 provides a heat map of the total chlorine residual measurements during the normal peak nitrification season (September to December). As a result of the measures taken by LWS, the City had significantly better control of total chlorine residual throughout the distribution system. During the peak nitrification season, chlorine residuals were less than 0.25 mg/L as Cl_2 in 14.5 percent of the distribution system monitoring sites, and the total chlorine residual exceeded 1.5 mg/L as Cl_2 in nearly 50 percent of the distribution system monitoring sites. Both of these parameters indicate major improvements to nitrification control relative to previous years.

While these operational changes have resulted in considerable improvements to distribution system water quality, taking the East Plant out of service is not a viable long-term strategy for nitrification control. Further investigation should be conducted to determine the direct impacts of taking the East Plant out of service and evaluate whether treatment modifications are required to continue utilizing the East Plant during peak nitrification seasons. Treatment modifications could include increased chloramine residual, biological filtration and/or application of sodium chlorite.

Distribution water quality improvements are shown in Figure 7-20, which summarizes the average monthly concentration of nitrite at monitoring sites throughout the distribution system from August to December 2018 (note that data in the month of November was not available). Particularly, in the months of October and December there is a notable reduction in the concentration of nitrite from Year 2017 to 2018, indicating that the nitrification control measures taken by LWS were effective.

However, while chlorine residual management and nitrification control improved significantly in the Low and High Service Levels, the areas surrounding Air Park, Northwest, Cheney, and southern parts of Southeast still had difficulty maintaining chlorine residuals greater than 0.5 mg/L between the months of October and December. Therefore, recommendations for distribution system water quality improvement will focus on these areas.

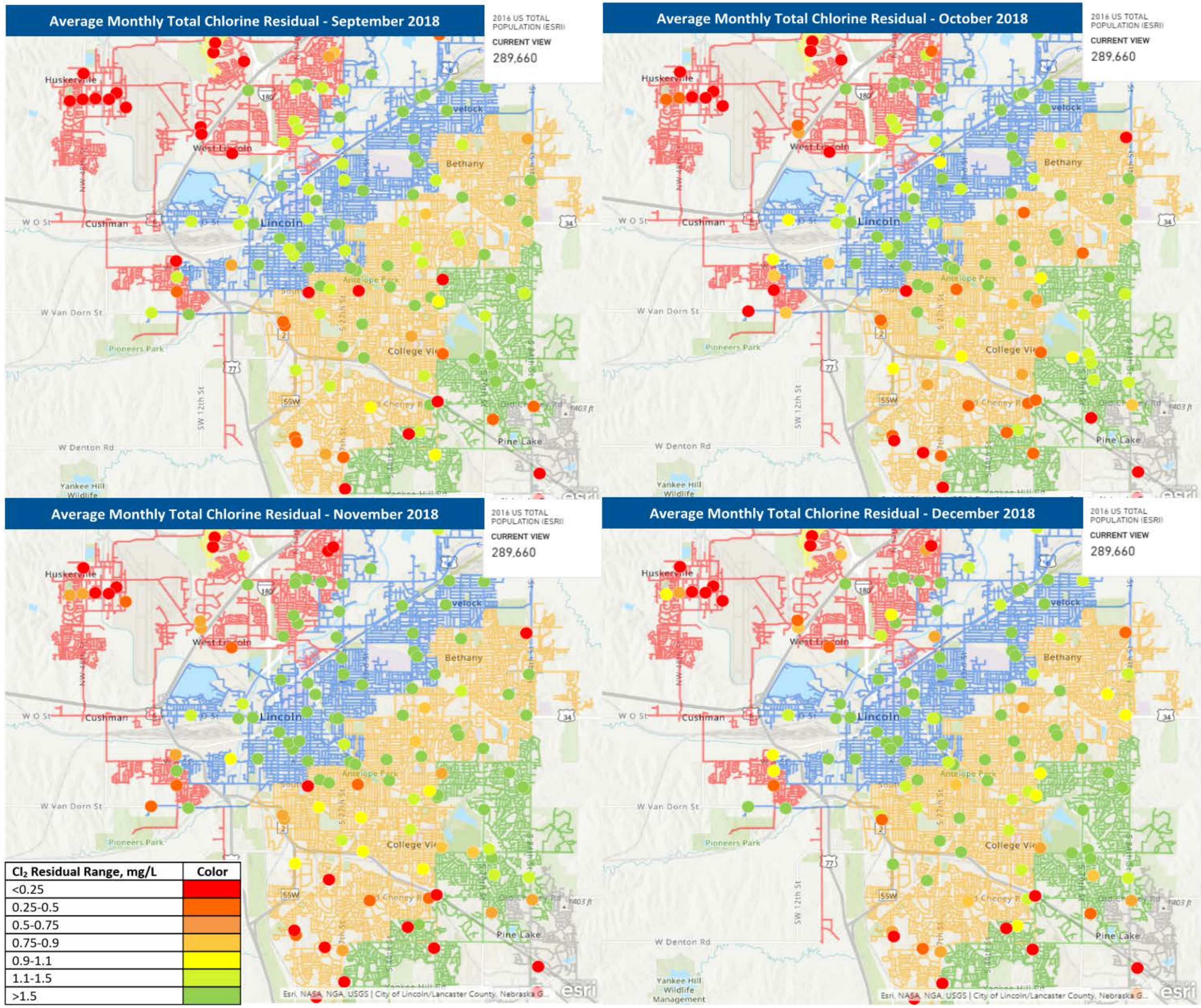


Figure 7-19 Total Chlorine Residual Heat Map During Peak Nitrification Season from September to December 2018, demonstrating effectiveness of nitrification control measures

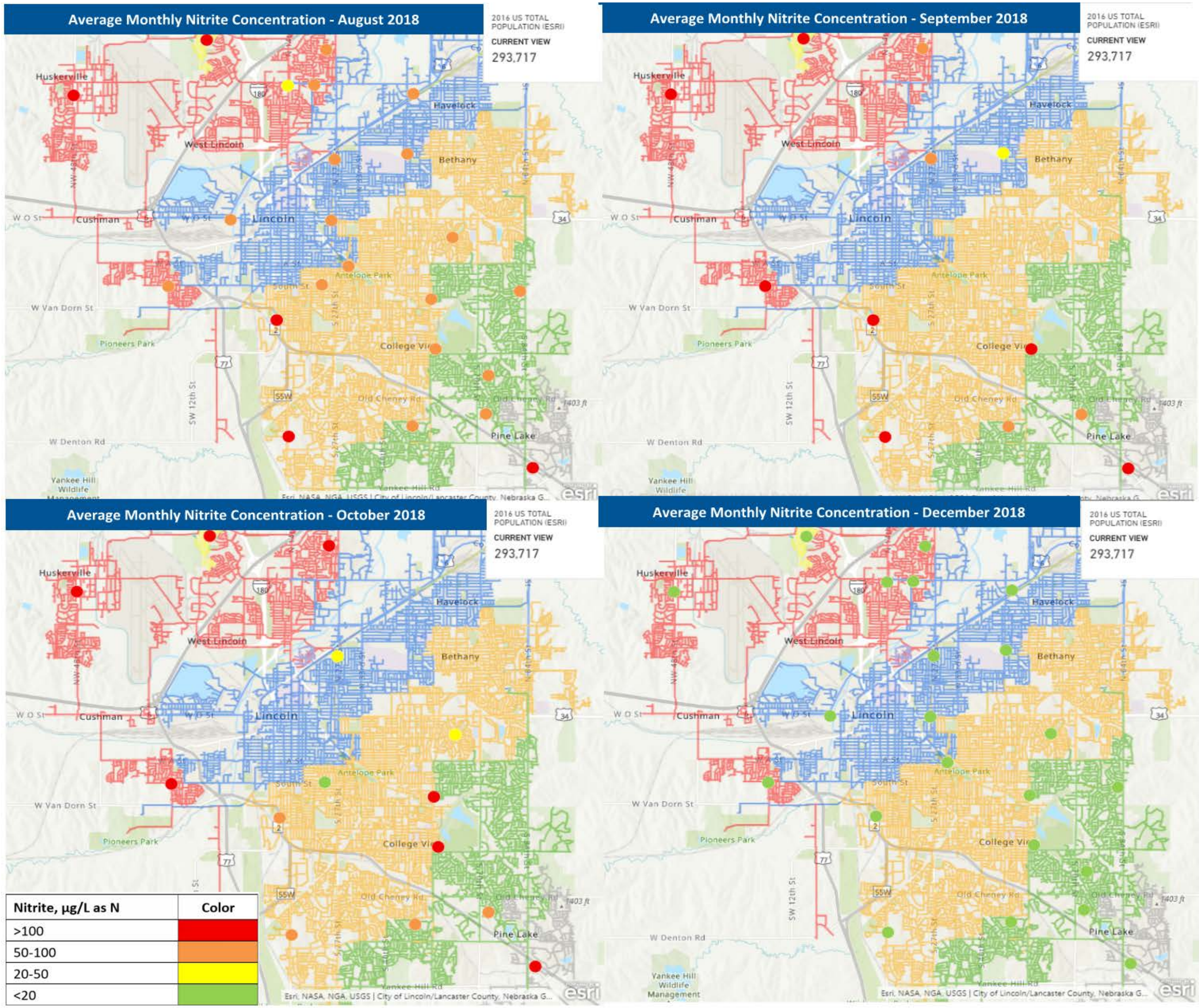


Figure 7-20 Heat Map Illustrating the Change in Nitrite Concentration Throughout the Distribution System from August to December 2018

7.6 Impacts of Distribution System Pipe Materials

Studies have shown that the type of pipe materials used in the distribution system have varying degrees of impact on the degradation of water quality. AWWARF Report No. 90950 – Influence of Distribution System Infrastructure on Bacterial Regrowth focuses on the relationship between pipe materials and biofilm development. This report confirmed previous research and field studies, which found that iron-based pipe materials - such as cast iron and ductile iron - have a higher probability of biofilm development and tend to form denser biofilm than cement, epoxy, or polyvinyl chloride (PVC) piping. Of all the materials evaluated in this study, PVC consistently had lower rates of biofilm development, resulting in lower HPCs in the biofilm and water passing through the piping.

The LWS distribution pipe materials include lined and unlined cast iron pipe, ductile iron pipe, prestressed concrete cylinder pipe (PCCP), and PVC pipe. Figure 7-21 provides a map of the overall distribution system infrastructure, categorized by year of installation (left) and pipe material (right). Color designations were assigned to each pipe material and installation timeframe based on its propensity for degradation of water quality. The maps demonstrate that there is a high proportion of cast iron pipe in the High Service Level, as well as in the areas surrounding Air Park within the Belmont Service Level. These parts of the distribution system may be subject to higher rates of biofilm development, which may result in faster degradation of chlorine residual and elevated levels of HPCs. To evaluate the relative impacts of pipe material on biofilm development, sampling could be performed to compare HPCs in biofilm and sample water for areas with different pipe materials located in the same Service Level. Recommendations for this field study are provided in Chapter 8.

Much of the cast iron pipe in the distribution system appears to have been installed in the early to mid-1900's. With LWS's ongoing distribution infrastructure repair and rehabilitation program, many of these aging pipes are in the process of being replaced or relined. Given the influence of pipe material on water quality, it is recommended that LWS continue with their existing repair and rehabilitation program, prioritizing the replacement of cast iron pipes with alternative materials (PVC or ductile iron) and lining/relining cast iron pipes as needed.

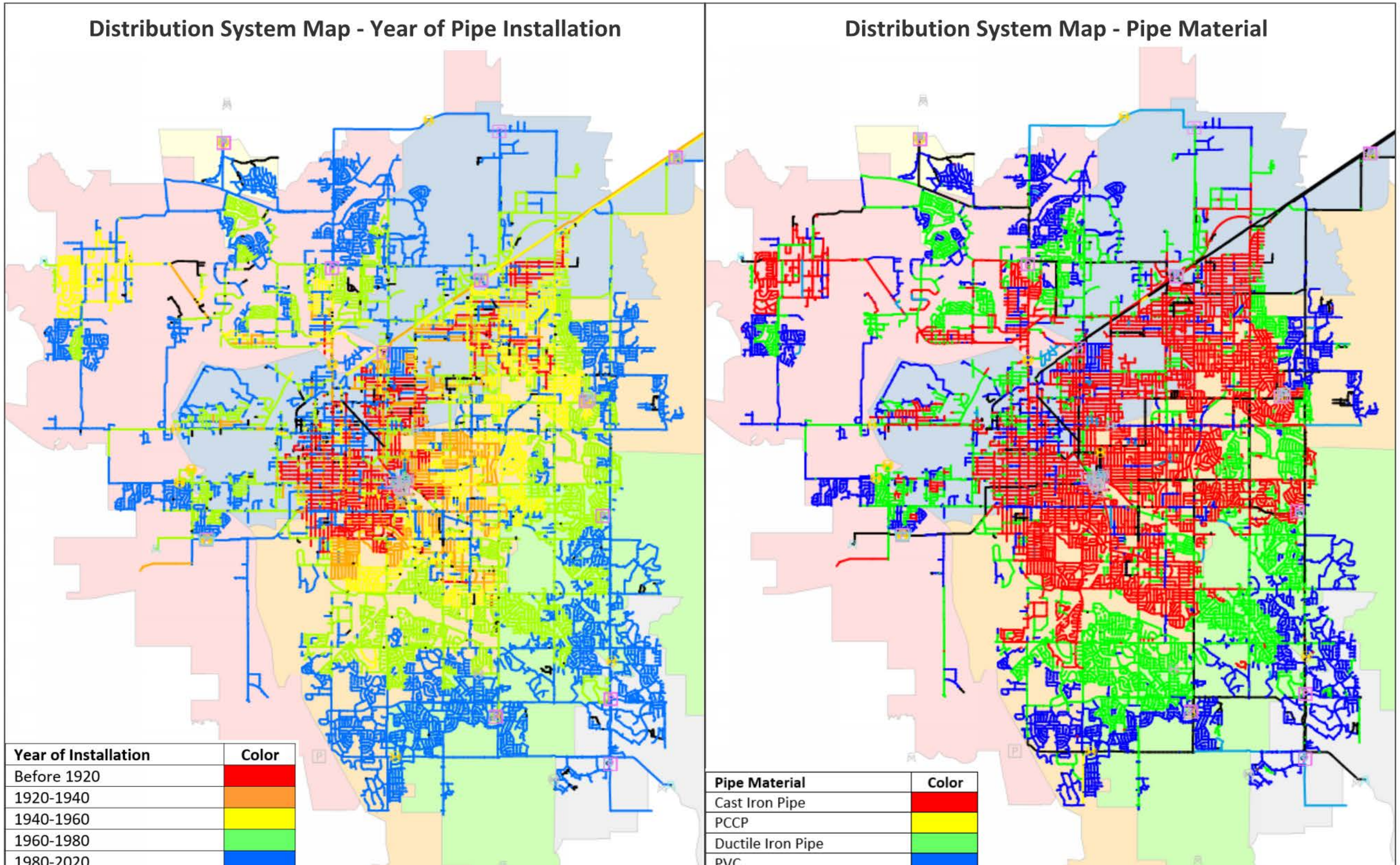


Figure 7-21 Map Identifying Distribution System Infrastructure by Year of Installation and Pipe Material

7.7 Distribution System Water Age Modeling

Distribution system water age modeling was performed with the goal of determining empirical relationships between water age and water quality characteristics. Water age modeling is often used in the industry as a surrogate for constituent modeling to evaluate degradation of water quality as water moves through the distribution system. True constituent modeling, for whichever parameters are desired, is possible in the InfoWater software but requires a high-confidence, high level-of-effort, water quality constituent calibration to develop model parameters for decay/formation potentials, bulk decay coefficients in all storage facilities and of the source water and pipe-wall decay coefficients for all model pipes. Because of the large level of effort required to perform a constituent-calibrated model for water quality analysis, water age modeling evaluations are often used as a surrogate. However, water age modeling alone should not be used to form definite conclusions about water quality without first comparing and establishing relationships to the observed data.

Two base year water age modeling scenarios were developed for Year 2020, based on average day demands and winter operations. This condition represents the system characteristics and demands during the months of October and November, as operations generally shift from Summer Operations to Winter Operations.

7.7.1 Winter Operations

A list of current operations used in the off-peak times of the year, implemented at the beginning of October, were provided by LWS. These operations are followed for energy management and to promote good water quality and include standard operating procedures for pump stations and the cycling of storage facilities, both floating and below-ground. They include the following:

- East Plant taken off-line.
- Airpark (or NW 12th Street) Reservoir isolated from the system.
- Northeast Reservoir placed in series operation.
- Vine St North reservoir isolated from system.
- Increased chlorine residuals leaving treatment plant(s) to 3.4-3.8 mg/L range
- Deep cycling of above ground storage reservoirs.
- Reduced operating volume of below ground storage reservoirs.
- Suspended operations of Southeast Pump Station and only a single pump at Vine Street East should be used to fill Yankee Hill.

During this time-frame, HPP 11 or HPP 12 will be used to pump water directly into the Low Duty system down the 54-inch or 60-inch transmission main. Expected rates will be from a minimum of 12 mgd to 20 mgd. At the higher end of this range a potable water pump can be turned off for additional energy savings as the 54-inch discharge main pressure will be sufficient for plant service water needs.

Using either HPP 1, HPP 2, or HPP 3, water will be pumped down the 48-inch transmission main to Northeast Reservoir (approximately 9 mgd) and the 36-inch transmission main (approximately 4 mgd) to 51st Street Reservoir. It should be ensured that the 36-inch main pressure remains at 30 psi or higher during low flow, winter conditions to reduce cavitation issues on HPP 1, HPP 2, or HPP 3.

Water from the 48-inch main will only enter the East Cell of NE reservoir via NE yard valve No. 30. Valve No. 32 or No. 33 will be used to regulate water coming into the station and Transfer Pump 1 will be used accordingly to move water out of NE. Valves No. 4 and No. 5 on the West Cell of Northeast Reservoir will remain closed. These Northeast valve combinations allow "series" operation of the reservoir which will provide adequate turnover and flow through them. A one pump rule at Northeast will still be in effect with the added condition that only Transfer Pump No. 1 is to be used during these winter pumping operations. No other low duty pumps are to be used unless an emergency exists. Pumps Nos. 2 - 6 will be put into local control during winter operations.

New operating levels and ranges for below ground storage reservoirs that do not provide floating storage were also provided, shown in Table 7-2. The goal is to cycle the levels in these reservoirs from the low alarm to the high alarm to promote turnover unless weather related issues, system facilities out-of-service, or other maintenance related activities dictate the need to fill these reservoirs back to their original high levels.

Table 7-2 Summer/Winter Operations, Ground Storage Alarm Levels

	Summer Operations Alarms				Winter Operations Alarms			
Reservoir	Lo-Lo	Lo	Hi	Hi-Hi	Lo-Lo	Lo	Hi	Hi-Hi
Northeast Reservoir	11	12	15.5	16	5	6	9	10
51st Street Reservoir	9.8	10	13	13.5	4	4.5	7	8
“A” Street Reservoirs 8 and 9	4	6.5	13	13.5	4	4.5	7	8
“A” Street Reservoir 6	6	7	12	13	4	4.5	7	8

Guidelines for floating storage reservoir refill initiation levels are noted as the following:

- Pioneers – 44 feet.
- Yankee Hill – 60 feet.
- Southeast – 47 feet.
- Airpark – 73 feet.
- NW 12th – 54 feet.
- S. 56th Street – 50 feet.
- Cheney – 22 feet.

Winter pump rules for pumping stations are noted as the following:

- Northeast - One pump only, No. 1 Transfer Pump. All low Duty pumps out-of-service.
- 51 St. - One pump only, either one Low Duty or one Booster pump.
- "A" St. Main Complex - One pump only, either one Low Service or one High Service pump.
- "A" St. Satellite No. 9 and No. 10 - Zero pump rule. Pump No. 9 or No. 10 are not to be operated.
- Belmont - One pump only, either Pump No. 1 or No. 2. Pump No. 3 or No. 4 are not to be operated.
- Cheney - One pump Rule. Pumps No. 4 or No. 5 are not to be operated.
- Yankee Hill - One pump Rule.
- Southeast - One pump only, either pump No. 1 or No. 2. Pumps No. 3 or No. 4 are not to be operated.
- Vine St. Main Complex - One pump rule.
- Vine St. East Complex - One pump rule.

7.7.2 Water Age Modeling Scenario No. 1

Using the Winter Operations detailed in the previous section, a 28-day water age modeling scenario was developed to assess water age. For the Water Age scenario No. 1, the Airpark Reservoir was placed out-of-service and all other controls were modeled as described in the Winter Operations section. The results were captured in a dashboard to allow for the ability of rapid filtering by Service Level and display of more than one scenario on-screen for comparison purposes. Figure 7-23 through Figure 7-28 on the following pages show the results by Service Level of the Water Age Scenario No. 1. These figures show the average water age (blended over time) that occurred within the evaluation on the left side of the figure, and the maximum water age (the highest-age plug of water during any time during the scenario) on the right. The maximum water age is experienced when storage is being drawn down in times of equalization. At the bottom of each map figure is a pie-chart which shows a relative percentage of how many model junctions fall within each category. A general legend for all map colors and how they are related to water age is provided below in Figure 7-22 since some of the figures on the following pages do not have junctions within all categories. A quick visual way to determine the overall average age of water within an area is to eyeball where the “50-percent line” would be from the pie-charts. Doing this with the sample on Figure 7-22 shows that the “50-percent” line occurs almost between the 2 to 4 days and 4 to 6 days category, or that the average age would be just over 4 days.

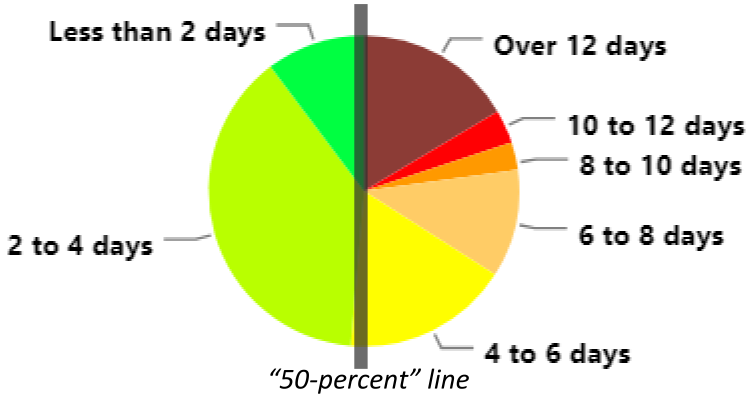


Figure 7-22 Category Legend for Figure 7-23 through Figure 7-28

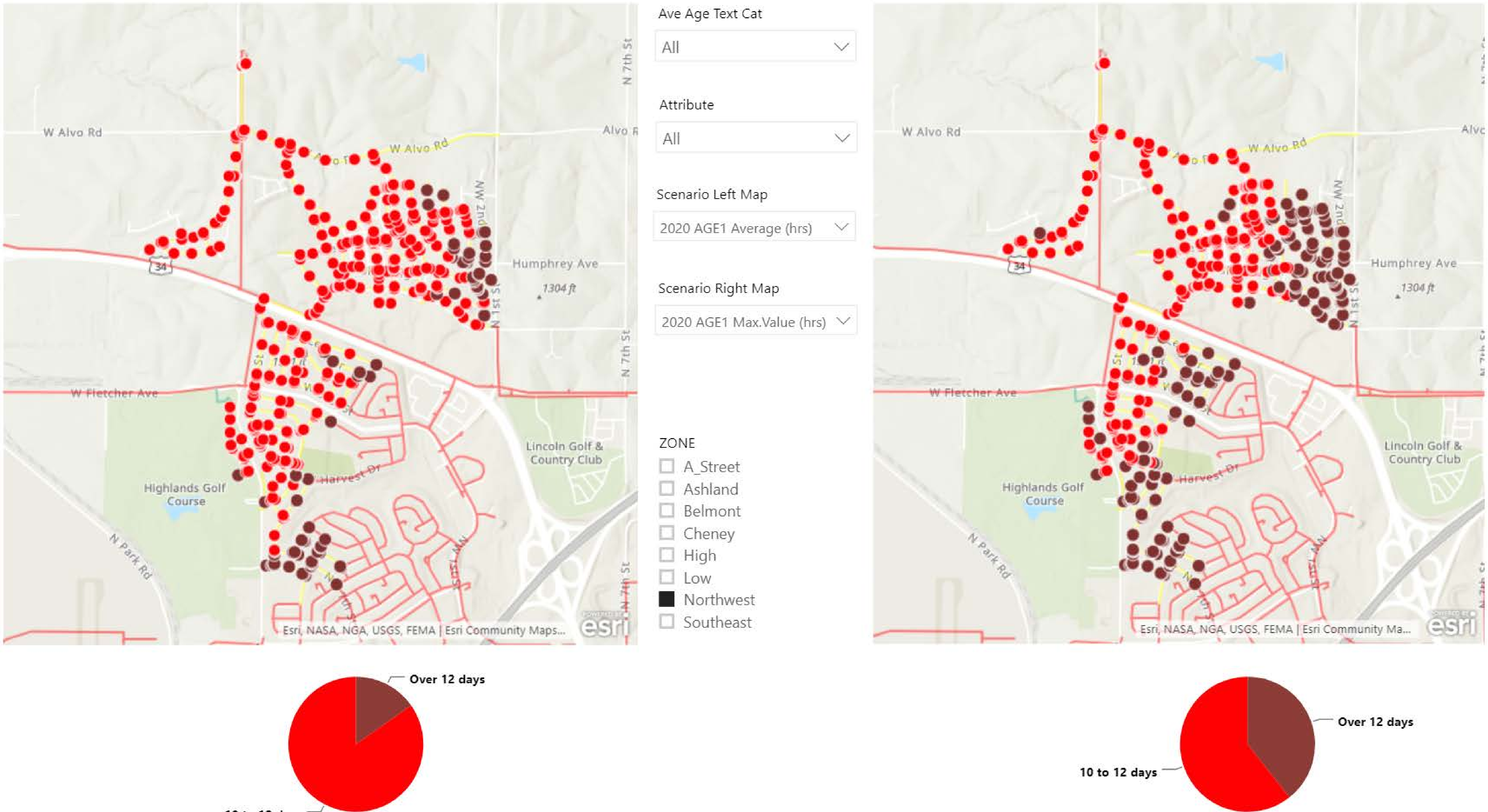


Figure 7-23 Northwest SL Water Age Scenario 1 (Average-Left, Maximum-Right)

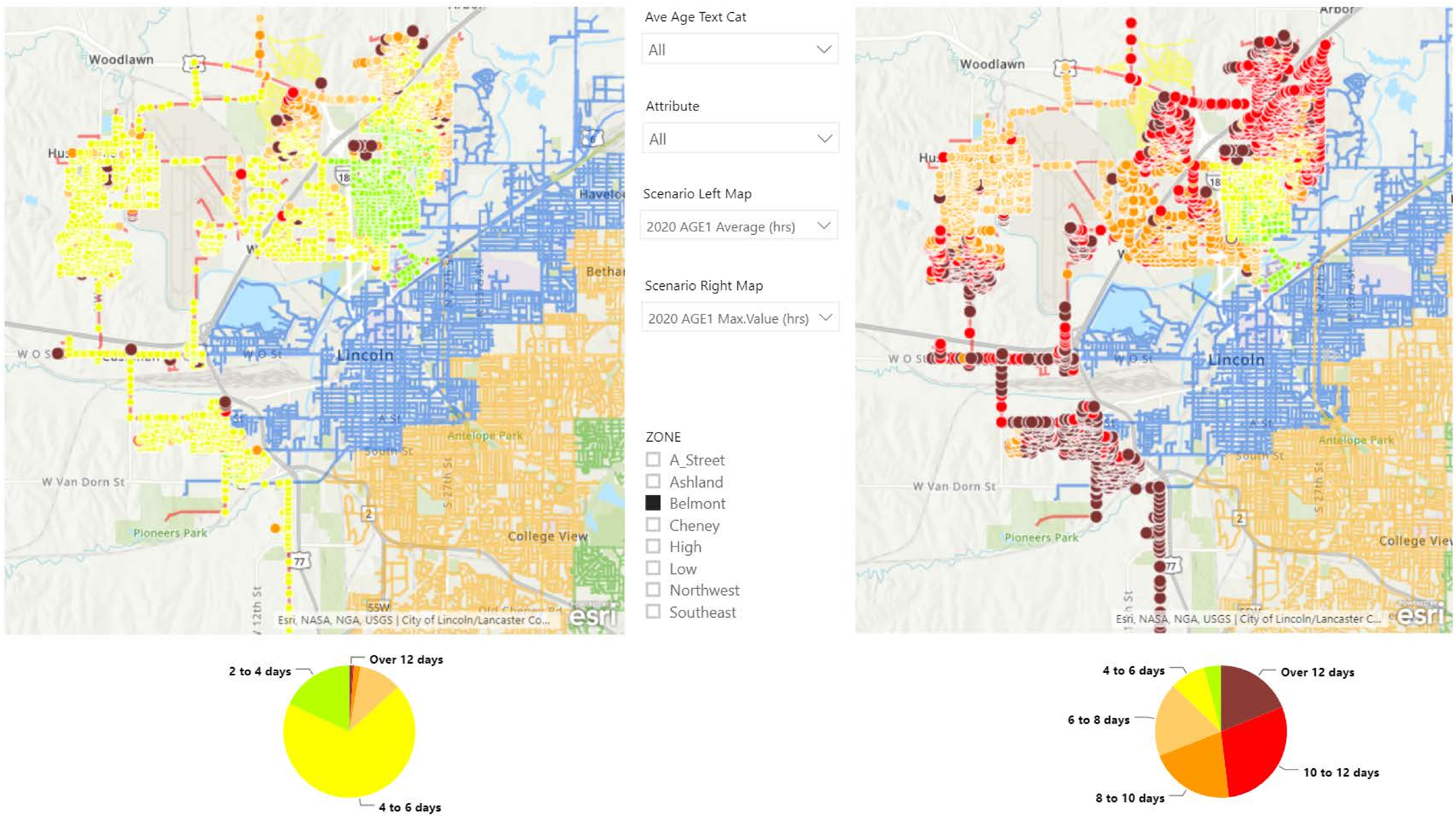


Figure 7-24 Belmont SL Water Age Scenario 1 (Average-Left, Maximum-Right)

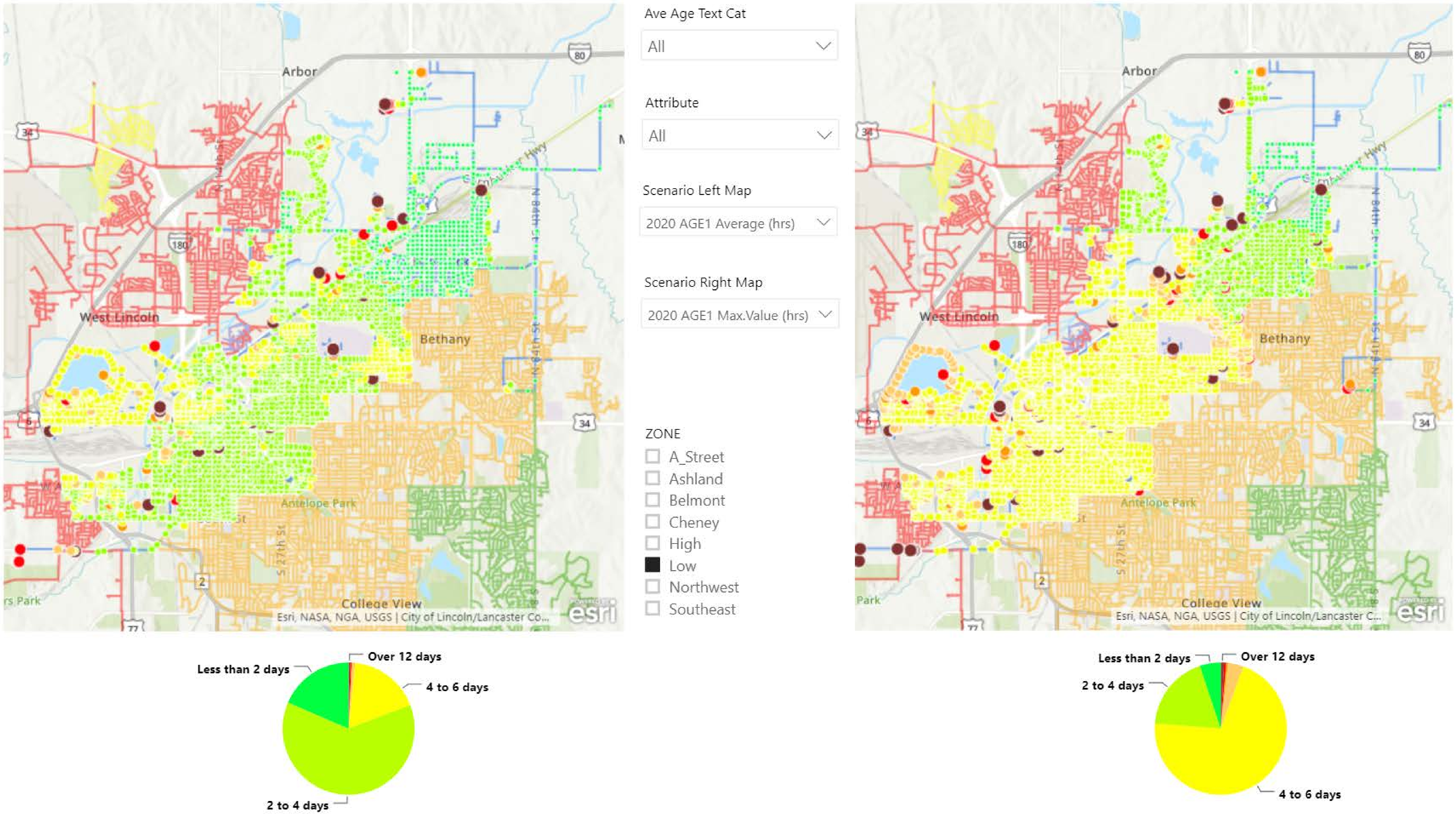


Figure 7-25 **Low SL Water Age Scenario 1 (Average-Left, Maximum-Right)**

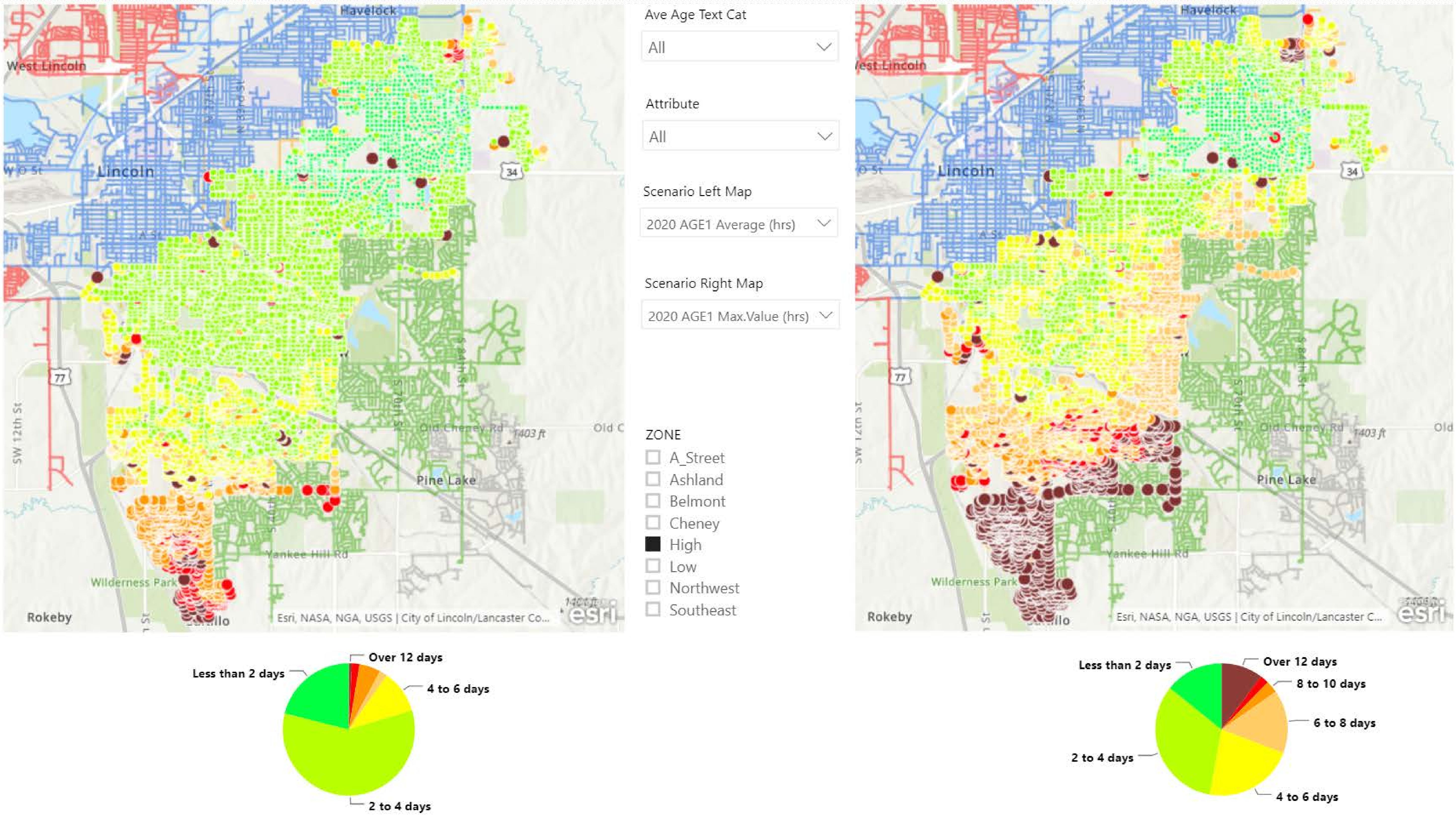


Figure 7-26 **High SL Water Age Scenario 1 (Average-Left, Maximum-Right)**

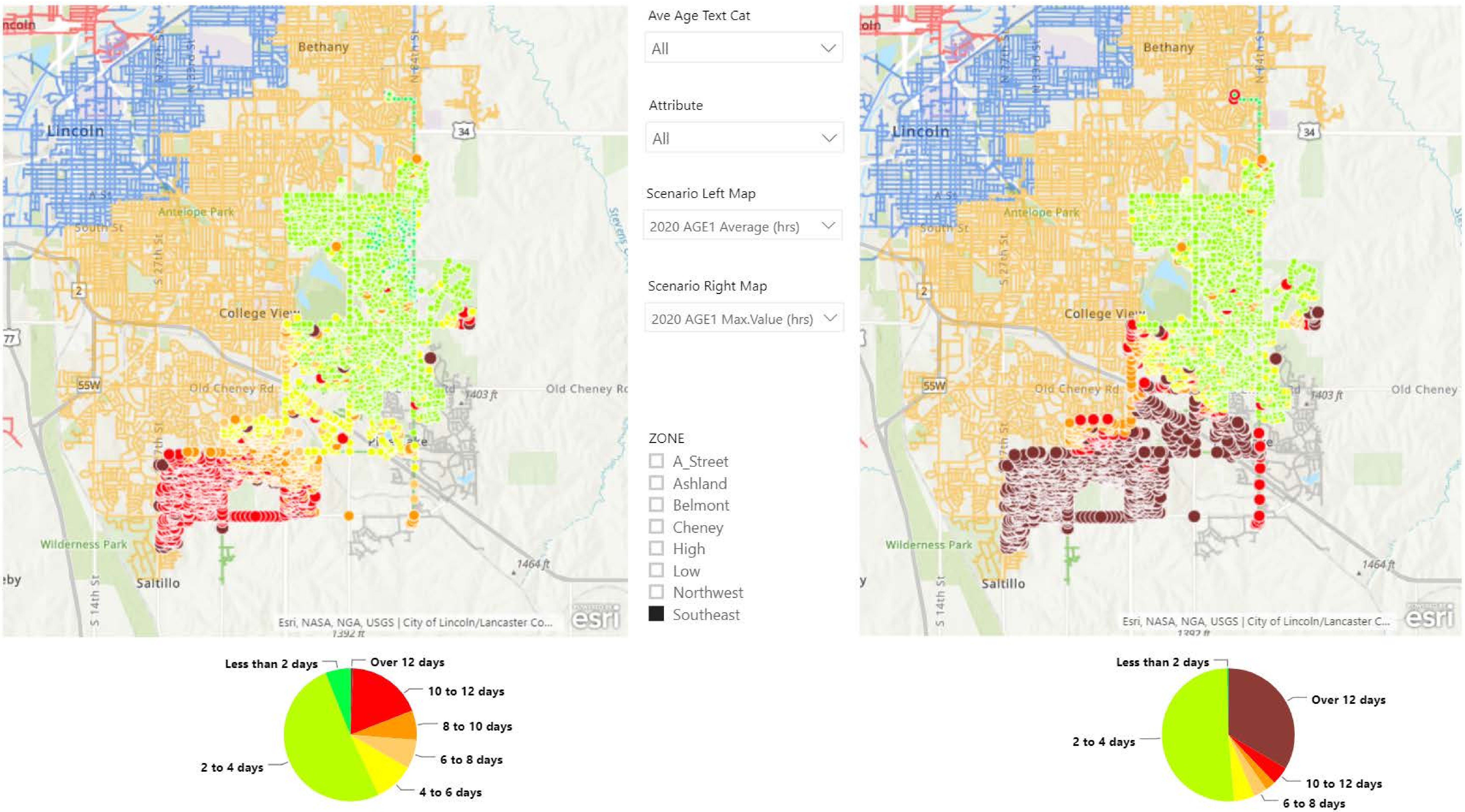


Figure 7-27 Southeast SL Water Age Scenario 1 (Average-Left, Maximum-Right)

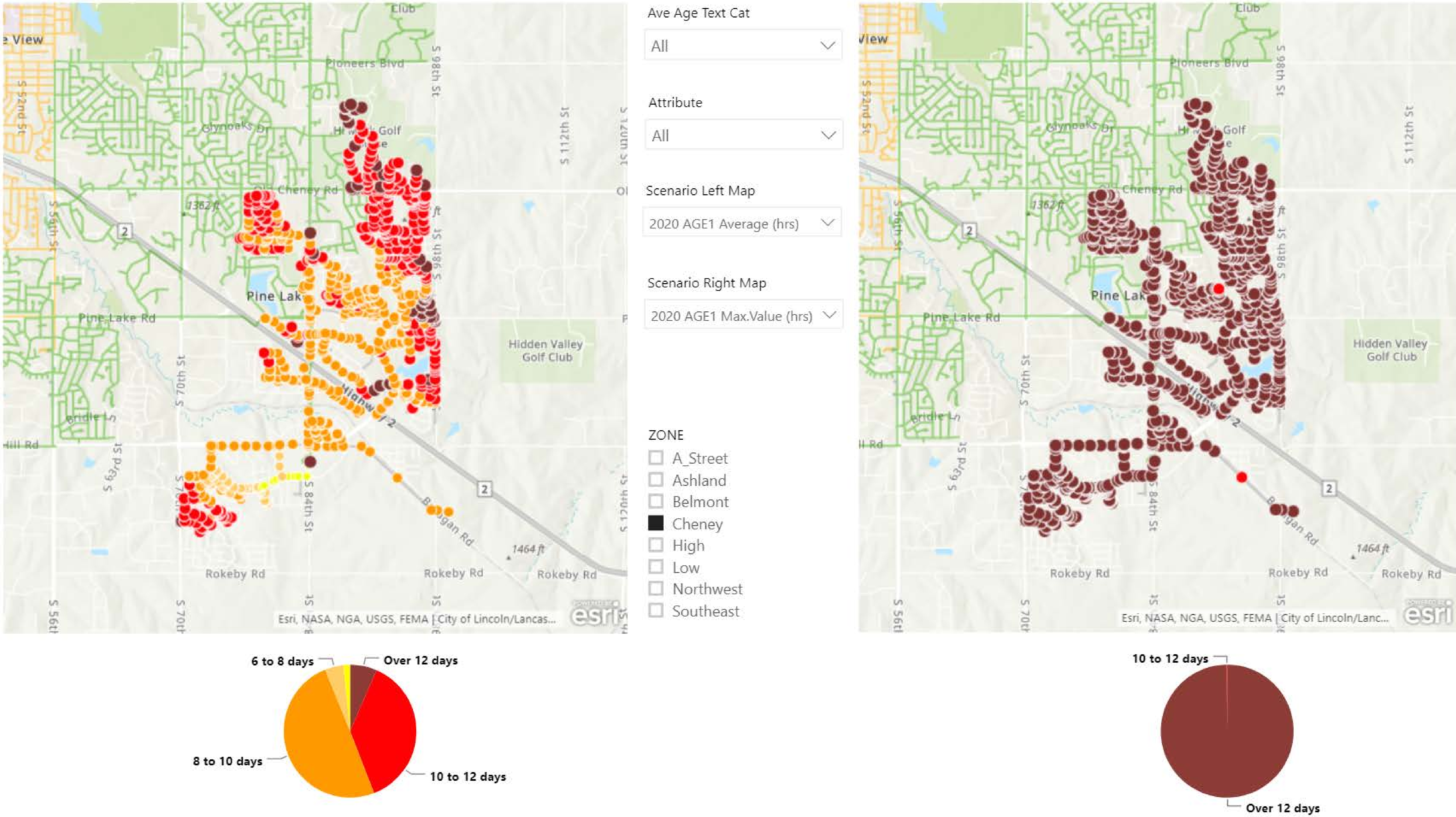


Figure 7-28 **Cheney SL Water Age Scenario 1 (Average-Left, Maximum-Right)**

7.7.3 Water Age Modeling Scenario No. 2

A second water age scenario was developed, identical to the first scenario with the exception that the NW 12th Reservoir was taken out of service instead of Airpark. This only impacted the water age results in the Belmont and Northwest Service Levels, and ages in other Service Levels did not change so these need not be shown in additional figures. To compare the difference between placing Airpark vs. NW 12th out-of-service during Winter Operations, Figure 7-29 (average age) and Figure 7-30 (maximum age) are provided on the following pages. The left side figures show the water age results from Scenario 1 (Airpark out of service, “o.o.s”) and the right-side results show the water age results with Scenario 2 (NW 12th out of service).

It is interesting to note that the water ages in the Belmont and Northwest Service Level are higher on average when the Airpark reservoir is taken out-of-service (left-side figures) compared to the scenario where NW12th is taken out-of-service (right-side figures). This is due to the fact that NW12th has a larger volume and adds more residence time to the water within the Belmont and Northwest Service Levels. However, the water age alone does not tell the complete story because the travel path of water needs to be considered. The water age might be higher in the Northwest Service Level with Airpark Reservoir out-of-service, but much of the water feeding the Northeast Service Level flows through newer pipes and fewer cast iron pipes. To illustrate this example, a source trace was performed for both of the base water age scenarios to show the relative blending zones between the Belmont Pumping Station water and the Pioneers Pumping Station water based on which reservoir is taken out-of-service. This is shown in Figure 7-30 with the 50/50 approximate blending zone line drawn in blue over the top of the figures. While the age may be lower with the Airpark Reservoir out-of-service vs. the NW 12th out-of-service, most of the water that is pumped into the Northwest Service Level has its source from the Pioneers Pumping Station and its flow path has gone through the Airpark area, where many older Cast Iron Pipes reside. Conversely, water age may be lower with the Airpark Reservoir out-of-service, but the water pumped into the Northwest SL is roughly a 50/50 blend of water between Belmont and Pioneers Pumping Stations. This means, on average, less of the water being pumped into the Northwest Service Level, when the Airpark Reservoir is out-of-service, has its source from the Pioneers Pumping Station which must flow through the higher-age Cast Iron Pipes in the Airpark area. This provides an example of why conclusions about system water quality should not be drawn based on water age alone, especially when considering constituents that are significantly impacted by the pipe-wall interactions such as Chlorine.

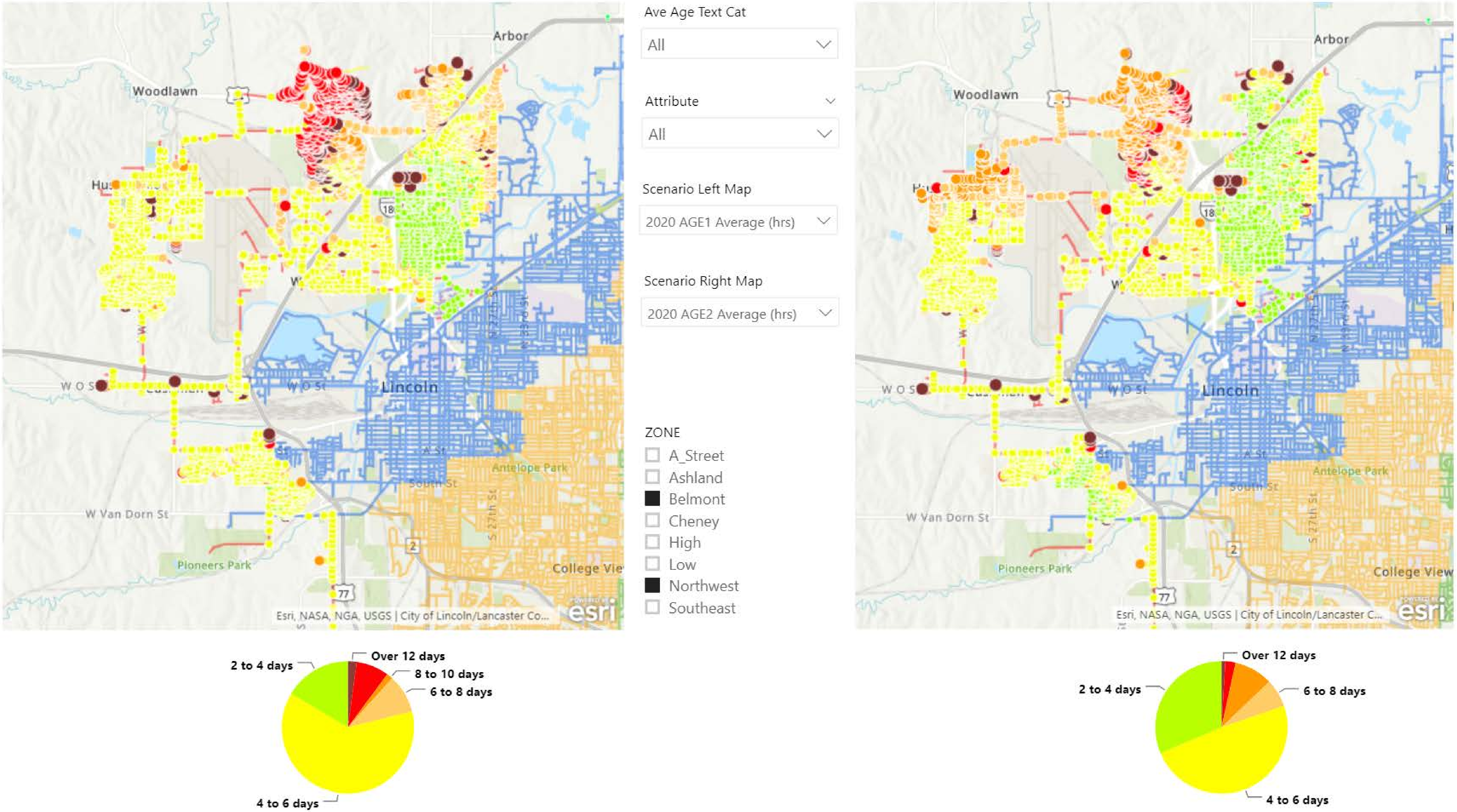


Figure 7-29 **Belmont/Northwest Average Water Age (Airpark o.o.s Left, NW 12th o.os Right)**

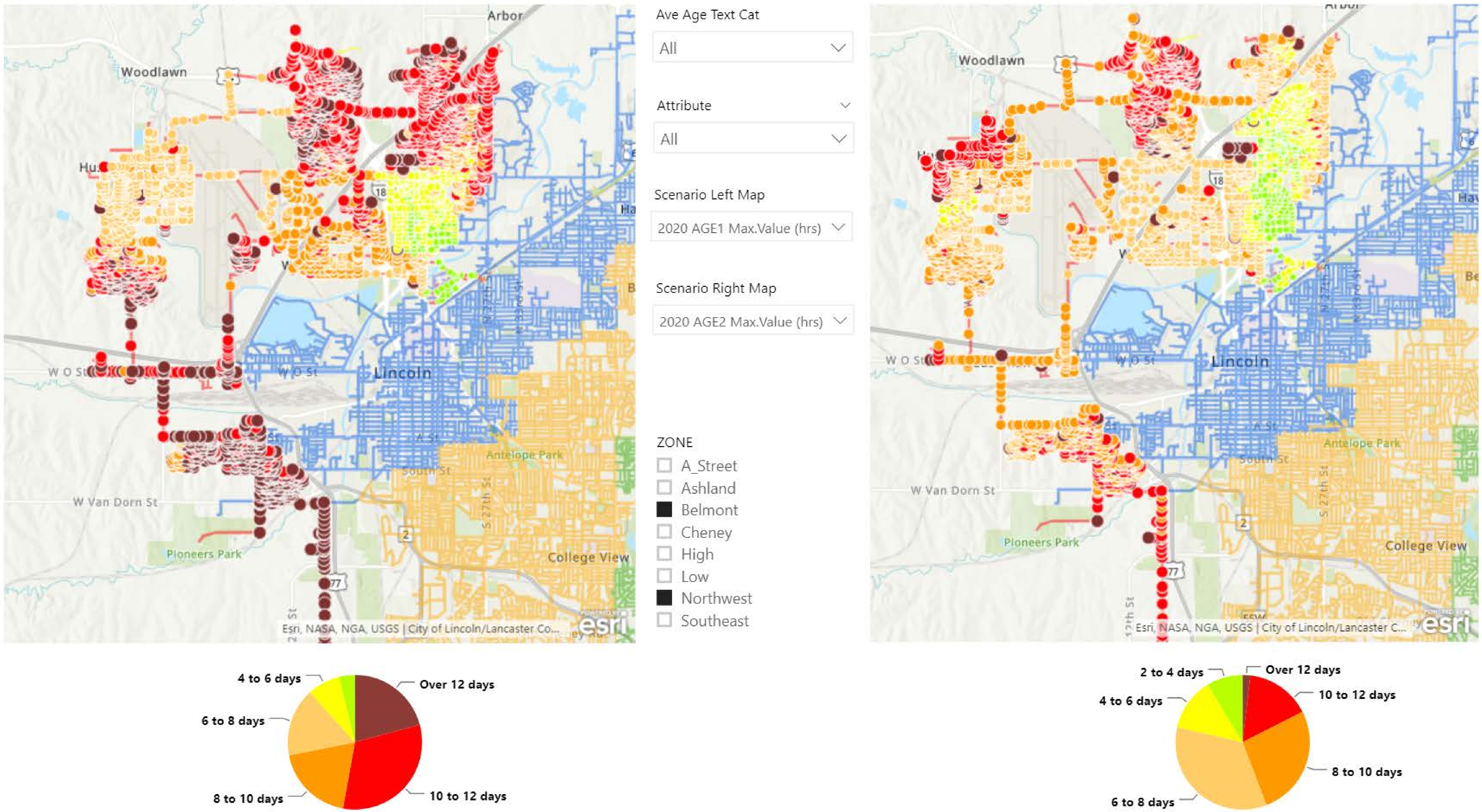


Figure 7-30 **Belmont/Northwest Maximum Water Age (Airpark o.o.s Left, NW 12th o.os Right)**

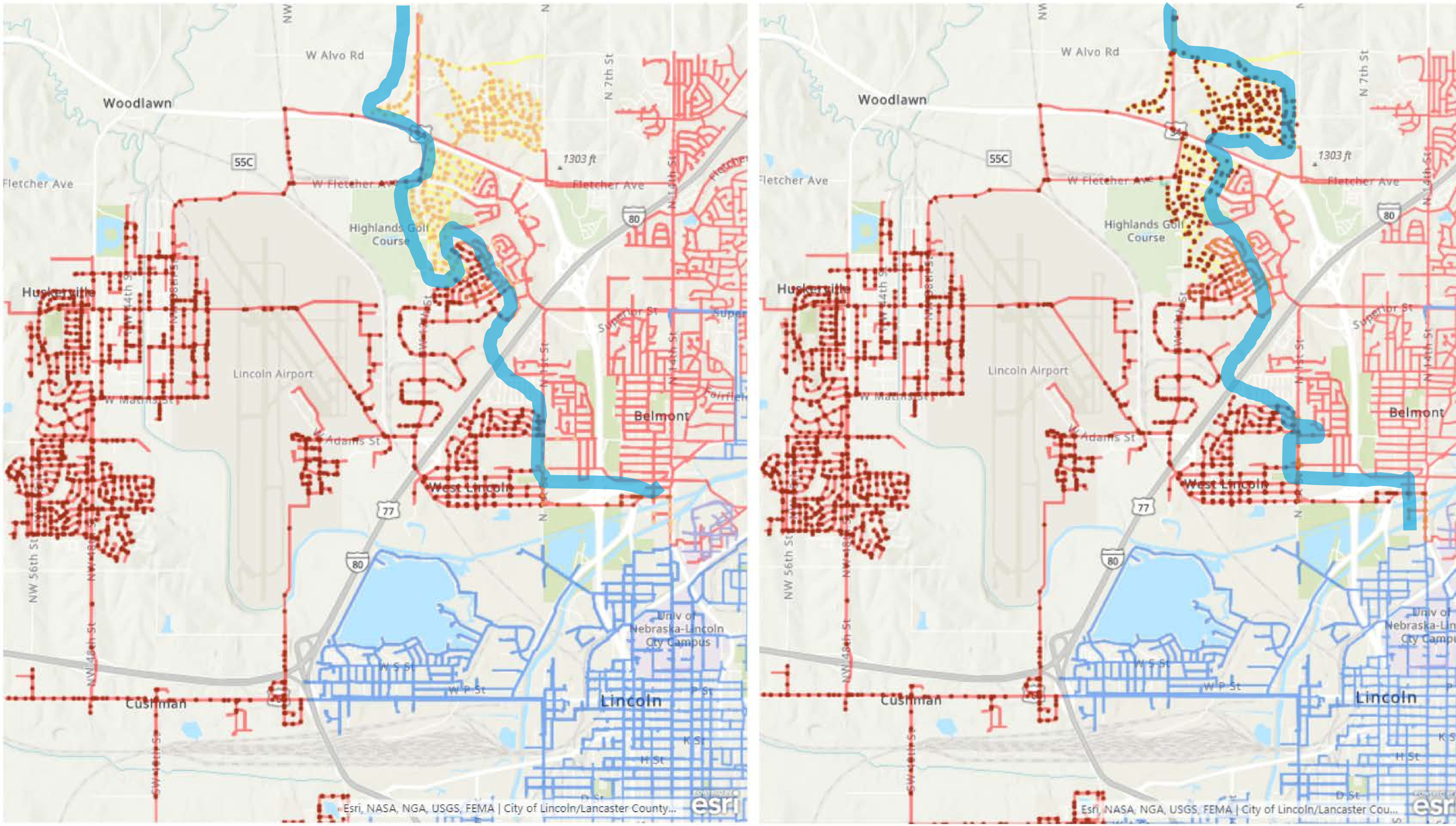


Figure 7-31 **Pioneers PS Influenced Area (Airpark o.o.s Left, NW 12th o.o.s Right)**

7.7.4 Modeled Water Age Relationship with Observed Water Quality

Further analysis was conducted to assess the relationship between the modeled water age and the observed water quality. The modeled water age results from Scenario No. 1, which simulates October operating conditions, were compared with the monitored water quality data for chlorine residual, nitrite, and nitrates from October 2018. This is the closest comparison that can be made because the scenario was developed based as closely as possible on the actual operating controls that occurred in October 2018. The process for developing a relationship involved assigning the monitoring locations to the closest model junction and pipe. This was done to determine if there was an observable difference in the water quality trends as they relate to the pipe material, relative pipe age, and the Service Level in which the monitoring is being conducted.

Scatter plots were developed in the dashboard to review observable patterns based on each of the parameters described above (pipe material, age, and service level). Scatter plots demonstrating water quality vs. water age for the entire distribution system are shown in Figure 7-32 with a map for reference on the left. Each scatter plot uses the same x, y relationship between modeled water age and chlorine residual. The details in each scatter plot highlight different distribution infrastructure parameters with material (top-right), decade category (middle-right) and Service Level (bottom-right).

The overall relationship between modeled water age and chlorine residual on the scatter-plots are representative of an empirical, or observed, chlorine decay curve. While there is some variability in the scatter plots, the scatter plots demonstrate the expected relationship between water age and chlorine residual, with residual declining as water age increases. Figure 7-33 through Figure 7-36 shows these relationships with the data isolated for individual service levels. Scatter plots demonstrating the relationship between modeled water age and nitrate and nitrite concentrations across the entire distribution system are provided Figure 7-37 and Figure 7-38, respectively. Because there is much less data available for these constituents, only an overall figure is provided. As expected, the scatter plots demonstrate increasing nitrite and nitrate concentrations with increasing water age. From a review of these figures, the following observations can be made:

- The scatter plots demonstrate that chlorine residual decay occurs most rapidly in the High Service Level.
- The next most rapid decay of chlorine residual was observed in the Belmont Service Level.
- The Low Service Level has a more moderate decay of chlorine residual. Distance from supply entry into the Low Service Level to the end user (travel path) is much shorter in this Service Level than in High or Belmont Service Levels.
- The Southeast and Cheney Service Levels have the least variability of all the scatter plots and show a much more gradual rate of chlorine decay.
- Decay relationships between pipe decade category and/or pipe material do not show any observable trends that can be separated out. One of the reasons for this is that the characteristics at the sampling site do not necessarily consider upstream piping that the water has already traveled through to get to the monitoring location. For example, a monitoring location may be served through new PVC pipe, but the water would have traveled through mostly older cast iron pipes before arriving at the monitoring location.

In order to illustrate how the relationships between the empirical, or observed, decay trends may be related to pipe materials and age, four additional figures were developed to show the relative percentages for each Service Level of pipe material by volume (Figure 7-39), pipe material by length (Figure 7-40), pipe age by volume (Figure 7-41), and pipe age by length (Figure 7-42). The larger relative amounts of cast iron pipe, by percentage of total volume/length, within the High and Belmont Service Levels supports that the more rapid decline of residual is likely occurring because of higher biological activity from denser biofilm. Although the same interactions are occurring in the Low Service Level, which also has a high percentage of cast iron pipe, there is less contact time through these pipes because the distance between the supply entering the Service Level to the end-user is much shorter. For the High Service Level, there is much longer travel distance for water to arrive at the southern portions and a higher probability that water has traveled through numerous sections of pipes with a cast iron material before it reaches the customer's tap.

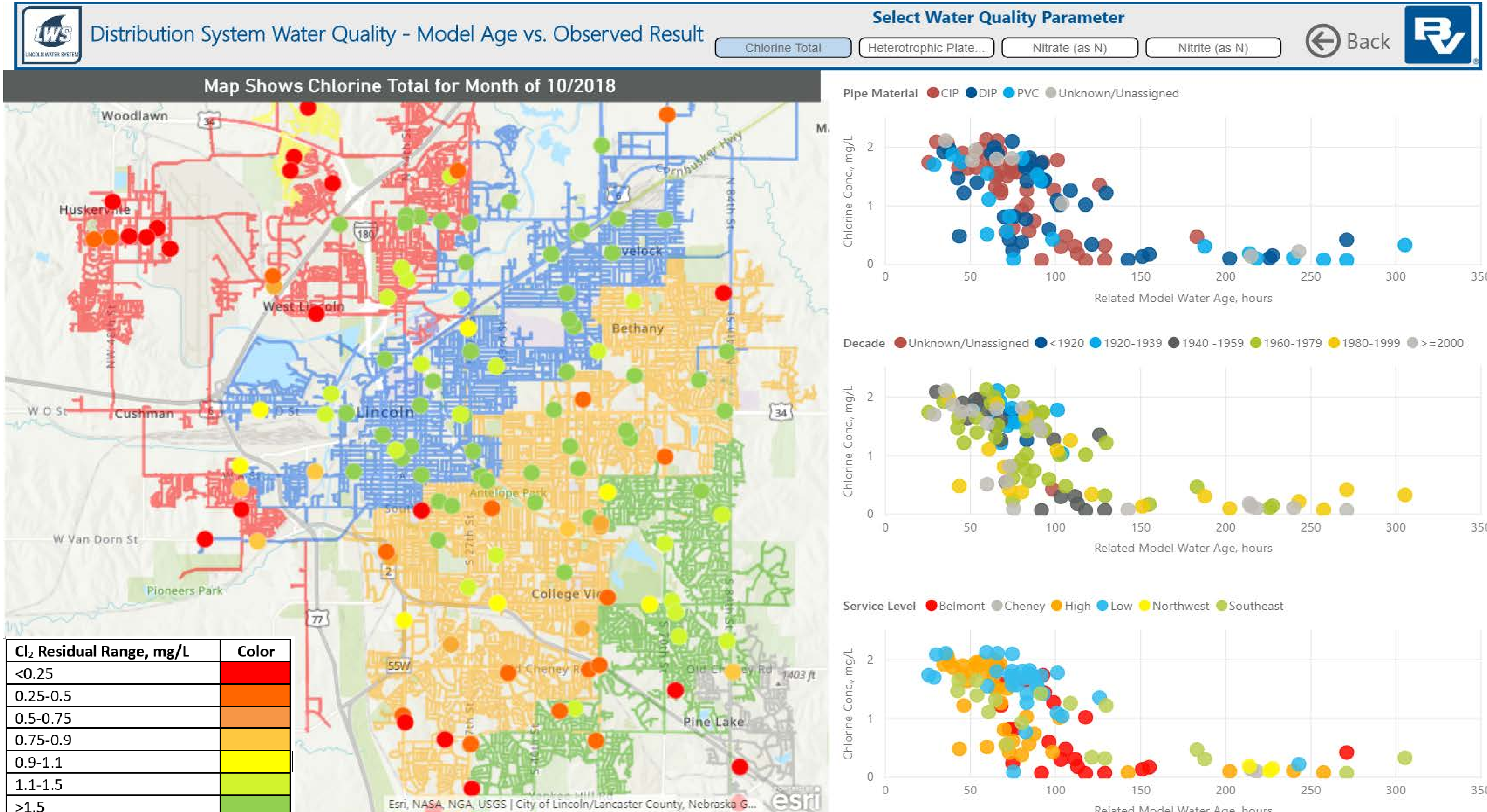


Figure 7-32 **Overall System Modeled Water Age vs. Observed Chlorine Residual**

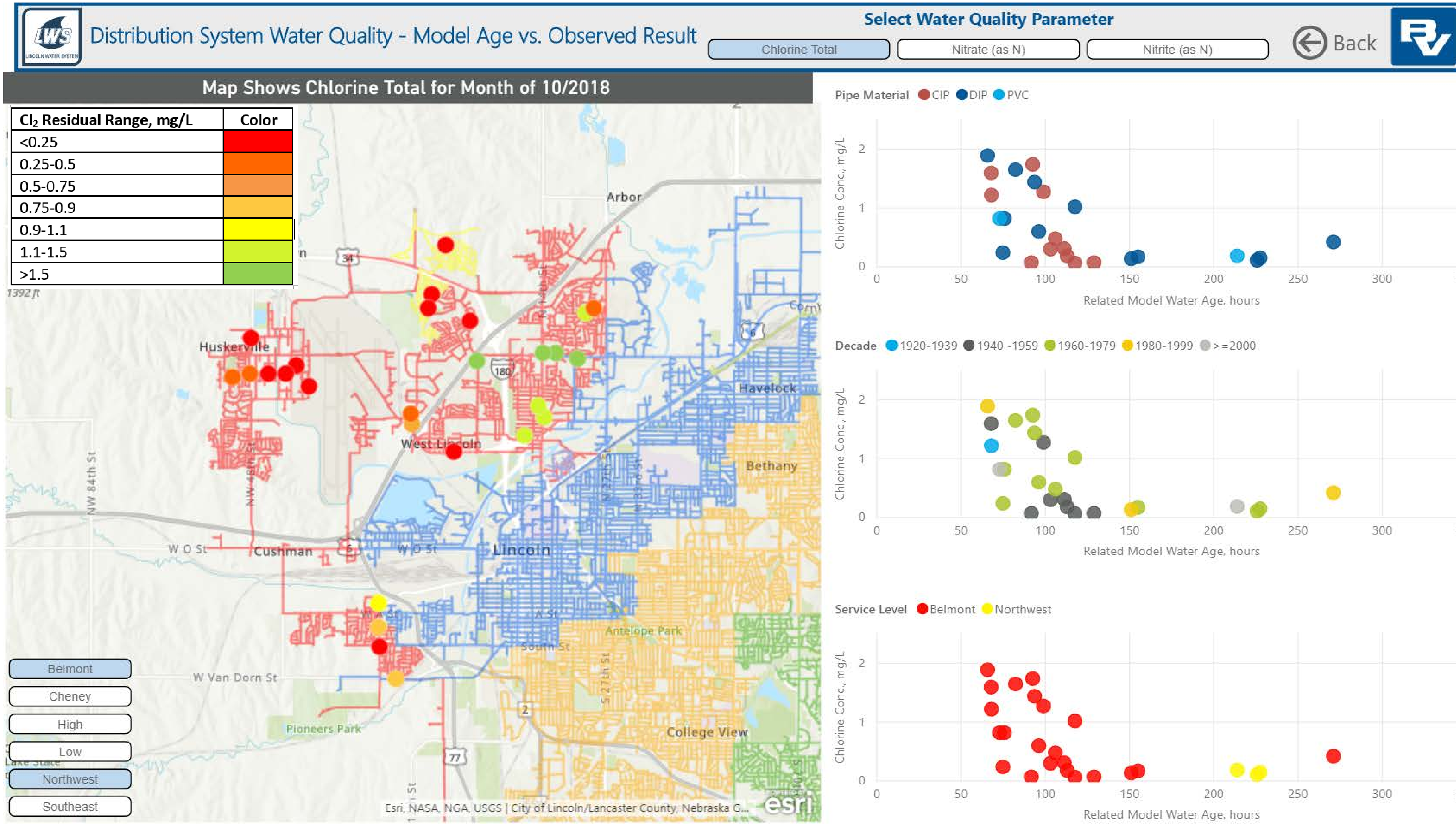


Figure 7-33 **Northwest/Belmont SL Modeled Water Age vs. Observed Chlorine Residual**

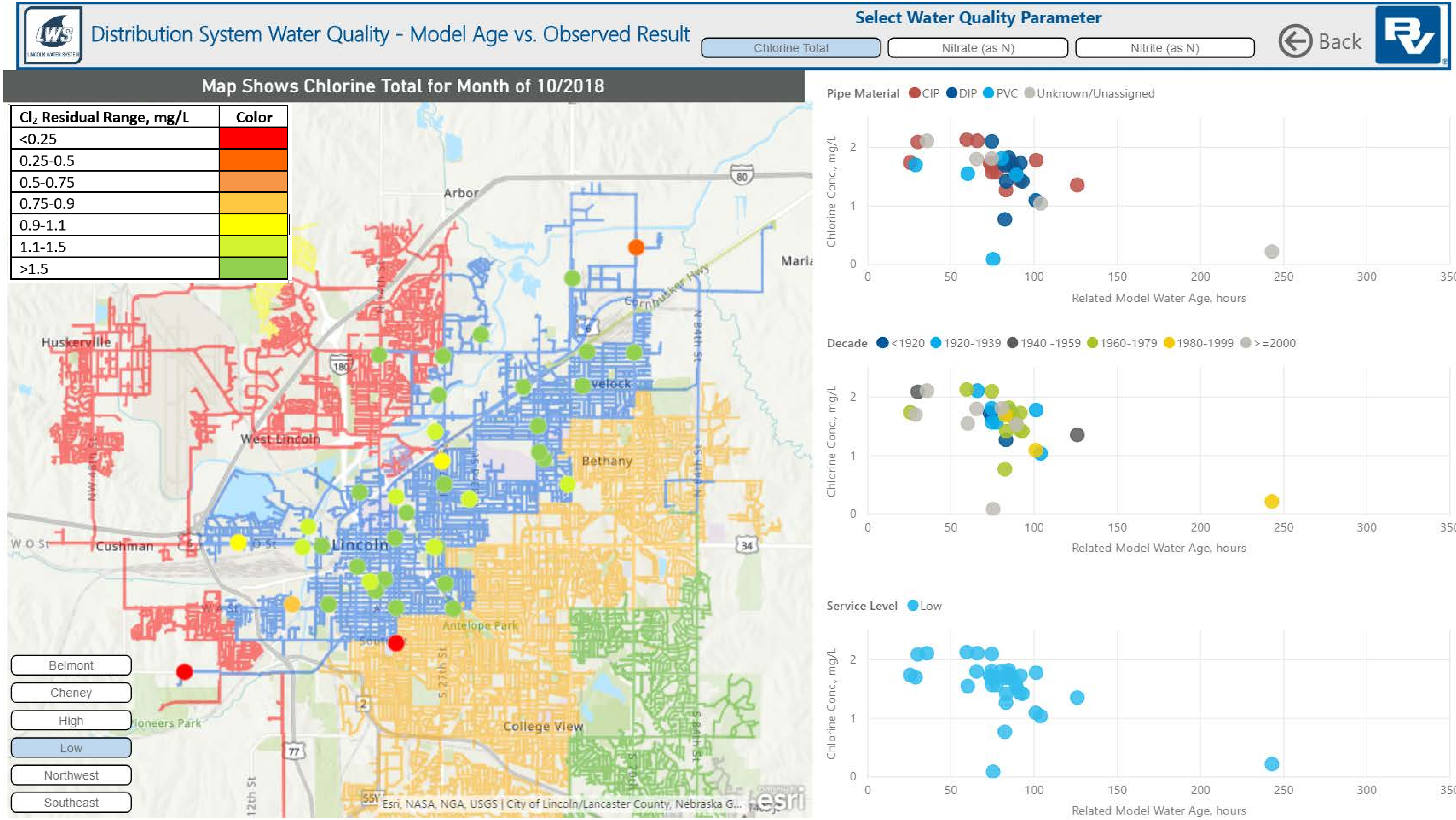


Figure 7-34 **Low SL Modeled Water Age vs. Observed Chlorine Residual**

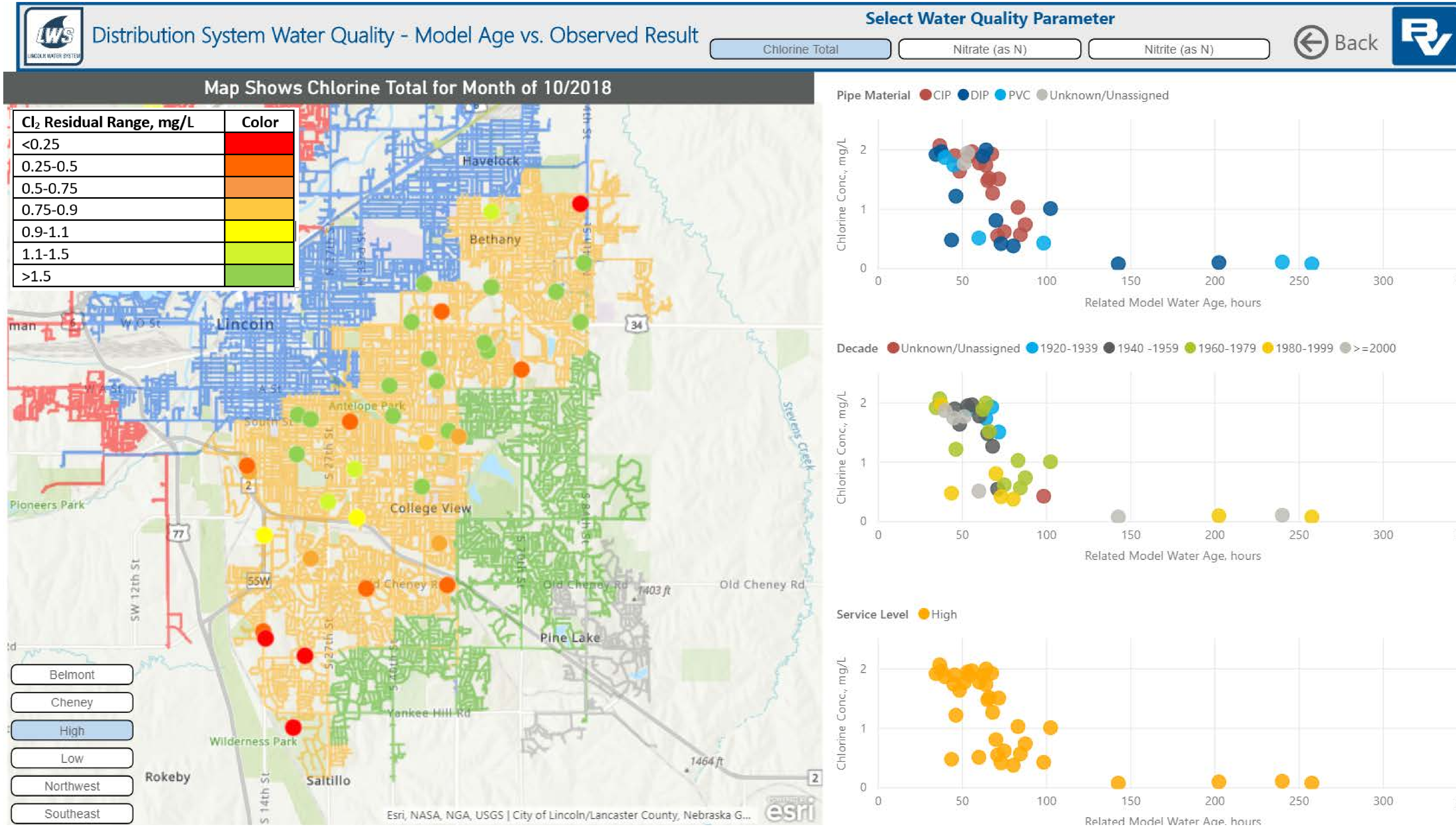


Figure 7-35 **High SL Modeled Water Age vs. Observed Chlorine Residual**

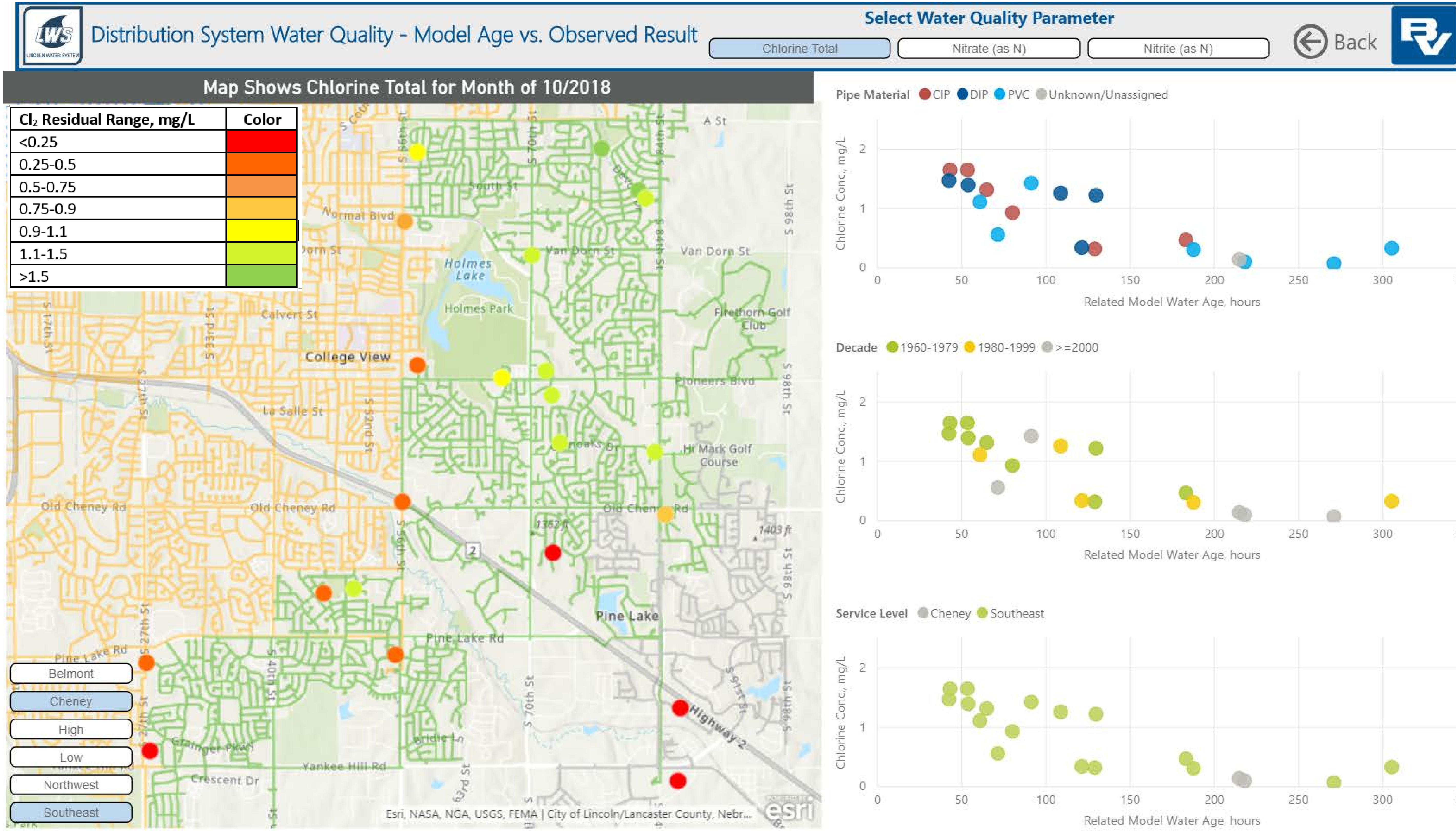


Figure 7-36 Southeast/Cheney SL Modeled Water Age vs. Observed Chlorine Residual

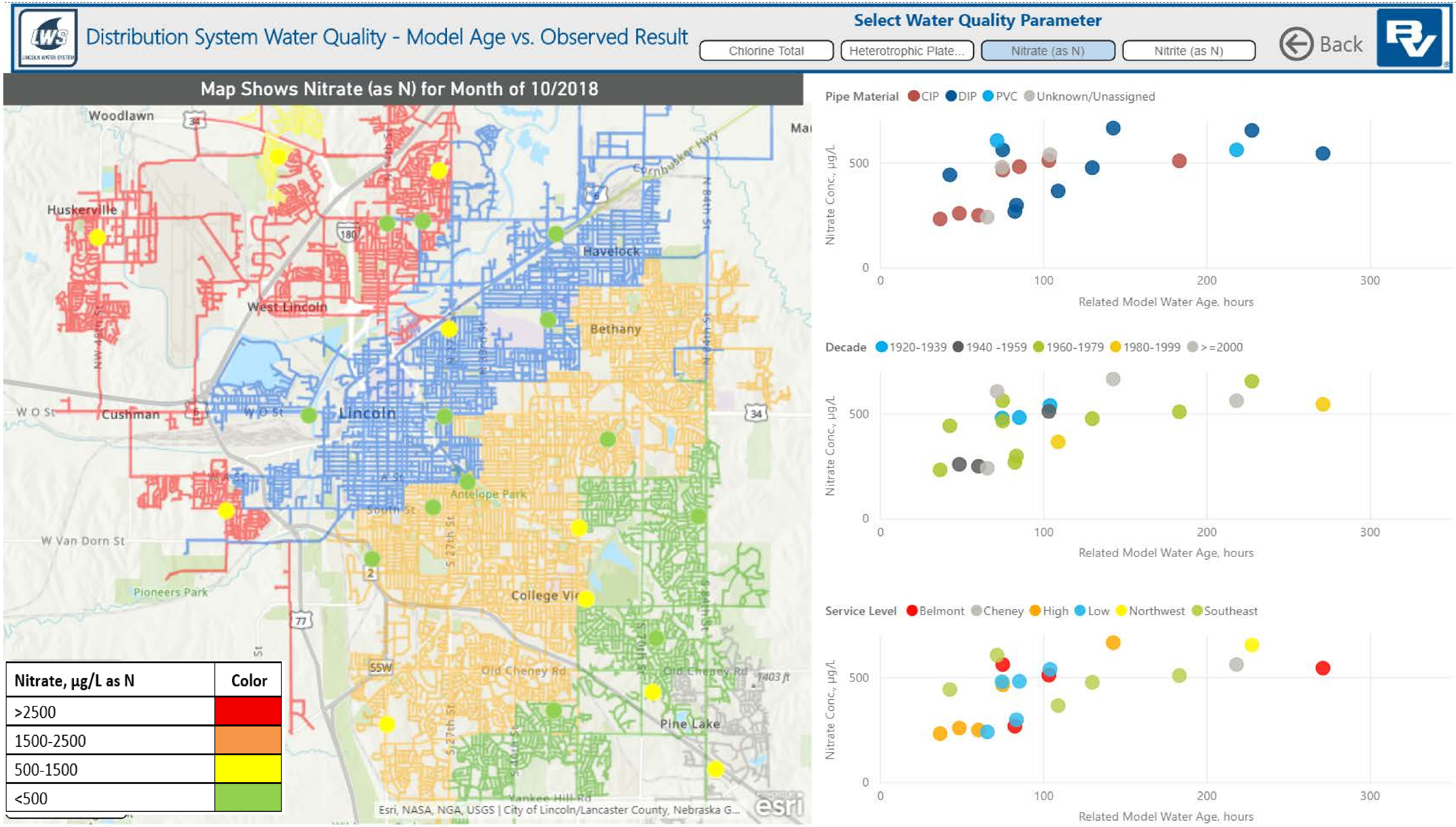


Figure 7-37 **Overall System Modeled Water Age vs. Observed Nitrate Concentration**

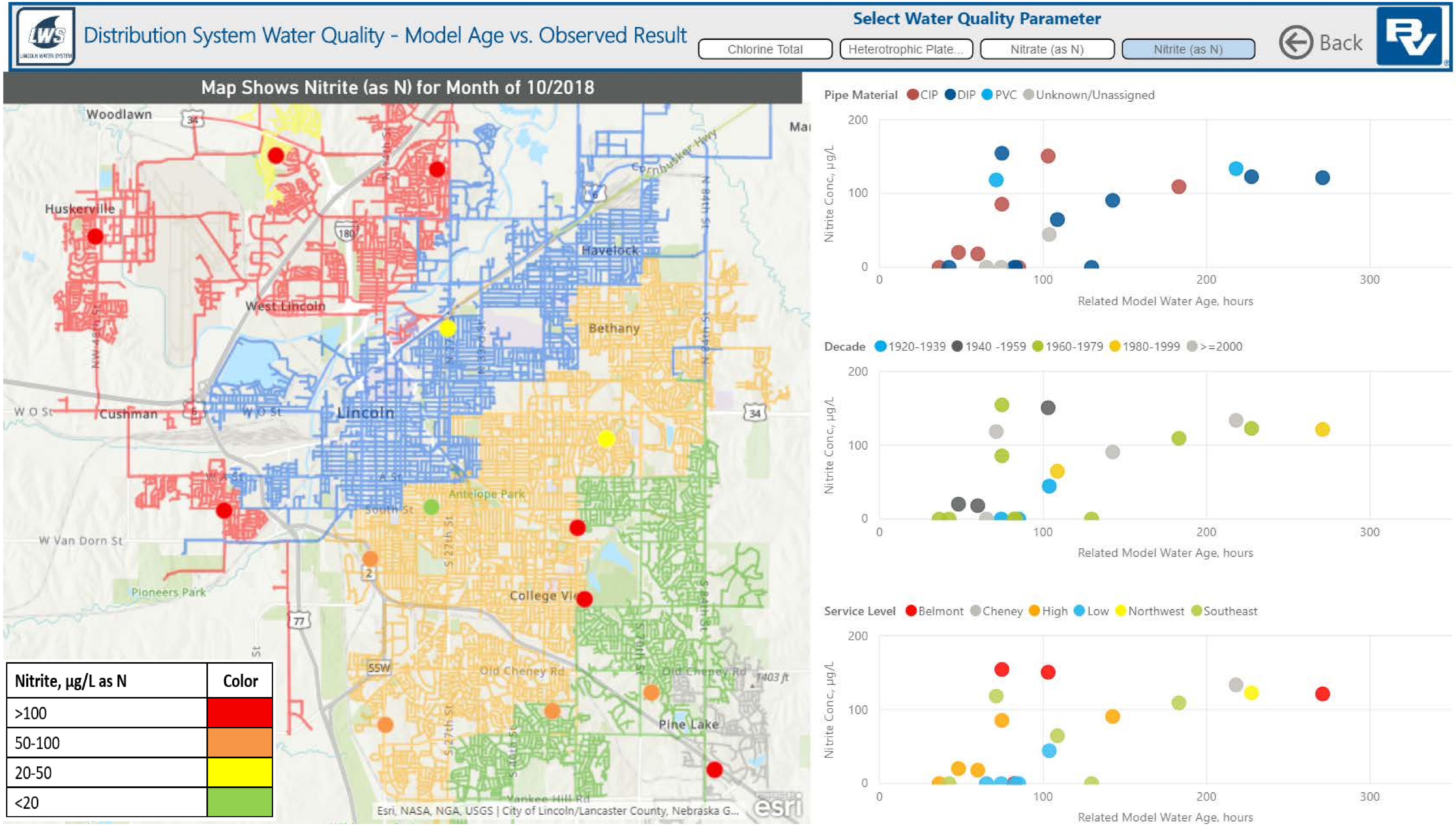


Figure 7-38 **Overall System Modeled Water Age vs. Nitrite Concentration**

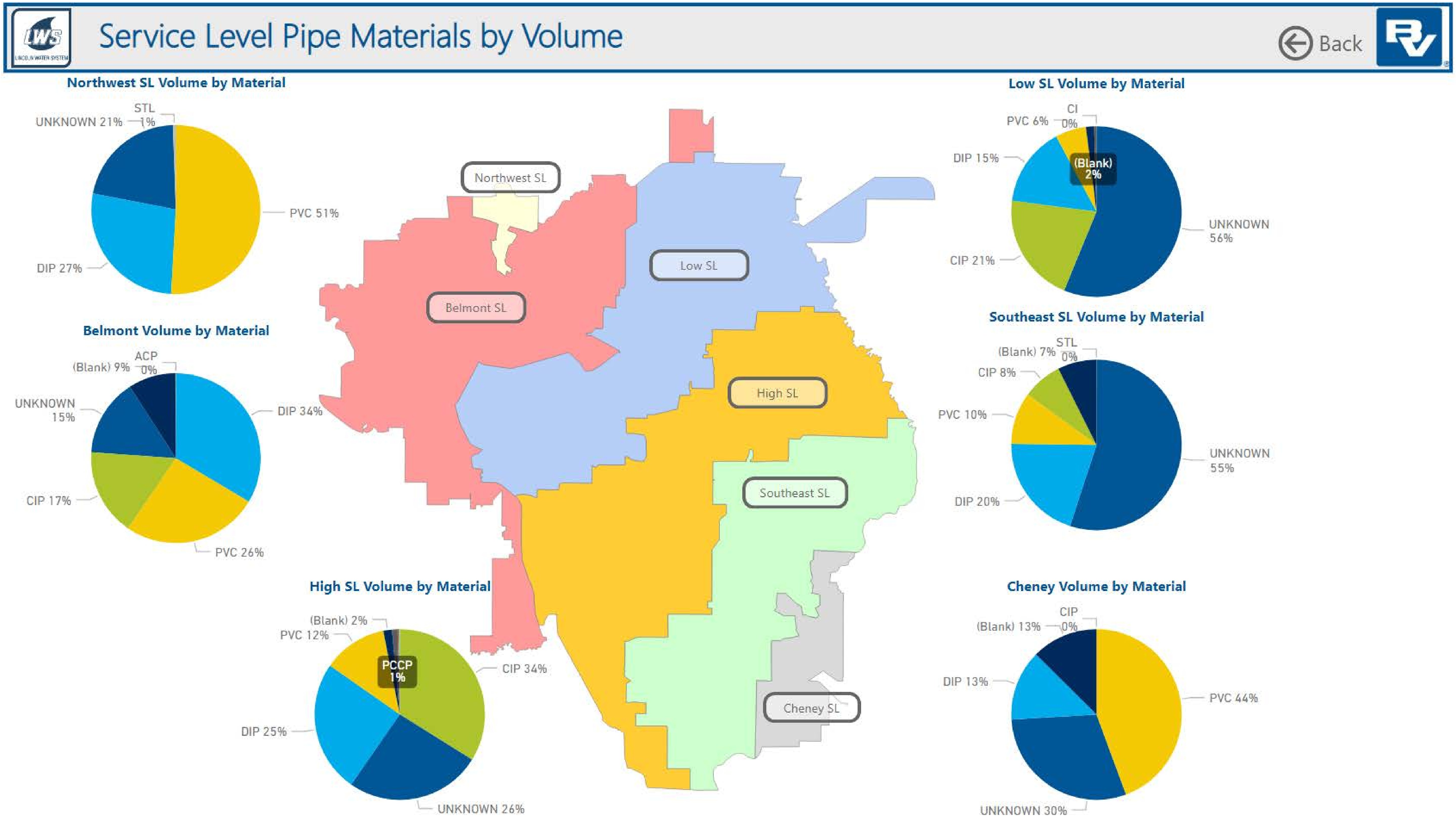


Figure 7-39 Service Level Pipe Material by Volume

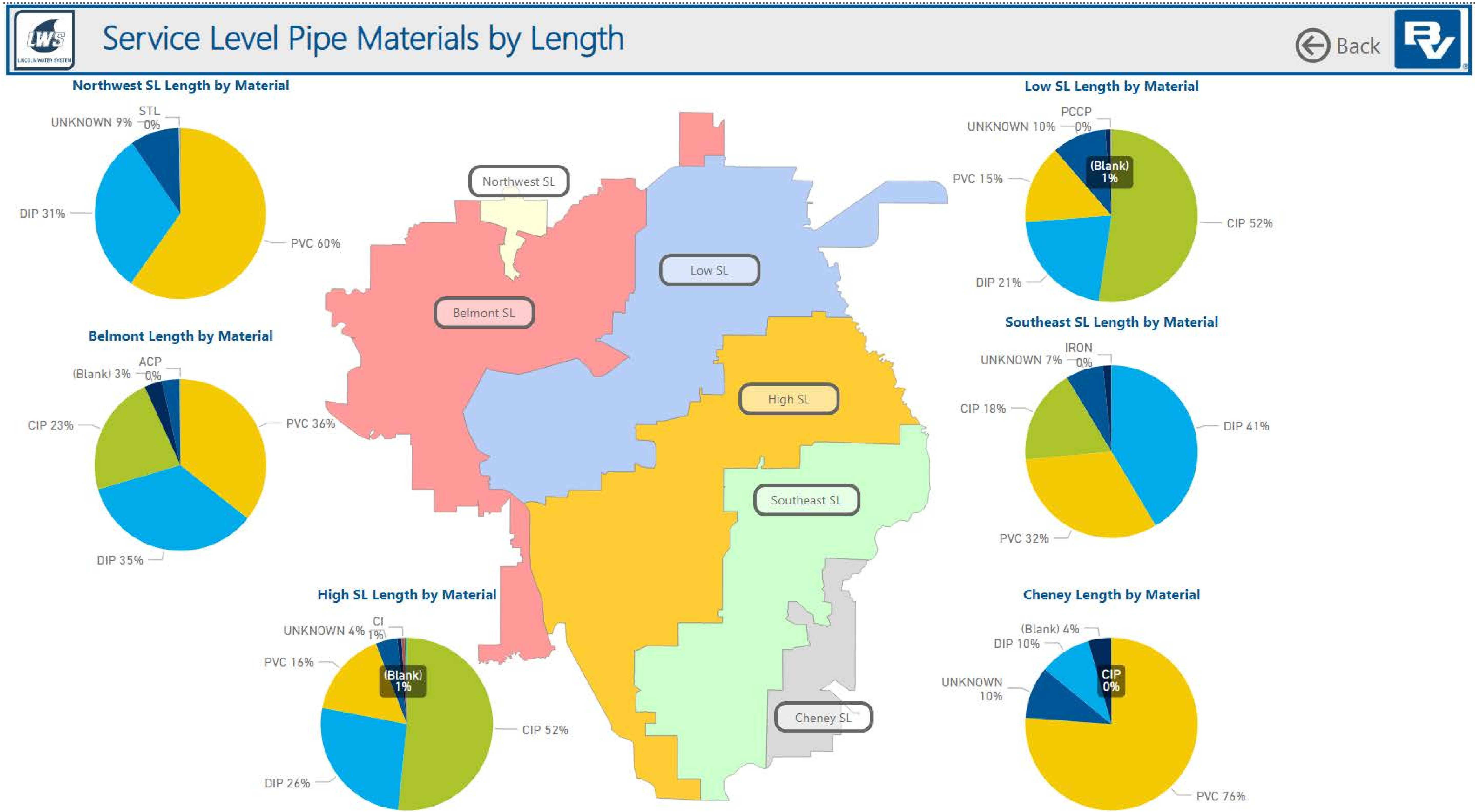


Figure 7-40 **Service Level Pipe Material by Length**

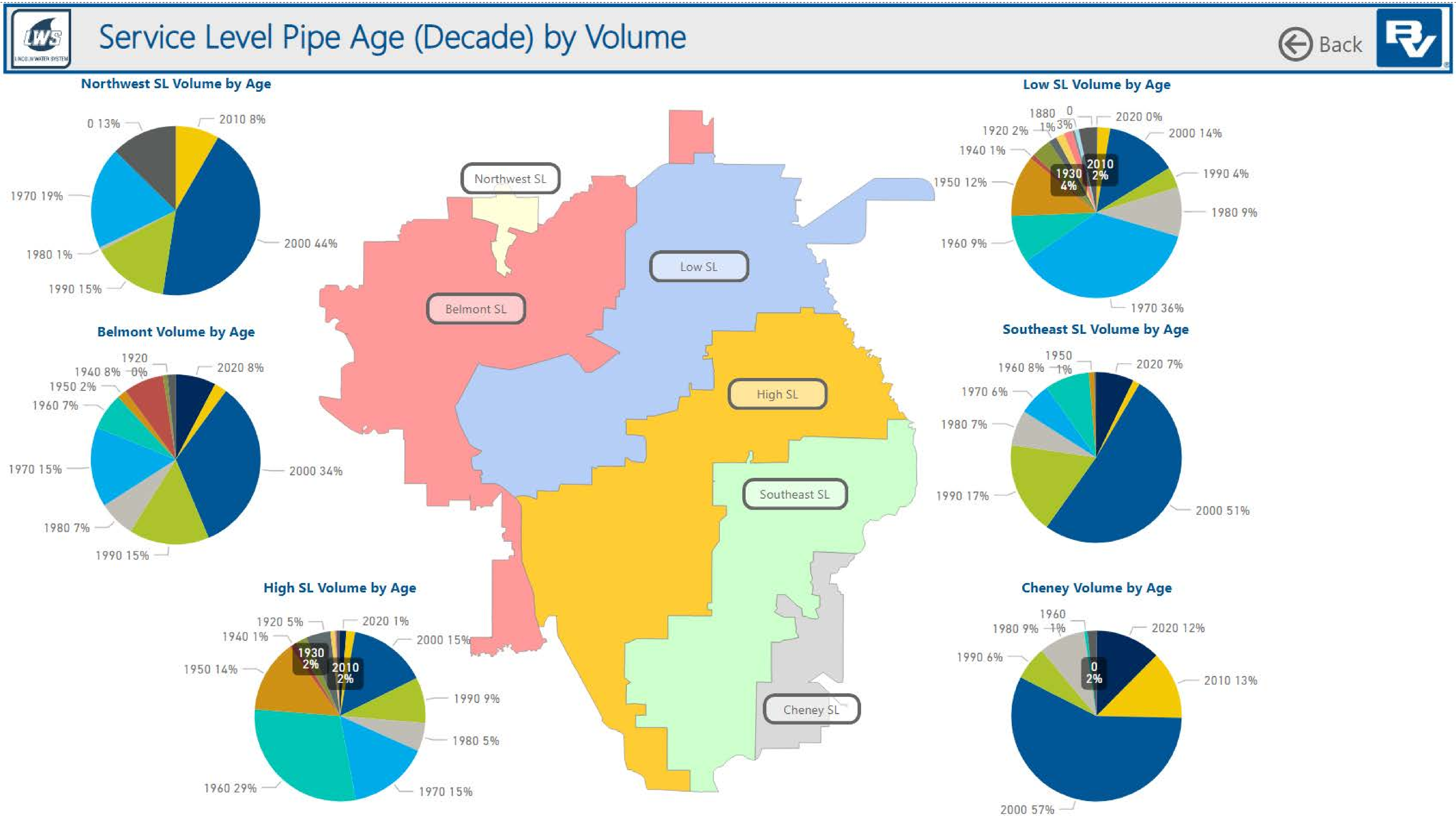


Figure 7-41 **Service Level Pipe Age (Decade) by Volume**

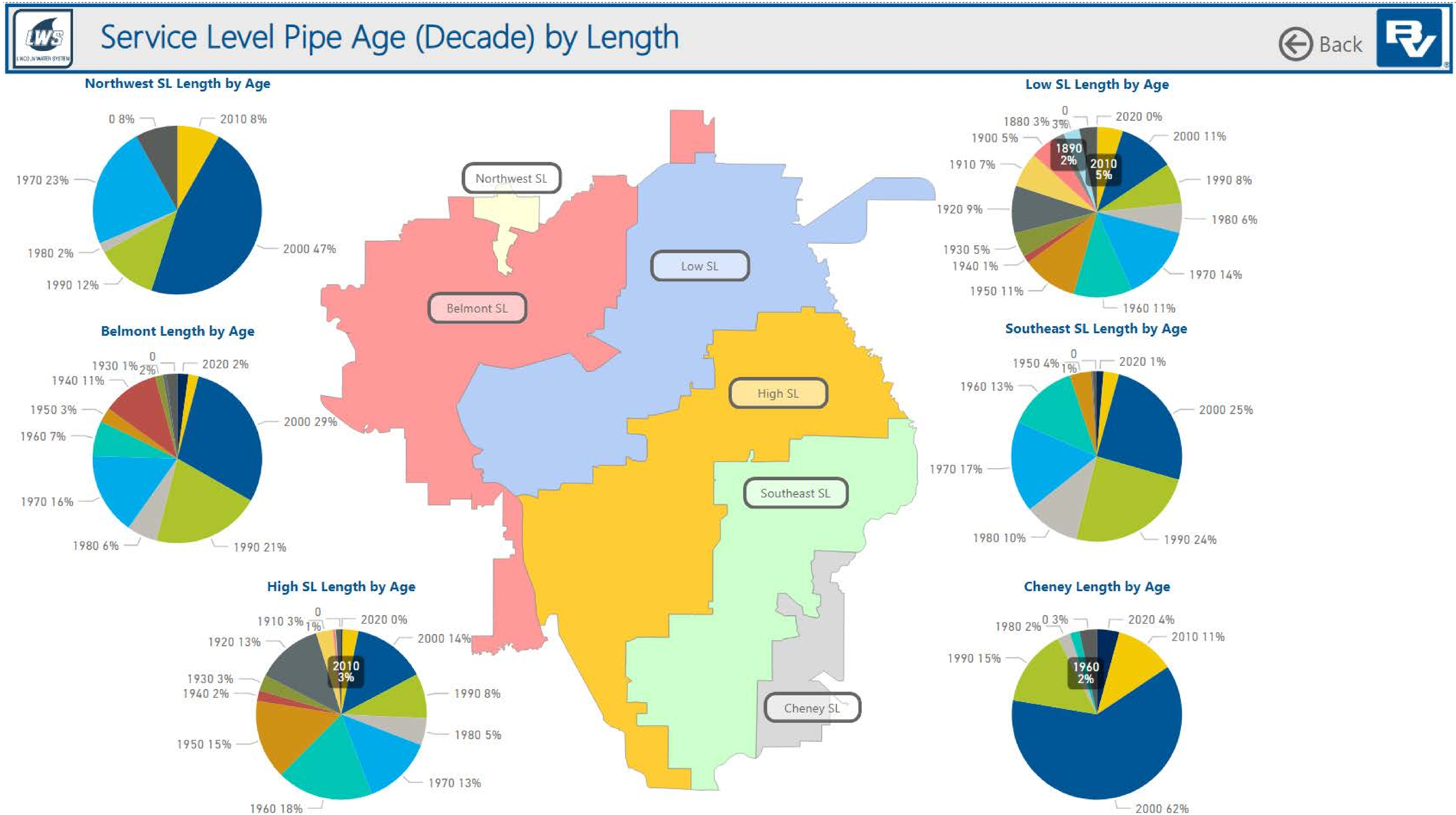


Figure 7-42 **Service Level Pipe Age (Decade) by Length**

7.1.3 Yankee Hill "Influence Zone"

One final modeling scenario was performed using the controls and operations of the Age Scenario No. 1 to determine the area impacted by the Yankee Hill Reservoir. Source trace analysis is used to identify the percentage of water that comes from a given source (e.g. reservoir or pump station). A source trace using the Yankee Hill Reservoir as a "source" was performed and the model results were captured in the dashboard. Figure 7-43 compares the areas that are source influenced by Yankee Hill (e.g. water which feeds the area has passed through the reservoir) on the left compared with the water age modeling results on the right. The highest ages in the southwest portion of the Southeast Service Level can be attributed almost exclusively to the fact that a blended 80-percent of the water has passed through the Yankee Hill Reservoir and then comes across to this area through the new main along Yankee Hill Drive. An alternative way to think about this blending area is that 8 times out of 10, when a customer opens their tap, they would be receiving water that has resided in or passed through the reservoir.

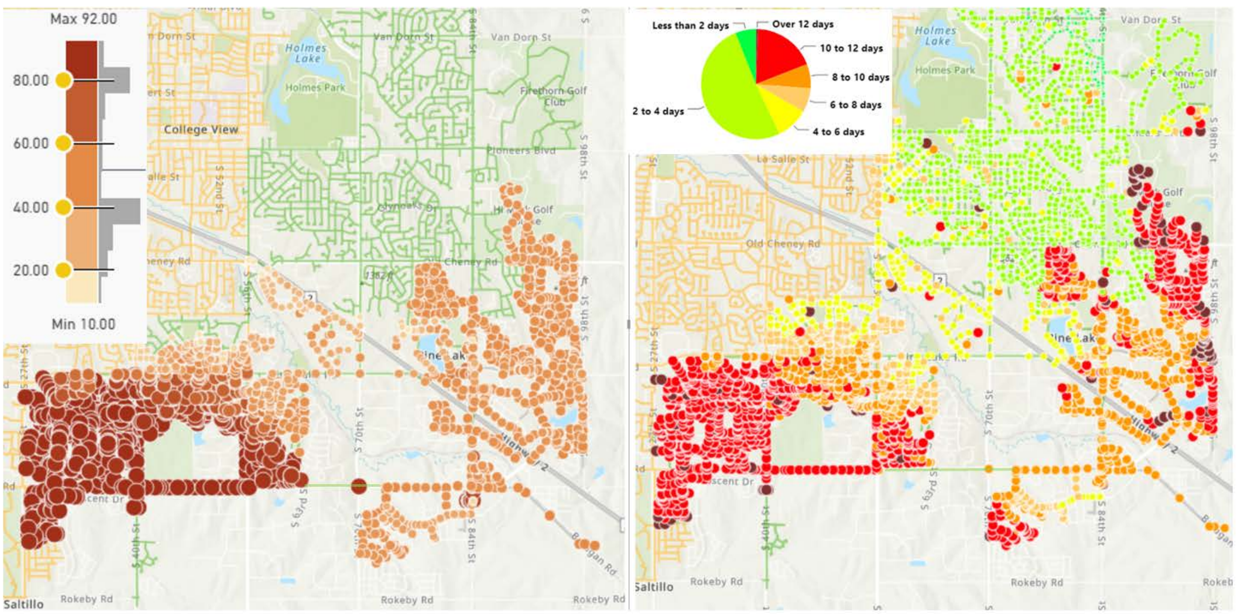


Figure 7-43 Yankee Hill Source Influence vs. Southeast/Cheney Water Age

7.8 Distribution System Water Quality Improvement Alternatives

7.8.1 Water Quality Modeling Alternatives

Additional water age modeling was performed to evaluate the impact that auto-flushers may have in reducing water age and consequently improving water quality. Generally, it takes a large volume of water flushed before major improvements in water quality can be observed. To provide an example of this, in 2018 Black & Veatch worked on an auto-flushing optimization project with a confidential utility in the southeast part of the country. As part of this project, extensive field testing and sampling was performed with the utility's current auto-flushing program. This utility maintains over 30 auto-flushers in a system a little larger than the Belmont Service Level. The utility desired to understand the impact on water quality from operating this many auto-flushers and to quantify the benefits relative to the expense of increased operational costs. Water quality sampling was performed at 50 different locations at, near, and distant from the existing auto-flushers for a week when the auto-flushers had been turned off for the prior three-weeks and for a week when the auto-flushers had been operating for the prior three-weeks. Chlorine residuals leaving the plant were maintained at a constant value through these periods to provide as much consistency with the sampling data as possible and utility operations for pumping and the use of storage were also maintained consistent during the non-flushing and flushing sampling periods. The weather and temperature, fortunately, were relatively consistent through both periods.

The results of the program demonstrated that in the sampling areas where auto-flushers were located, there was an improvement in water quality. However, the water quality benefits were not as significant as originally anticipated. Additionally, locations further away from the auto-flushers (a few blocks over or somewhere else in the system between the supply and the auto-flusher locations) experienced a range of marginal to almost negligible difference in water quality. As a result of this project, the utility decided to reduce the number of existing auto-flushers rather than to continue installing more auto-flushers. The utility was able to identify and prioritize auto-flushing locations that had the highest positive impact on water quality.

This example was provided not to discourage the installation of auto-flushers or to argue against them but to provide a cautionary example of implementing auto-flushers as a global solution to water quality. Locations should be carefully developed when installing auto-flushers, as should the dates/times/rates of the auto-flushing. The *2014 Master Plan* identified some key locations that auto-flushers could be placed to improve water quality and in review of these locations they appear to be well-placed. The modeling age modeling with flushing which will be shown on the following pages, also supports these locations. With the addition of auto-flushers, it should be recognized that their zone of influence could be more localized and will not resolve most water quality concerns in a Service Level, unless the flushing-to-demand ratios are very high. One additional consideration when using auto-flushing equipment that senses chlorine residual and responds in kind, is that flushing could occur very frequently if left on auto-control at some locations because chlorine residuals may be consistently below the desired threshold during the more water quality challenging times of the year. Caution should be taken when setting up auto-flushing controls to avoid over-flushing and having too large of an impact on the cost of operations.

To quantify the benefits to water quality vs. the volume of water flushed, four locations were selected in the model to simulate auto-flushers and to review the water age results. A conservatively large flushing volume was used at these locations and they were simulated as an every-day flush at a rate of 150 gpm for a two-hour period in the early morning. This relates to a conservatively large daily volume of 18,000 gallons per auto-flusher. They were simulated to come on at the time when reservoirs began to fill so that it could be ensured that fresh water coming from the points of supply entry was being pulled through the system instead of pulling water from storage and creating a sloshing effect. The locations where these auto-flushers were simulated are the following:

- Cheney Service Level - Dempster Drive & Countryview Road
- Northwest Service Level - Isaac Drive & NW 10th Street
- Southeast Service Level – Whispering Wind Boulevard and S. 29th Street
- Belmont Service Level - Folsom Street & W. Denton Road

The results of the third water age scenario with high auto-flushing volumes at the four locations noted above were compared against the base water age scenario to illustrate the difference. These are shown in Figure 7-44 through Figure 7-51 by Service Level for both the average water age and for the maximum water age. There were insignificant differences in water age in the Low Service Level and the High Service Level where auto-flushers were not simulated, so these are not shown. These figures indicate that the selected locations for the auto-flushers will have a positive impact on water age, though it could be less than modeled depending on the selected volumes being flushed. The smaller Service Levels of Cheney and Northwest show the most impact and it is more globally seen than in the larger Service Levels of Belmont and Southeast, where the improvements are more localized. In general, flushing at these rates could improve water age by a day or two.

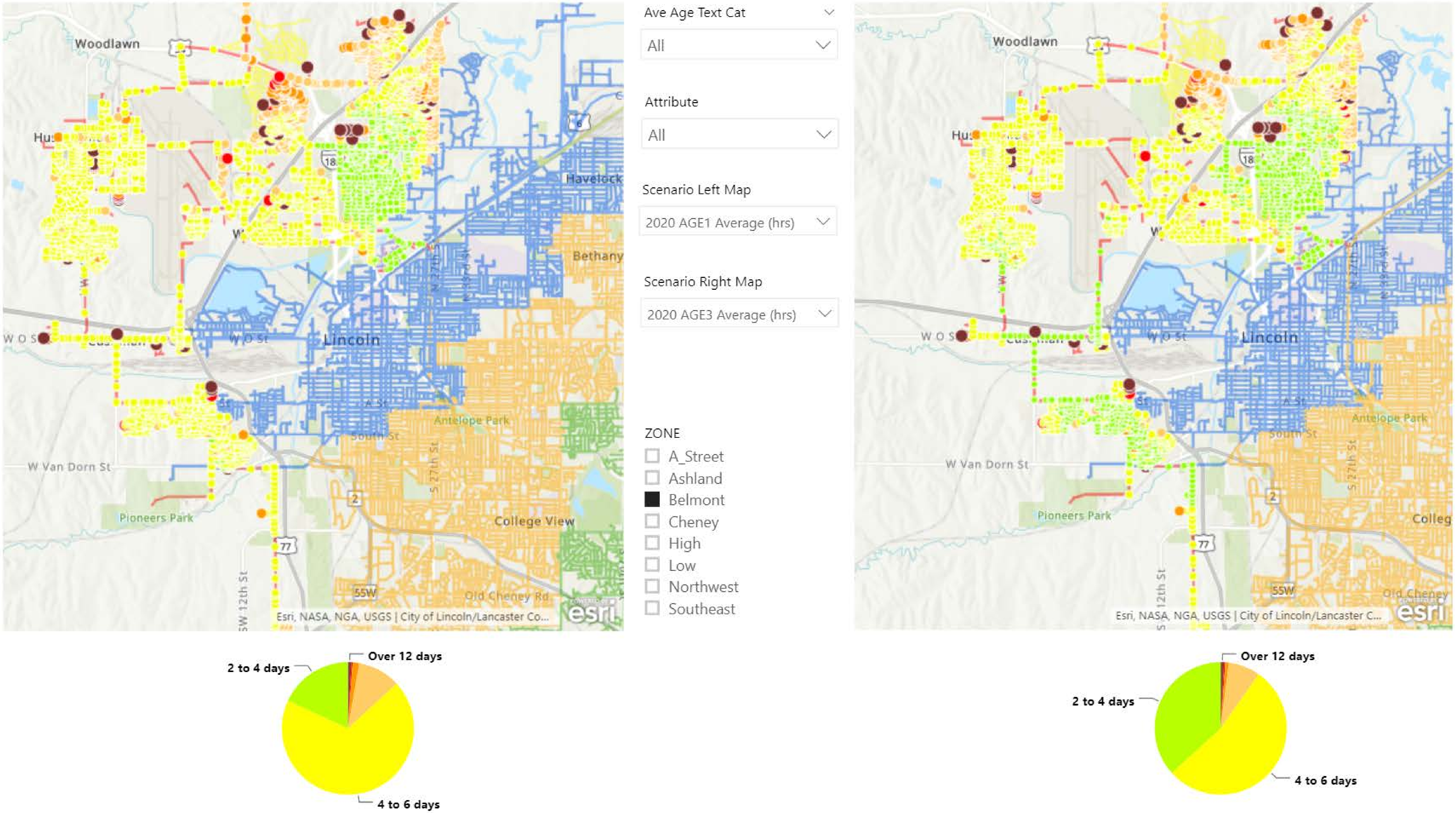


Figure 7-44 Average Water Age Comparison with Flushing – Belmont Service Level

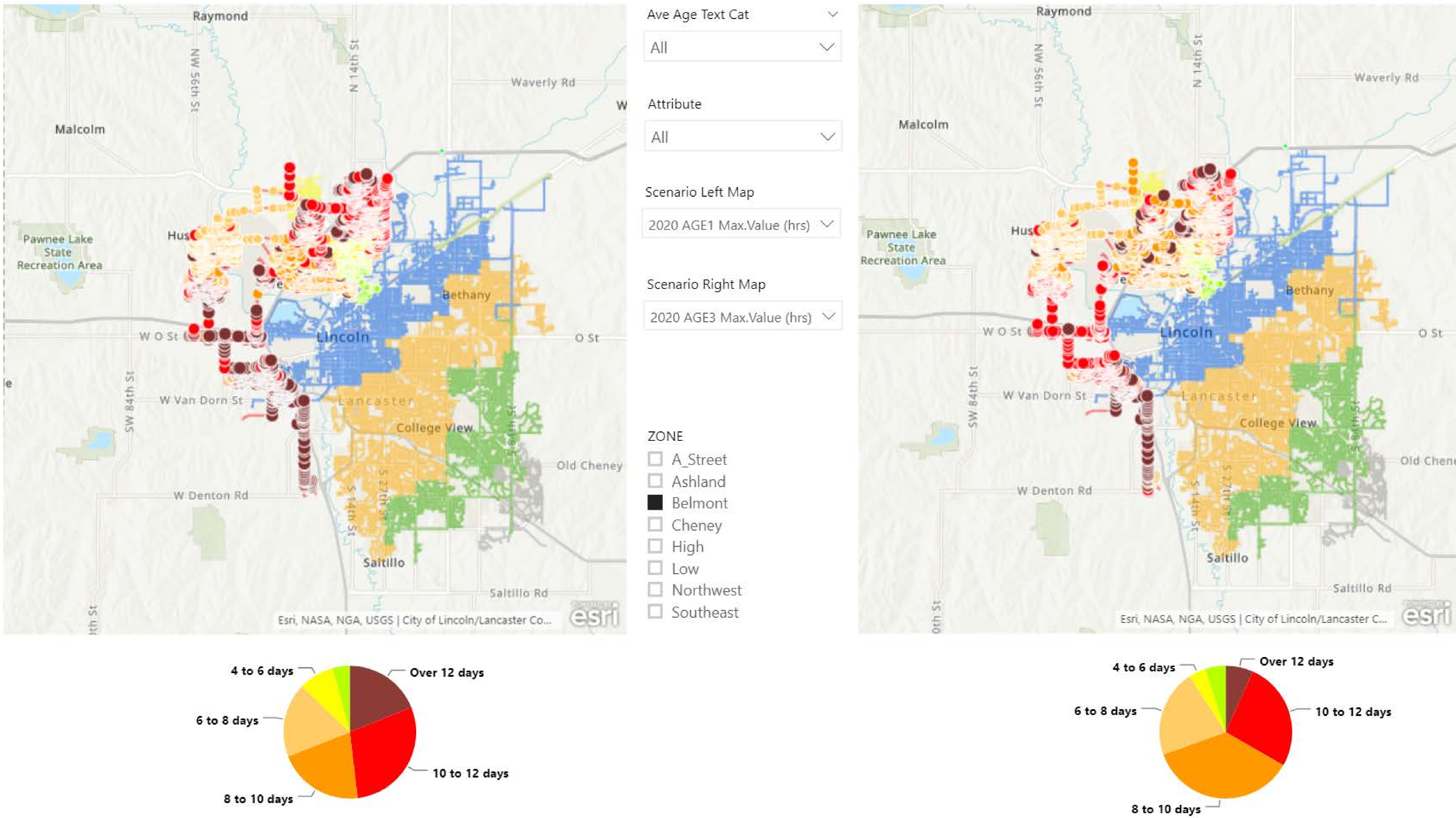


Figure 7-45 **Maximum Water Age Comparison with Flushing – Belmont Service Level**

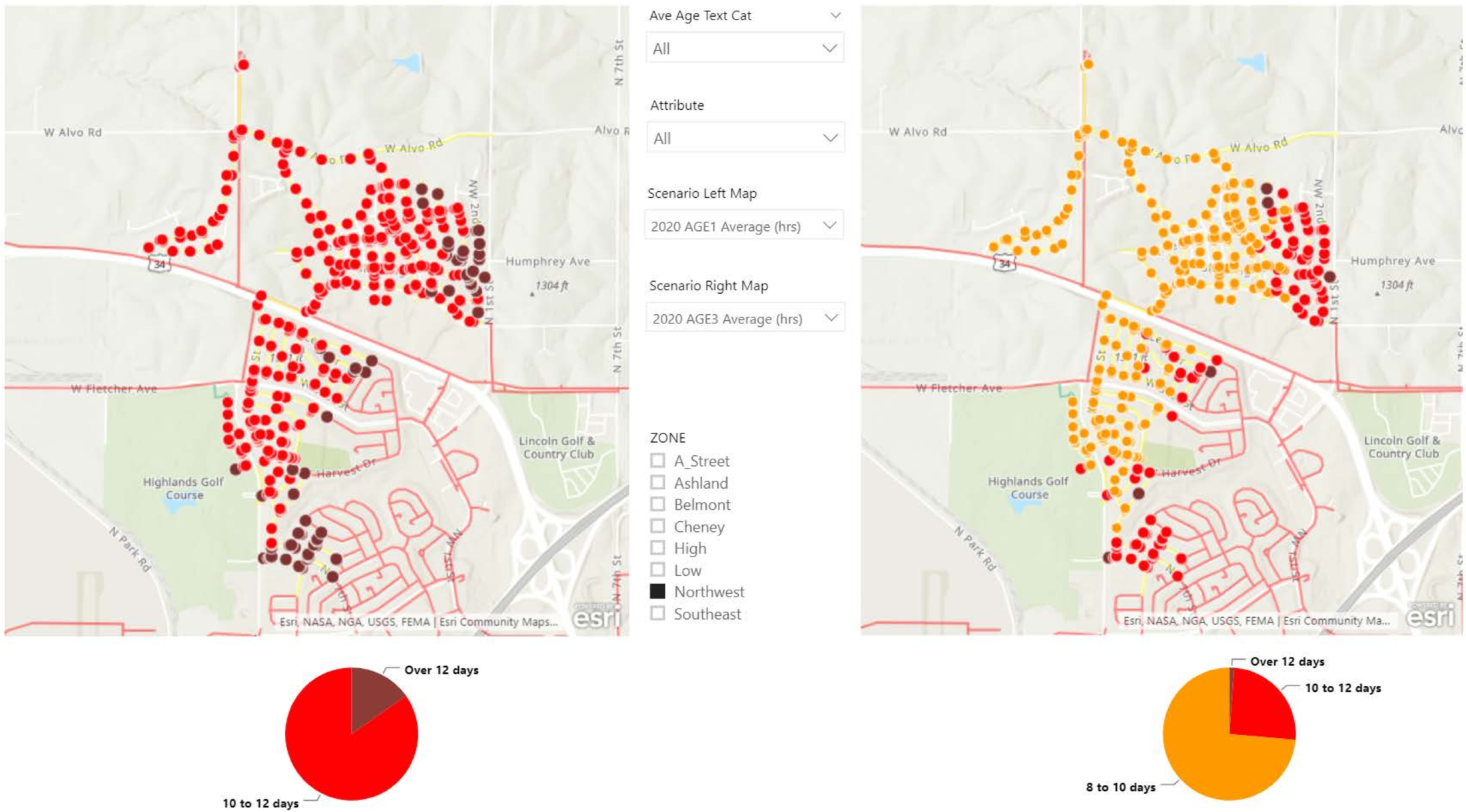


Figure 7-46 Average Water Age Comparison with Flushing – Northwest Service Level

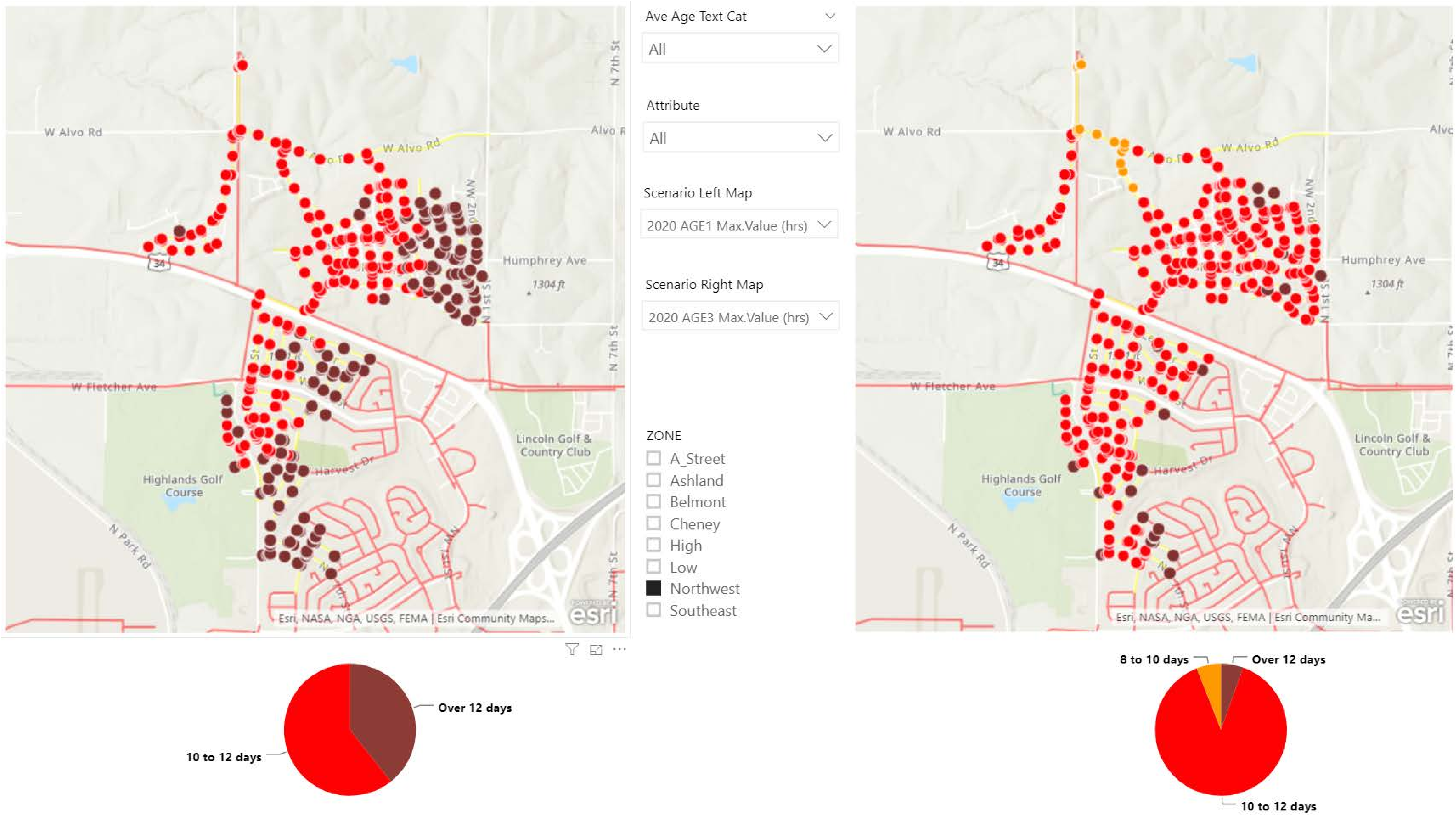


Figure 7-47 Maximum Water Age Comparison with Flushing – Northwest Service Level

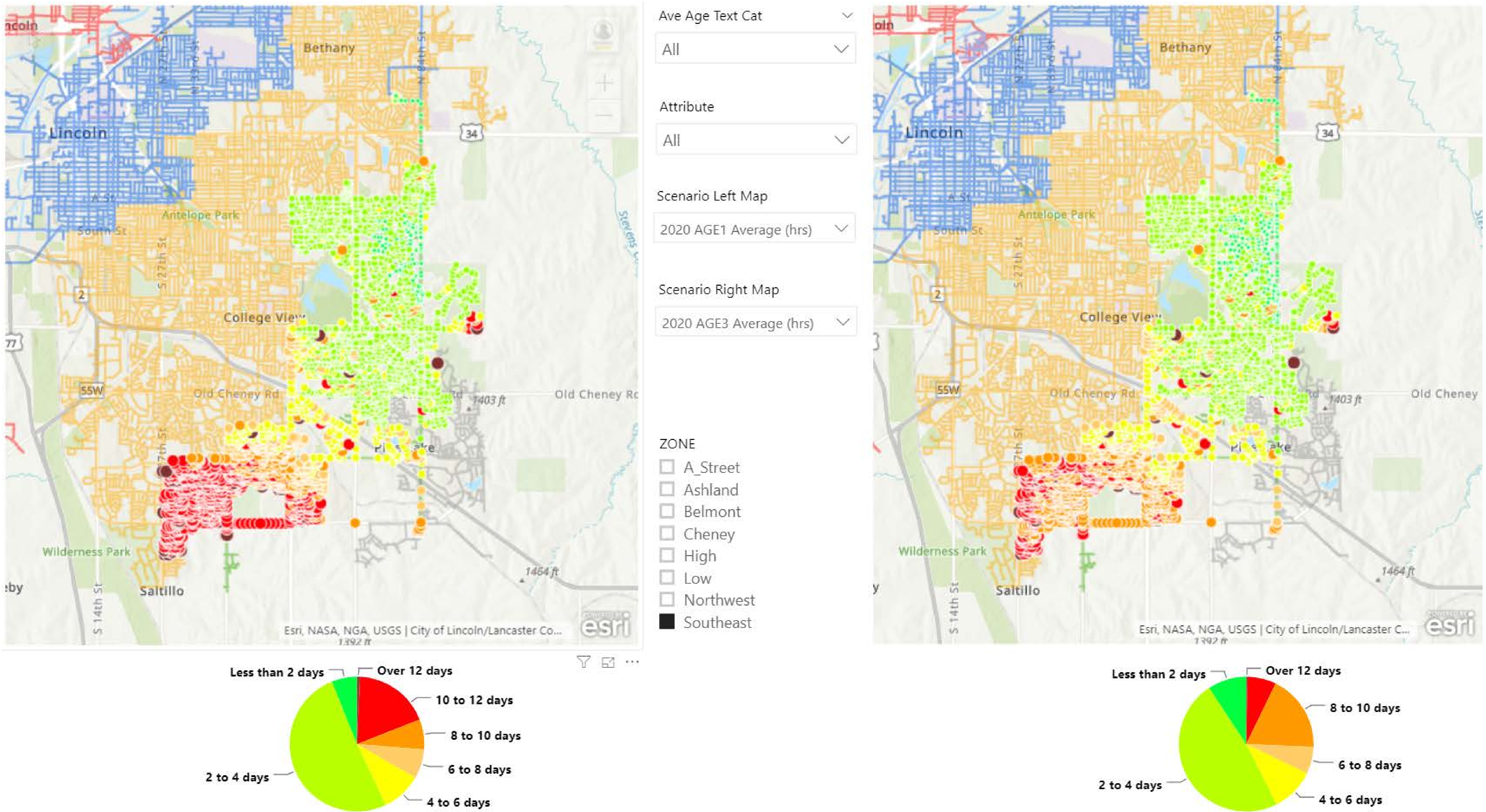


Figure 7-48 Average Water Age Comparison with Flushing – Southeast Service Level

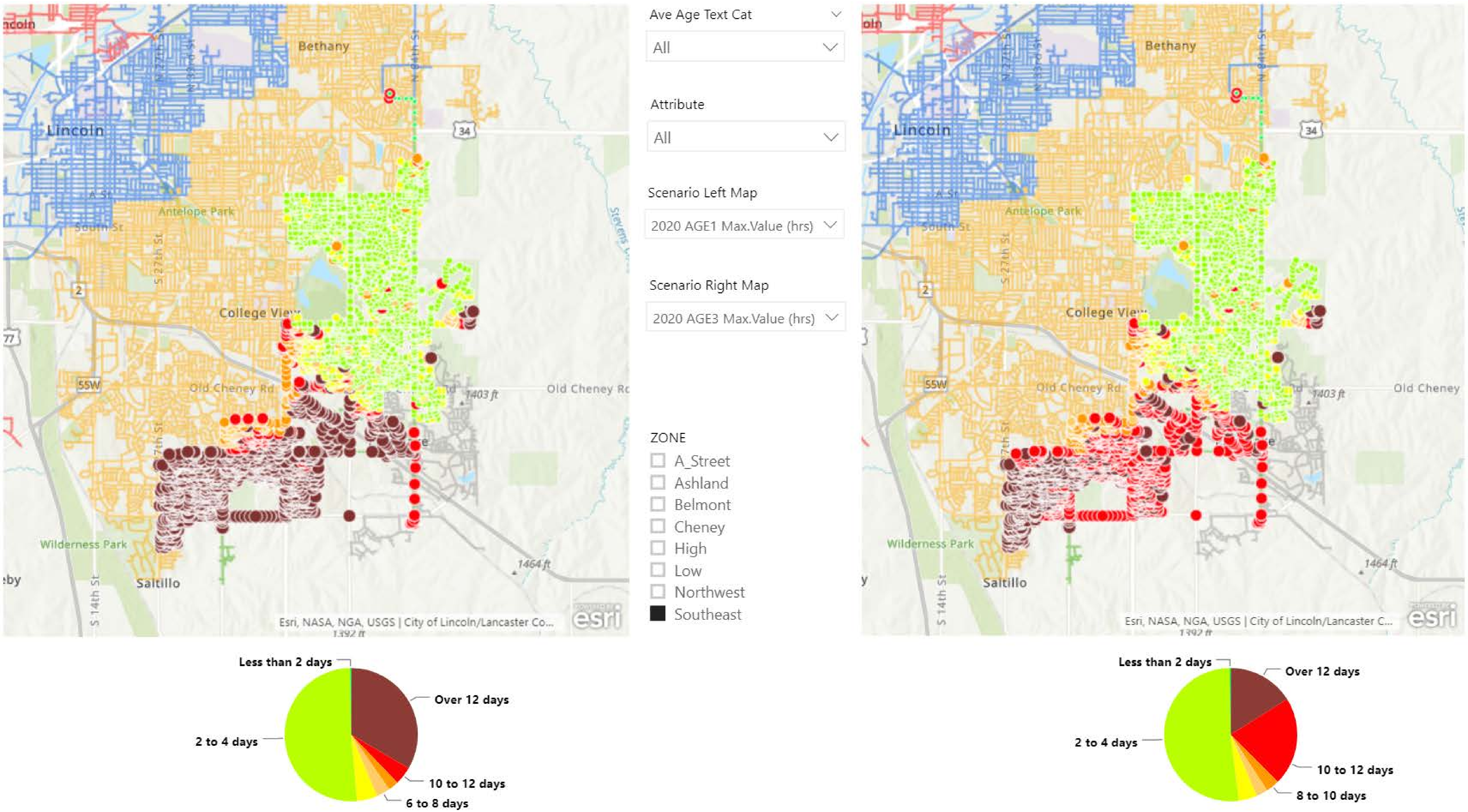


Figure 7-49 Maximum Water Age Comparison with Flushing – Southeast Service Level

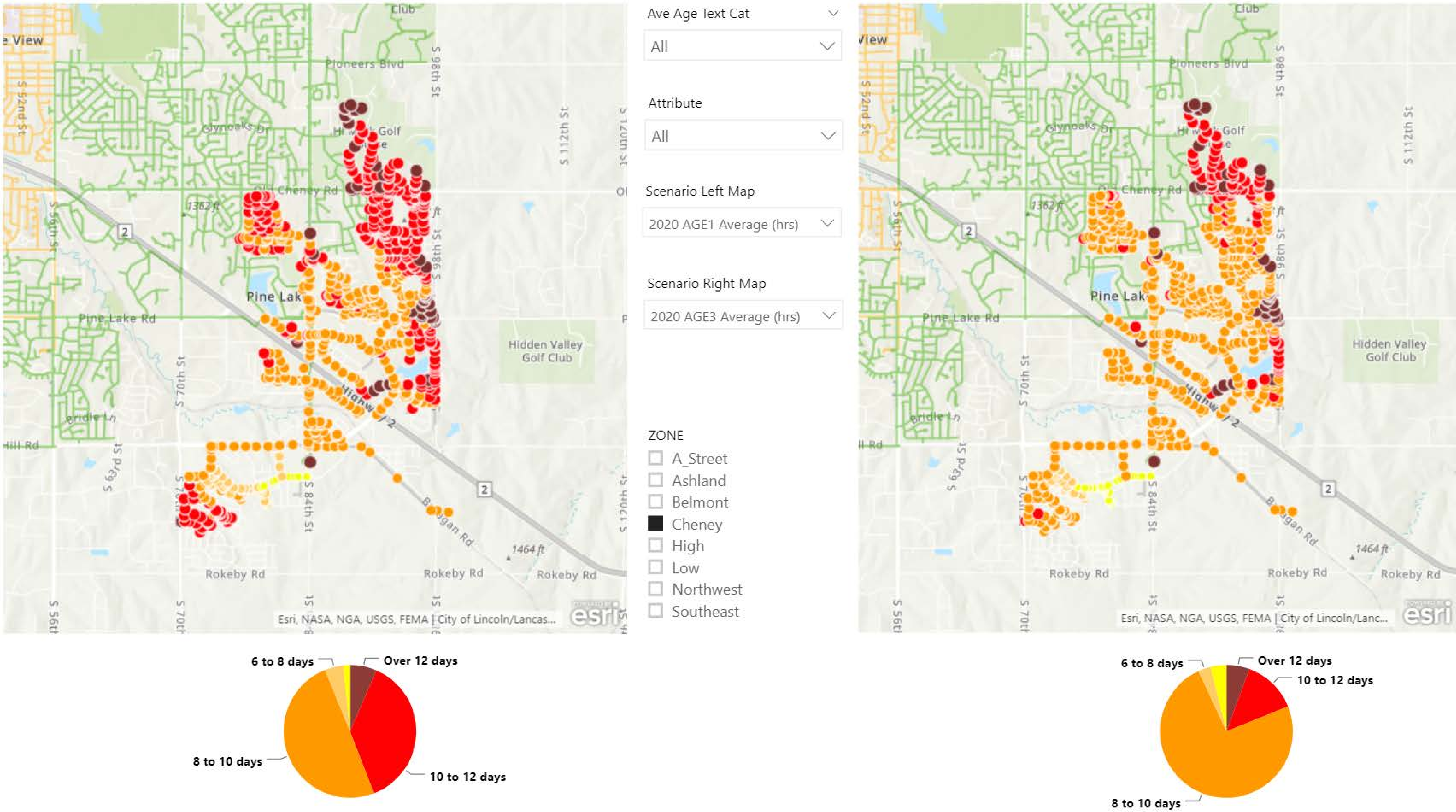


Figure 7-50 Average Water Age Comparison with Flushing – Cheney Service Level

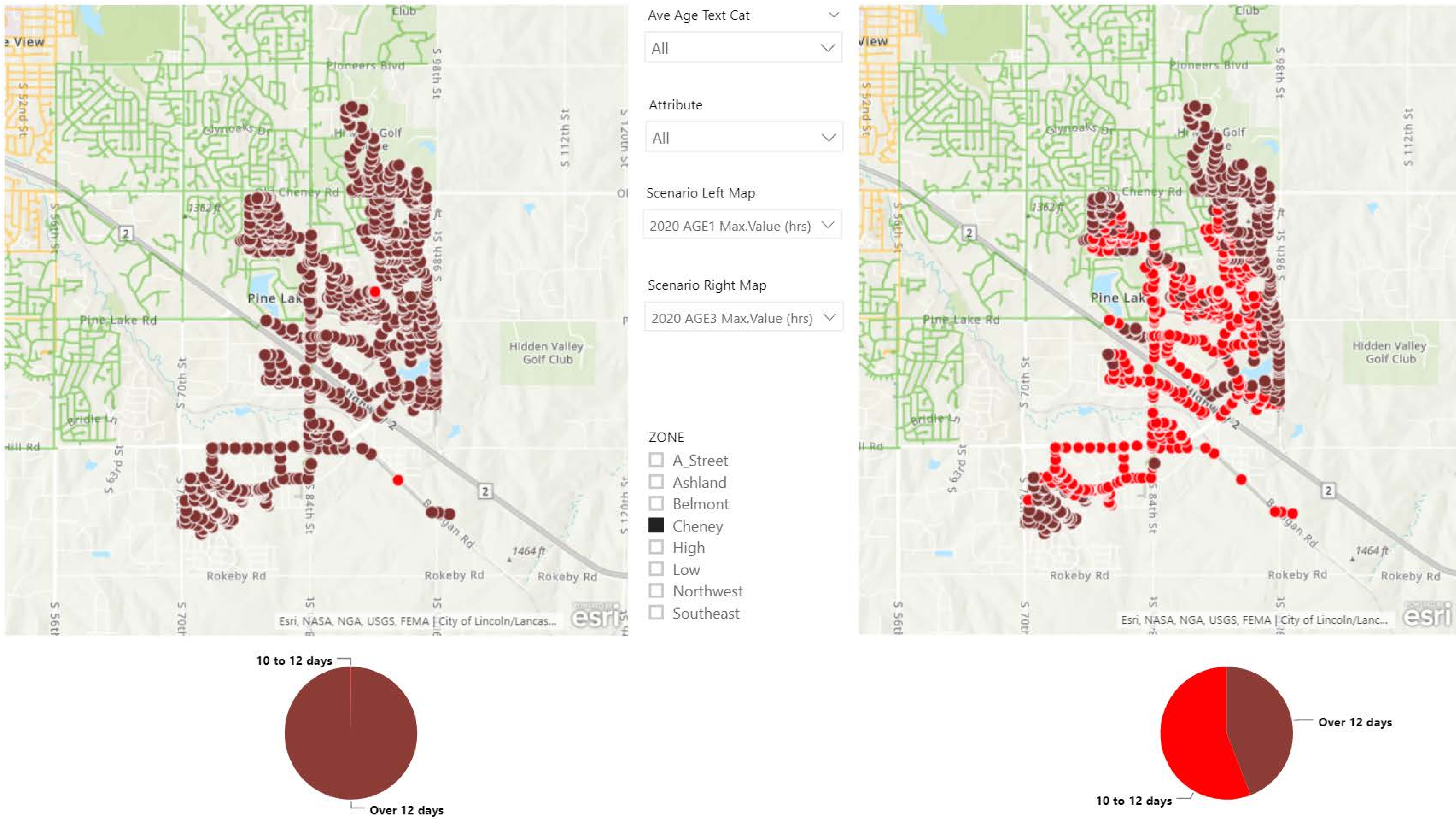


Figure 7-51 Maximum Water Age Comparison with Flushing – Cheney Service Level

7.8.2 Other Distribution Water Quality Improvement Alternatives

This section describes potential alternatives to further improve distribution system water quality, with emphasis on improving water quality in the areas surrounding the Northwest Service Level, Air Park, Cheney (southeast Lincoln), and southern parts of Southeast and High Service Levels. These areas generally encompass the southern and western areas of the distribution system, which are furthest from where water enters the distribution system on the northeast corner and historically have had difficulty with chlorine residual degradation.

7.8.2.1 Chloramine Booster Systems

Chloramine booster systems are being implemented throughout the United States to address degradation of disinfectant residual by increasing the chloramine residual in distribution system reservoirs. Chloramine booster systems are remote-operated systems that include chlorine and ammonia chemical storage and feed equipment, in-tank mechanical mixing equipment, online analyzers, and a programmable logic controller. Figure 7-52 provides a process schematic of the UGSI Monochlor® system, which is one of the equipment suppliers for chloramine booster systems.

Figure 7-52 Process Schematic of UGSI Monochlor® Chloramine Booster System

Chemicals are typically supplied in liquid form as sodium hypochlorite and liquid ammonium sulfate to minimize operational complexity and eliminate safety concerns associated with gaseous chlorine. A mechanical mixing system is installed within the reservoir to provide adequate dispersion of chemical and create a homogeneous mixture within the reservoir. Chlorine residual analyzers are then used to control the chlorine and ammonia feed rates, based on a target total chlorine residual and a chlorine-to-ammonia ratio of 5:1. Typically, a target total chlorine residual ranging from 1.5 mg/L to 2.5 mg/L is selected, and the PLC is used to control chlorine and ammonia feed rates based on breakpoint chemistry.

Given the challenges with maintaining the total chlorine residual in the far reaches of the distribution system (Belmont, Northwest, portions of Southeast and Cheney Service Levels), it is recommended that LWS implement chloramine booster systems to address residual degradation in those areas. The Yankee Hill Reservoir has been identified as the preferred location for implementation of a chloramine booster system to improve water quality in the Cheney Service Level, as well as along the southern reaches of the Southeast Service Level. To improve water quality in the Belmont and Northwest Service Levels, two locations have been proposed: Pioneers Reservoir and Northwest 12^{th} Street Reservoir (NW12^{th}). Water age and source trace modeling was conducted to simulate the potential improvement to water age (as an indicator of water quality) associated with implementing chloramine booster systems at the two locations. Preliminary findings from this analysis indicate that both locations provide water quality improvements with similar degrees of influence. Implementing a rechloramination system at Pioneers provides immediate water quality improvements to Air Park, whereas implementing at the NW12th location may not directly address Air Park based on the operating conditions and flow paths simulated by the model. Alternatively, the NW12th location is expected to provide water quality improvements over a broader area within Belmont and the Northwest Service Level, so additional consideration should be given to account for overall impact on water quality, ease of operations, and constructability at each location.

7.8.2.2 Biological Filtration

Biological filtration is often implemented downstream of ozonation as a means for reducing the biological organic matter (BOM) formed during ozonation. BOM is typically characterized and measured by the concentration of assimilable organic carbon (AOC) or biodegradable dissolved organic carbon (BDOC) present in a water sample. As noted in Chapter 5, the East Plant finished water has moderate to high concentrations of AOC. Between January 2001 and July 2009, the average and maximum concentration of AOC in the East Plant finished water was 154 µg/L and 350 µg/L, respectively.

Given that ozonation is utilized at the East Plant, biological filtration may present an opportunity to improve biological stability in the distribution system. Biological filtration is a process that reduces or eliminates the presence of chlorine residual upstream of filtration, allowing biological growth to occur on top of the filter media for enhanced removal of organics and inorganics. Biological filtration is often effective for decreasing biological activity in the distribution system, since the biomass that develops on the filter media utilizes biodegradable organic matter as a substrate.

A filter pilot study was conducted by Black & Veatch in Year 1995 to evaluate the performance of oxidant and polymer application for manganese removal. In the pilot study, a low chlorine residual was maintained over the filters to form a manganese oxide coating on the filters for enhanced removal of manganese. The low chlorine residual may have allowed for biological activity in deeper parts of the filter bed. Since the primary goal of this study was to evaluate the feasibility and effectiveness of manganese removal, additional pilot testing should be considered to evaluate the merits of biological filtration for removing AOC to reduce biological activity in the distribution system. Pilot testing should be conducted over a 9-month period overlapping with summer/fall months. Pilot testing should be done as a side-by-side comparison against current filter operations to evaluate the effectiveness of biological filtration on AOC removal relative to a baseline condition. The biological filtration column will require chemical feed to eliminate the chlorine residual upstream of the filter. Both columns should be monitored for water quality parameters related to biological stability and finished water quality, including AOC, TOC/DOC, turbidity, pH and alkalinity.

7.8.2.3 Sodium Chlorite

The addition of sodium chlorite at low doses has shown potential in some systems for controlling microbial regrowth in the distribution system. Chlorite is particularly effective at inactivating ammonia oxidizing bacteria, which makes it a potential alternative for nitrification control and distribution system water quality improvement. Systems that utilize sodium chlorite to control nitrification and bacterial regrowth typically feed at a dose of 0.30 mg/L, which is well below the MCL of 1.0 mg/L. Pilot testing should be conducted over a 9-month period to determine the feasibility of this treatment approach. Pilot testing usually consists of benchtop bioreactors and small-scale pipe systems with stagnation periods to simulate distribution water age. The bioreactors contain coupons, which can be extracted at various stages in the pilot study for DNA speciation to quantify the effectiveness of chlorite for reducing AOBs in biofilm. Two bioreactors would be required for this pilot study to compare the effectiveness of sodium chlorite against current operations (plant finished water).

7.8.2.4 Improvements to Tank Mixing

Poor tank mixing can lead to stratification within tanks, which can affect effluent water quality (particularly temperature and chlorine residual). Implementation of new mixing equipment in the distribution system reservoirs could potentially improve issues with chlorine residual management. Given the recent installation of tank mixing equipment in the South 56th St and Pioneers Reservoirs, it is recommended that LWS conduct a study to evaluate the effectiveness of existing tank mixing systems. The study would focus on the benefits of the existing mixing equipment and determine whether LWS should continue implementing in other tanks. The study could be conducted over a two-week period, monitoring the inlet and outlet water temperature and chlorine residual. In the first week of the study, the existing mixers would be in operation, and in the second week of the study, the mixers would be turned off. Flow rate in and out of the tank should be relatively constant over the two-week testing period in order to draw effective comparisons between the two operating conditions.

7.9 Distribution System Water Quality Summary & Recommendations

Based on a review of distribution water quality data, LWS has demonstrated effective management of DBPs and as a result, is on reduced monitoring for bromate, TTHM and HAA5. LWS has maintained a bromate RAA of less than 25 percent of the MCL since 2013. Similarly, the LRAA for TTHMs has consistently been less than 40 μg/L (50 percent of the MCL), and the LRAA for HAA5s has been maintained at less than 20 μg/L (33 percent of the MCL).

LWS is also on reduced monitoring for lead and copper, which requires LCR compliance data to be collected every three years. The 90th percentile values for lead and copper compliance monitoring in 2013, 2016 and 2019 have been below the action levels of 15 μg/L and 1300 μg/L, respectively. The proposed LCR revisions have proposed a new lead trigger level of 10 μg/L to prompt water systems to take proactive actions to reduce lead levels prior to exceeding the lead AL. Since 2004, the 90th percentile value for lead has been less than 5 μg/L, which is well below the proposed trigger level. Additionally, given the LWS's existing LSL inventory and replacement plan, LWS is well-positioned to comply with the potential requirements for implementing a publicly available LSL inventory and proactive, full LSL replacement program.

Between 2014 and 2017, LWS experienced challenges with nitrification between the months of August and December. Nitrification was characterized by rising water temperatures, loss of chlorine residual, increases in nitrite concentration, and in some locations, occurrences of HPCs. In 2018, LWS made significant improvements in distribution system water quality through various nitrification control measures, which resulted in increased chlorine residuals throughout the distribution system and reduced nitrite and nitrate concentrations. The nitrification control measures included increasing the chlorine residual at the POE, taking the East Plant out of service during peak nitrification season, and reducing water age in the distribution system by isolating and reducing operating volumes in reservoirs.

This resulted in considerable improvements to distribution system water quality in the High, Low and Southeast Service Levels. However, the areas surrounding Air Park, Northwest, Cheney, and southern parts of Southeast still had difficulty maintaining chlorine residuals greater than 0.5 mg/L at the distribution system monitoring sites. Additionally, alternative long-term solutions should be investigated, since taking the East Plant out of service limits the overall plant capacity and is not sustainable for future operations. Potential long-term solutions include:

- Chloramine booster systems within the distribution system.
- Improvements to tank mixing in distribution system reservoirs.
- Biological filtration at the East Plant.
- Sodium chlorite feed at the East and West Plant.

Given the continued challenges in Air Park, Northwest, Cheney and southern parts of Southeast Service Levels; a source trace analysis was conducted to identify optimal locations for chloramine booster systems. Source trace analysis is used to identify the percentage of water that comes from a given source, allowing for easier identification of areas that can provide a high impact on water quality. Based on the source trace analysis, it was determined that the most beneficial locations for installation of chloramine booster systems would be at Yankee Hill and Pioneers Reservoirs.

- **Yankee Hill** – Most of the water in the Cheney SL and southern parts of Southeast SL has passed through the Yankee Hill reservoir, making it an ideal location for rechloramination. It is also recommended that a PRV be installed around 84^{th} and South Street to allow rechloraminated water to be transferred to the High SL to address pockets with low chlorine residual.
- **Pioneers** –The source trace analysis found that during winter operations, over 80 percent of the water in Air Park has been pumped through Pioneers Pumping Station. With such a high proportion of water from Pioneers being delivered to these areas, there is a meaningful opportunity to improve distribution water quality through rechloramination at Pioneers.

For the time being, it is recommended that LWS continue with their current nitrification control measures, while other in-plant treatment and distribution system management alternatives are evaluated. The following alternatives for distribution system water quality improvements are recommended for further evaluation through pilot testing. Each of the proposed treatment alternatives should be compared with the plant's current operating conditions to establish a baseline and determine the preferred approach for nitrification control.

- **Biological filtration** – This alternative considers implementation of biological filtration in the East Plant to reduce the concentration of AOC, which is increased during the ozonation process. Reducing the AOC in the finished water will improve biological stability in the distribution system, which could allow for continued use of the East Plant during peak nitrification seasons.
- **Sodium chlorite** – This alternative considers feeding 0.3 mg/L of sodium chlorite to the plant finished water. Sodium chlorite is particularly effective at inactivating ammonia oxidizing bacteria and has proven to be effective for nitrification control for other utilities in the Midwest.
- **Improvements to Tank Mixing** – This alternative considers field-testing to evaluate the performance of existing distribution system tank mixing systems to provide guidance on future implementation strategies to reduce potential for stratification.

7.10 References

AWWA Research Foundation. (1994). *Ozone and Biological Treatment for DBP Control and Biological Stability.* Denver: AWWA Research Foundation and American Water Works Association.

AWWA Research Foundation. (1995). *Nitrification Occurrence and Control in Chloraminated Water Systems.* Denver: AWWA Research Foundation and American Water Works Association.

AWWA Research Foundation. (2000). *Evaluating Biological Regrowth in Distribution Systems.* Denver: AWWA Research Foundation and American Water Works Association.

AWWA Research Foundation. (2000). *Investigation of the Biological Stability of Water in Treatment Plants and Distribution Systems.* Denver: AWWA Research Foundation and American Water Works Association.

AWWA Research Foundation. (2002). *Influence of Distribution System Infrastructure on Bacterial Regrowth.* Denver: AWWA Research Foundation and American Water Works Association.

LSLR Collaborative. (2019). *Lead Service Line Replacement Collaborative*. Retrieved September 13, 2019, from https://www.lslr-collaborative.org/

8.0 Recommended Improvements

8.1 General

Based on the findings of the hydraulic analyses, the water quality analyses for both the treatment and distribution system, evaluation of the criticality of transmission mains, condition assessment of the water treatment plants, and overall system growth; a comprehensive capital improvements program was prepared. This comprehensive CIP includes budget costs and is staged and prioritized to identify improvements for additional capacity and reliability through Year 2032.

It should be recognized that the alignments shown for the recommended improvement mains are approximate locations. Specific street locations for the mains should be determined during the preliminary design. Improvement mains in undeveloped areas are subject to location change to conform to growth patterns and actual development. Factors that may accelerate or delay improvement mains include availability of right-of-way, scheduling of street improvements, and construction of other utilities. For residential service it is recommended that the City continue its general policy of installing minimum sizes of 16-inch mains on a 1-mile grid and 12-inch mains on half-section alignments, adjusted to accommodate local street patterns.

8.2 Cost Estimates

In every engineering study that develops a capital improvements program it is necessary to make estimates of the project costs required to implement the program. To that end, basic cost data must be obtained or developed for each type of construction and system components laid out in sufficient detail to permit determination of approximate project costs.

The total project cost necessary to complete a project consists of expenditures for land acquisition, construction costs, all necessary engineering services, contingencies, and such overhead items as legal, administrative and financing services. The various components of project costs are considered in the following paragraphs.

The cost of land acquisition is not included in the project costs presented in this report. In most cases, the construction of pipelines will not require purchase of private property or acquisition of easements. Pipeline routes, insofar as possible, follow public streets and roads. Although land acquisition is a significant activity that determines whether a project occurs, the cost of land acquisition is generally a small portion of the overall program cost. Relative to supply and treatment projects, all proposed facilities will be constructed on property currently owned by the City.

Construction costs cover the material, equipment, labor and services necessary to build the proposed project. Prices used in this study were obtained from a review of previous reports and pertinent sources of construction cost information. Construction costs used in this report are not intended to represent the lowest prices which may be achieved but rather are intended to represent a median of competitive prices submitted by responsible bidders.

Such factors as unexpected construction conditions, competitiveness of the bidding environment, the need for unforeseen mechanical and electrical equipment, and variations in final quantities are a few examples of items that can add to planning level estimates of project cost. To cover such contingencies, an allowance of 20 percent of the construction cost has been included.

Engineering services may include preliminary investigations and reports, site and route surveys, foundation explorations, preparation of design drawings and specifications, engineering services during construction, construction observation, construction surveying, sampling and testing, start-up services, and preparation of operation and maintenance manuals. Overhead charges cover such items as legal fees, financing fees, and administrative costs. The costs presented in this report include a 20 percent allowance for engineering services, legal, and administrative costs.

8.2.1 Basis of Costs

In considering the estimates presented in this report, it is important to realize that they are reported in Year 2019 dollars, and that future changes in the cost of materials, equipment and labor will cause comparable changes in project costs. A good indicator of changes in construction costs is the Engineering News-Record (ENR) Construction Cost Index (CCI), which is computed from prices of construction material and labor and based on a value of 100 in the Year 1913. Cost data in this report are based on an ENR CCI (20-city average) of 11326, which is the annual average value for Year 2019 (though November).

8.2.2 Pipelines

The *2014 Master Plan* used a construction cost of $7.50 per diameter-inch per lineal foot plus a 25 percent contingency for the basis of pipeline construction costs. A review of the ENR average annual CCI shows that the CCI has increased from 9547 in April 2014 to 11326 in late Year 2019. This represents an increase of 19 percent over that 5.5-year time period.

For this Year 2019 update, opinions of probable construction costs for main improvements in currently undeveloped areas are based upon a unit costs of ($7.50 x 1.2 = $9.00) per diameter-inch per lineal foot plus a 20 percent contingency ($9.00 x 1.2 = $10.80 per diameter-inch when including contingencies). Comparatively, within the last several years the City has been tracking total construction cost on their pipeline installations and determined that a good overall cost is $11.00 per diameter-inch. The probable project costs are calculated by adding a value equal to 20 percent of the total construction cost (including contingencies) for engineering, legal and administrative costs. The total value for probable project costs in currently undeveloped areas is therefore $13.20 ($11.00 x 1.2) per diameter-inch per lineal foot. This compares to the value of $11.50 per diameter-inch used in the *2014 Master Plan*. Installation of mains in urban areas is substantially more expensive due to cost associated with utilities, paving, scheduling, and site restoration.

For construction in fully developed and congested areas, a project cost of $19.00 per diameter-inch per lineal foot was used except for improvements relating to fire flow deficiencies which costs were defined on an individual basis depending on location and diameter. These unit costs and individual fire flow costs typically constitute an allowance for street removal and replacement as well as additional coordination with other utilities.

The costing utilized for main improvements as part of the capital improvement program are as shown in Table 8-1.

Table 8-1 Main Cost Utilized for Capital Improvement Program

Main Size (Inch)	Construction Cost ($/ft) Rural	Capital Cost ($/ft) Rural	Capital Cost ($/ft) Urban
8	$73	$106	$152
12	$110	$158	$228
16	$147	$211	$304
20	$183	$264	$380
24	$220	$317	$456
30	$275	$396	$570
36	$330	$475	$684
42	$385	$554	$798
48	$440	$634	$912
54	$495	$713	$1,026
60	$550	$792	$1,140

As indicated in previous Master Plans, it is recommended that the City continue its general policy of installing minimum sizes of 16-inch mains on a one-mile grid and 12-inch mains on half-section alignments, adjusted to accommodate local street patterns, for residential service. As a general guideline, the cost of one mile of 16-inch main would be about $1,100,000 and the cost of one mile of 24-inch main would be about $1,650,000. This report includes distribution main extensions which are necessary for development, but does not account for cost which are the responsibility of the developer.

8.2.3 Pumping

The total opinion of construction costs for a booster pumping station is highly dependent on the overall size of the facility. Specifically, the cost per gallon for small pumping stations is more expensive on a cost per gallon basis as the structure represents a higher overall percentage of the facility cost. The cost of the recently completed Yankee Hill Pumping Station (YHPS) provides a good reference for a small pumping station. The YHPS was designed to provide an initial total capacity of 7 mgd (4 mgd firm) with a buildout capacity of 24 mgd (18 mgd firm). The low bid for the project was $3,000,000 which represents an initial cost of $425,000/mgd based upon the initial total capacity. For reference only, other recent larger pumping stations constructed in the Midwest have been completed for approximately $250,000/mgd. The pumping facilities being completed for this study are smaller in size. Therefore, it is recommended that budgeting be based on a net cost of $400,000 per mgd of installed capacity plus 20 percent for contingency and 20 percent for engineering, legal, and administrative costs. These costs are based on typical Lincoln Water System pumping stations with permanent structure and sized for expansion. Therefore, the total probable project cost for new pumping stations is $610,000 per mgd.

The construction costs of installing a new pump in a pumping station which is designed for the addition of a pump, or for replacing a pump in an existing pumping station, are based on a unit cost of $60,000 per mgd of installed capacity. This cost includes the addition or replacement of electrical equipment. Probable project costs are calculated by adding a value equal to 20 percent of the total construction cost for engineering, legal and administrative costs. Therefore, the total probable project costs for capacity increases at existing pumping stations is about $86,400 per mgd. Budgeting for the inclusion of adjustable frequency drives, where applicable, was included as a separate cost.

8.2.4 Storage

The project cost for distribution system storage varies considerably, depending on such factors as type, material, capacity and support system. Estimated total unit project costs were developed for three types of facilities that are similar to those currently in service. The estimated total unit project costs include site work, reservoir foundation, the reservoir, site piping, controls and miscellaneous appurtenances.

Steel or pre-stressed concrete ground level reservoirs would be used primarily for larger reservoirs having capacities of over 2 MG and may be above-grade or buried below-grade. The construction cost of an above-ground ground level reservoir is based on a unit construction cost of $1.00 per gallon plus a 20 percent contingency. Probable project costs are calculated by adding a value equal to 20 percent of the total construction cost for engineering, legal and administrative costs. Therefore, the total probable project cost for an above-grade ground level reservoir is about $1.44 per gallon.

The construction cost of a buried below-grade reservoir is based on a unit construction cost of $1.50 per gallon plus a 20 percent contingency. Probable project costs are calculated by adding a value equal to 20 percent of the total construction cost for engineering, legal and administrative costs. Therefore, the total probable project cost for a buried below-grade ground level reservoir is about $2.16 per gallon.

The construction cost of elevated reservoirs is based on a unit cost of $2.00 per gallon plus a 20 percent contingency. Again, probable project costs are calculated by adding a value equal to 20 percent of the total construction cost for engineering, legal and administrative costs. Therefore, the total probable project cost for elevated reservoirs is $2.88 per gallon.

8.2.5 Pressure Reducing Valve Stations

Pressure reducing valve (PRV) stations transfer water from a higher service level to the next lower service level. It is assumed that the piping, valves, electrical and instrumentation components (including a flow meter) for a PRV station will be housed in a below-grade concrete vault structure. The construction cost for each PRV station is estimated to be $125,000. With the addition of contingencies, engineering, legal, and administration the total probable project cost for PRV stations is $180,000.

8.2.6 Pressure Monitoring Stations

Pressure monitoring stations are used monitor pressures in the distribution system in areas of interest. It is assumed that the electrical and instrumentation components will be housed in a small pre-packaged structure including a small enclosure located above grade, complete with necessary instrumentation. The total project cost for each pressure monitoring station is estimated to be $41,500.

8.3 Recommended Phased Improvements

The recommended phased improvements summarized in this report represent an update to the *2014 Master Plan*. Changes to the capital improvement program are a result of updated demand projections, which in turn impact the schedule for capital project implementation. Other changes to the CIP were predicated on additional input from the City, along with alternative analysis by Black & Veatch. The phases of the program are summarized below:

- The "Phase I – Immediate Improvements" are those that have been identified as higher priority as a result of their immediate need or as a result of currently anticipated development and correspond to FY 2019/2020 thru 2025/2026. These improvements are intended to meet the needs of the Comprehensive Plan – Tier 1 (Priority A) growth areas.
- Improvements recommended to meet FY 2026/2027 thru 2031/2032 demand conditions are referred to as "Phase II – 12-year Short-term Improvements". The Phase II improvements will extend service to the limits of the Tier I –Priority B area.
- Improvements beyond Year 2032 were not evaluated as part of this report, but cost for selective long-term improvements have been provided.

The recommended phased improvements for the distribution system are shown on Figure 8-1 at the end of this chapter and are described in the following sections. A detailed tabular summary of recommended Phase I and Phase II distribution improvements, along with recommended improvements for supply and treatment, is provided at the end of this chapter. Summary tables for fire flow improvements and distribution main extension projects are also provided.

8.3.1 Phase I – Immediate Improvements (by Year 2026)

Phase I recommended improvements will provide service to the limits of Tier I – Priority A development areas. The Phase I immediate improvements are recommended to correct existing deficiencies and provide a list of projects that should be implemented in the next six years of the LWS capital improvement program (CIP). Phase I also captures "carryover" projects from the previous master plan as well as CIP projects currently in the LWS six-year CIP.

The Phase I immediate improvements should be included in the six-year CIP, and include the following:

- Valve Replacement and Automation at 51st Street Reservoir and Pumping Station (IM-1). Required due to condition of existing valves and desire to automate valves to bypass 51st Street Pumping Station with approximately 14-15 mgd from the WTP straight to the Low Service Level. Benefits include increased operational flexibility, temporary shutdown of at least one 51st Street Reservoir, energy savings, and water age improvements. This improvement is proposed for Year 2020.
- NW 12th Street Pumping Station (IM-2). The existing Northwest 12th Street pumping station has adequate capacity but is reaching the end of its useful life as it was intended as a temporary pumping station. Therefore, we recommend replacement with a permanent facility similar to the recently completed Yankee Hill Pumping Station. Recommendations from the *2014 Master Plan* indicate the facility should have an initial firm capacity of 5 mgd (8 mgd total) and an ultimate firm capacity of 8 mgd (12 mgd total). Modeling conducted under this update indicates that the firm capacity in Year 2032 only needs to be 2.9 mgd. Therefore, the overall sizing should be revisited during preliminary design. This improvement is proposed for Year 2020.

- Vine Street Pumping Station East – Add Pump No. 8 w/ AFD (IM-3). The existing Vine Street Pumping Station East has two pumps, each with a capacity of 10.1 mgd. The *2014 Master Plan* recommended the replacement of one pump with a new 20 mgd pump with AFD capability. While this would add total capacity, it would not increase the overall firm capacity of the facility. Furthermore, the existing facility is already configured to accept a third pump. The existing two pumps are only approximately 20 years old, and do not have any known operational issues, therefore we would recommend that a pump be added versus replaced.

  Vine Street Pumping Station East was originally configured to be capable of providing a firm capacity of 40 mgd which would maximize the capacity of the 48-inch transmission main which extends from the Vine Street facilities to the Southeast Service Level. Additional hydraulic analysis indicated that a firm capacity of 30 mgd by the Year 2040 is warranted. Therefore, we recommend that a 20 mgd pump, with adjustable frequency drive, be added for increased conveyance to the Southeast Service Level. This improvement is proposed for Year 2020.

- Innovation Campus – Phase I – 16-inch main (IM-4). One of the improvements identified in the previous master plan to improve the level of service to the Innovation Campus was a new 16-inch main from approximately Highway 6 and North 14th Street to the Innovation Campus pipe network. This main is recommended due to the condition of the existing main serving the area which was installed in 1963 and is in poor condition. This improvement is proposed for Year 2020.

- I-80 & 56th Street - Pumping Station (IM-5). The area north of I-80 near the 56th Street interchange has been discussed from a master planning perspective as far back as the *2007 Facilities Master Plan Update*. To provide adequate pressure for ground elevations in this area, a new pumping station will be required as discussed in Chapter 6 of this report. Our recommendation based upon current demand projections for the area would be to construct a pumping station with an initial capacity of 10 mgd (6 mgd firm). This recommended size is predicated on the Belmont Loop (IM-6) being implemented as well to provide a firm capacity of 10 mgd to the area. Relative to ultimate sizing of the pumping station, the 2040 Comprehensive Plan and populations projection do not identify any additional growth for the area. Therefore, when the pumping station is designed it may be prudent to provide space for an additional future pump. This improvement is proposed for Year 2021.

- I-80 & 56th Street – Belmont Loop (IM-6). Currently all areas served by the Lincoln Water System have redundant means of water service. In order to provide the same level of reliability and redundancy we recommend that a main be constructed to connect the new service area to the Belmont Service Level. In addition to the redundancy provided, this connection also provides the added benefit of floating storage on the system for improved pumping operations and backup during power outages. This improvement is proposed for Year 2021.

- Arsenic/Atrazine Study and Preliminary Design (IM-7). As discussed in Chapter 5 – Water Treatment, the amount of arsenic and atrazine in the raw water are of concern, particularly as the City continues to become more reliant on horizontal collector well water. Specifically, the wells have exhibited a slightly higher background level of arsenic and are substantially impacted by atrazine in the Platte River due to their hydraulic connectivity. An arsenic and atrazine study will evaluate potential treatment alternatives through desktop analysis and bench-scale testing to develop a basis for conceptual life cycle costs. This project is proposed for Year 2021.

Initial concepts to reduce arsenic and atrazine levels include:

- Post-filter adsorption with arsenic-adsorbing media and ozone/hydrogen peroxide for atrazine removal.
- Ferric sulfate with sedimentation for arsenic removal and powdered activated carbon for atrazine removal.
- NF/RO for removal of both arsenic and atrazine.

- Distribution Water Quality Improvements – Phase I (IM-8). As discussed in Chapter 7 – Distribution Water Quality, installation of chloramine booster systems is recommended to increase the total chlorine residual in areas of the distribution system that suffer from chlorine residual degradation. Phase I of the distribution water quality improvements includes installation of a chloramine booster system at the Yankee Hill Reservoir to address water quality issues in the southern-most portions of the distribution system. Additionally, Phase I includes pilot testing to evaluate the effectiveness of in-plant treatment approaches, such as biological filtration and sodium chlorite feed, to reduce biological activity in the distribution system. The pilot study would likely be conducted over a 9-month period. These improvements are proposed for Year 2022.
- 16-inch Main on NW 56th St, "O" Street to Partridge Lane (IM-9). This improvement is required for redundancy and looping and to support future growth to the northwest area in the Belmont Service Level. Benefits include increased system resiliency and support of future development. It is also significant to note that this main improves the capability to serve the Belmont Service Level from the Pioneers Pumping Station. This improvement is proposed for Year 2022.
- Decommission Merrill Street Pumping Station (IM-10). Required due to small pumping station which is no longer used. The surge standpipe on the Merrill Street property must be kept in service for surge protection of the 36-inch transfer main. Benefits include less maintenance, reduced operational complexity and freed up resources. This project is proposed for Year 2022.
- Rehabilitate Eddy Current Drive Northeast Pumping Station No. 6 (IM-11). Pump No. 6 at the Northeast Pumping Station has been unusable for almost 20-years due to a faulty eddy current drive. A recent inspection was performed by the manufacturer which determined the drive is still viable but needs control components upgraded. The recommended plan for repair includes installation of a new EC-2000 controller along with a factory rehab and service of the drive and the motor since they have been sitting idle for a significant period of time. This improvement is proposed for Year 2022.
- West Water Treatment Plant Rehabilitation (IM-12). It has been almost 30 years since any major rehabilitation work was completed at the West Water Treatment Plant. A condition assessment was completed in conjunction with City staff and list of necessary rehabilitation items were developed to ensure the reliability and continued service of the facility. A detailed listing of improvements is provided in Appendix D, but in general includes coatings, selective valve replacement, crack repair, and HVAC updates. It has also been budgeted for the City to proceed with filter rehab on two filters with dual media so the City can determine the full-scale benefits of converting all the filters to dual media in the future. This improvement is proposed for Year 2022.

- 31st and Randolph Valve Vault Relocation to "A" Street (IM-13). Currently at 31st and Randolph Streets, there is a butterfly valve that is in a vault in the street which is used to control the transfer of water from the Vine Street Reservoir to "A" Street Reservoirs. This valve has been used to throttle gravity flows and the seat is worn and will not shut tight. Working conditions in the vault are less than desirable with no head room to work. LWS would like to see the valve replaced as a buried valve for shut off purposes and a new ball valve and mag meter should be placed near the "A" Street Reservoirs Nos. 8 and 9 (30th Street and Capital Parkway). The old vault at 31st Street and Randolph should be demolished. Benefits include increased transfer control, enhanced operations, and better access to the vault. This improvement is proposed for Year 2023.
- Water Treatment Plant South Pumping Station Pump No. 13 (IM-14). Additional WTP High Service Pumping will be required as growth occurs. A new Pump No. 13, with a rated capacity of 20 mgd and rated head of 350 feet (similar to the existing Pump No. 11 and Pump No. 12) should be installed by Year 2023. The addition of Pump No. 13 will fill all existing high service pumping bays at the WTP. Benefits include increased operational flexibility and high service pumping capacity into the transmission system. This improvement is proposed for Year 2023.
- 2023 Water Facilities Master Plan (IM-15). Systems planning is fundamental to management of the utility and ensures prudent investment on behalf of your ratepayers. LWS historically has completed master plans every six years, alternating between a comprehensive plan followed by a condensed update to the plan. The *2023 Water Facilities Master Plan* will be a comprehensive version. This project is proposed for Year 2023.
- Add AFD's at Pioneers Pumping Station (IM-16). The *2014 Master Plan* recommended the addition of AFD's at multiple pumping stations in the distribution system, with Pioneers Pumping Station having the highest priority. Although more expensive initially, AFD's were recommended instead of eddy current drives or discharge control valves due to their comparative inefficiencies. It was recommended that AFD's should be installed on all of the pumps in the pumping stations to maximum flexibility of operations and enable the smaller pumps to be used during lower flow conditions. At a minimum, it was recommended AFD's should be added to Pump Nos. 1 and 2 at Pioneers Pumping Station since they are operated most frequently to smooth out operations in the Belmont Service Level. Benefits include improved flow control, reduced cavitation issues, and controlled pressure variations during pump start-up and shut-down. This improvement is proposed for Year 2024.
- Pressure Monitoring Stations (IM-17). Additional monitoring locations are recommended to provide feedback on low and high pressures. Three low pressure and one high pressure monitoring locations are recommended in each improvement phase. The pressure monitoring locations can be built at any time as recommended in each phase; however, all four locations recommended in each phase should be constructed in one project for potential cost savings. Benefits are increased awareness of system performance, improved operations warning system, and additional data for hydraulic model calibration. These improvements are proposed for Year 2024.
- East Water Treatment Plant Rehabilitation (IM-18). With the exception of upgrades to the Ozone generation system, the East Water Treatment Plant has had minimal rehabilitation work since it was originally constructed almost 30 years ago. A condition assessment was completed in conjunction with City staff and list of necessary rehabilitation items were developed to ensure the reliability and continued service of the facility. A detailed listing of improvements is provided in Appendix D, but in general replacement of the ambient ozone

analyzers, coatings in the filter pipe gallery, and other miscellaneous rehabilitation items is recommended. This improvement is proposed for Year 2024.

- Decommission South 56th Street Pumping Station (IM-19). Required to take the pumping station out of service by removing pumps and VSDs (which should be salvaged, if possible). The facility itself must remain in order to maintain operation of the PRV at the facility which transfers water from Southwest Service Level to the High Service Level. Benefits include reduced maintenance efforts and reuse of the building as a potential maintenance storage facility. This project is proposed for Year 2024.
- Condition Assessment of 36-inch Cast Iron Main from 51st Street to "A" Street (IM-20). Historical knowledge of this main would indicate that in general the cast iron has not experienced corrosion of significance and any leakage is occurring at the joints. Therefore, to keep cost to a minimum, we recommend that the first step would be implementation of a technology to examine joint leakage. As indicated in Appendix C, we recommend that the SmartBall® technology be used for condition assessment of the 1930's 36-inch cast iron main. Dependent upon results, additional testing may be required, and may also trigger inspection of the segment from Ashland to 51st Street Pumping Station. It should be noted that Hydromax (Nautilus) also offers similar technology which could be considered at a slightly lower cost, but it is less proven in the US market. This project is proposed for Year 2024.
- Condition Assessment of 48-inch PCCP from Ashland to Northeast Pumping Station (IM-21). As discussed in Appendix C, the gold standard for inspection of PCCP mains is the use of electromagnetic (EM) inspection, but this is simply cost prohibitive given that the main has not shown any indications or degradation or leakage. Therefore, as a first step for determining condition we recommend inspection with the SmartBall® or Nautilus technology, which will determine if any leaks are occurring. Quotes received from the vendors indicate that SmartBall® will cost almost double that of Nautilus, so this may be a good opportunity to test Nautilus. If areas of concern are detected, future EM inspection may be warranted. This project is proposed for Year 2024.
- Condition Assessment of 48-inch PCCP from Northeast Pumping Station to Vine Street (IM-22). The 48-inch PCCP from Northeast Pumping Station to Vine Street Pumping Station is arguably one of the most critical assets in the distribution system. In addition, this pipe was installed during a time period (early 1970's) when the prestressing wires in PCCP were known to have brittle properties. Therefore, a more robust assessment is recommended, specifically EM inspection using a tethered robot. This main is approximately 5 miles in length, but our recommendation is to begin with inspection of approximately 16,000 feet. This can be accomplished through two entry points. This project is proposed for Year 2024.
- Water Treatment Plant Improvements for Arsenic Removal (IM-23). Additional studies are recommended to further develop the effectiveness and cost of these alternatives. For the purposes of establishing a placeholder for the CIP, costs were developed based upon post filter adsorption with AS media. This alternative includes the construction of a low lift pumping station downstream of the East WTP Filter Complex to convey water through adsorption vessels located in a new facility. Based upon high level assumptions for adsorption efficiency, and an effluent arsenic goal of 6 ppb, up to 18 adsorption vessels will be required. This improvement is proposed for Year 2025.

8.3.2 Phase II - Short-term Improvements (by Year 2032)

The Phase II short-term improvements will provide service to the limits of Tier 1 – Priority B development in the 12-year CIP, and include the following:

- Northwest Reservoir (2 MG) and Pipeline (ST-1). Required due to lack of redundancy to Northwest 12th Street Pumping Station and need for floating storage in the Northwest Service Level. This 2 MG storage facility should have an overflow elevation of 1460 and be located near the existing NW 12th Street Reservoir to reduce transmission main cost and minimize water quality impacts. Benefits include smoother operation of Northwest 12th Street Pumping Station, service level supply redundancy, emergency storage for multiple service levels, and more uniform service level pressures. This improvement is proposed for Year 2026.
- Belmont to Low PRV Station – "O" Street and N 12th (ST-2). Required due to fire flow deficiencies at the edge of the Low Service Level in this vicinity. Benefits include additional supply during high flow and fire flow periods and reduced estimated fire flow deficiencies. This improvement is proposed for Year 2026.
- Decommission NW 12th Street Pumping Station (ST-3). Required due to deteriorating condition of existing Northwest 12th Street Pumping Station and its scheduled replacement. Benefits include reduced maintenance and addition of permanent pumping facilities for the Northwest Service Level. This project is proposed for Year 2027.
- Decommission Cheney Pumping Station (ST-4). Required due to deteriorating condition of existing Cheney Pumping Station. Benefits include reduced maintenance and addition of permanent pumping facilities for the Cheney Service Level. This project is proposed for Year 2027.
- Yankee Hill Pumping Station – Add Pump No. 4 (ST-5). Upon decommissioning of the Cheney Pumping Station, the firm capacity of the Yankee Hill Pumping Station will need to increase. In order to maintain firm pumping capacity to the Cheney Service Level, we recommend that a 6.0 mgd pump be installed into the available pump slot at that time. This improvement is proposed for Year 2027.
- Southeast to High PRV Station at Southeast Pumping Station (ST-6). The existing PRV at the South 56th Street Pumping Station is commonly used to transfer water from the Southeast Service Level to the High Service Level. Furthermore, it is an essential asset for operations when the Yankee Hill Reservoir is out of service for cleaning and maintenance. In order to provide redundancy and increased operational control between the Southeast Service Level and High Service Level, the *2014 Master Plan* recommended a similar PRV be added near the Southeast Pumping Station. Benefits include increased operational flexibility and increased ability to take the South 56th or Southeast Reservoirs offline for maintenance. This improvement is proposed for Year 2027.
- Innovation Campus – Phase II – 12-inch (ST-7). The Phase II connection to Innovation Campus involves extending a 12-inch main from North 20th and Cornhusker to the campus as shown on Figure 8-1. This main extension would have a length of 4,600 feet and would provide redundant supply to the campus. This improvement is proposed for Year 2027.
- Distribution Water Quality Improvements – Phase II (ST-8). The second phase for improvement to the Distribution Water Quality is implementation of a chloramine booster system at Pioneers Reservoir or the Northwest 12th Street Reservoir to address chlorine residual degradation in the western-most portions of the distribution system. Source trace

modeling has indicated installation at Pioneers would be more impactful. However, this would require running Pioneers Pumping Station at high rates as compared to the Belmont Pumping Station. These improvement are proposed for Year 2028.

- Adams Street Reservoir and Pipeline (ST-9). Required to support growing demands in High Service Level. Benefits include increased storage to support development and operational flexibility from Vine Street and "A" Street Pumping Stations into the High Service Level. This improvement is proposed for Year 2030.
- 54-inch Main from Northeast Pumping Station to 88th and Holdrege (ST-10). The *2014 Master Plan* recommended this pipeline be constructed by Year 2025. Based solely on the total hydraulic capacity of the system, this improvement could be deferred beyond Year 2032. However, it does provide desirable benefits beyond hydraulic capacity which make it recommended in the short term. Specifically, this main will provide increased reliability/redundancy to a critical area in the system, increase operational flexibility, and will provide the ability to avoid re-pumping at Northeast Pumping Station. Results of the condition assessment for the existing 54-inch main (IM-22) should be monitored closely as they could impact timing of this improvement. This improvement is proposed for Year 2032.

8.3.3 Other Improvements – Long-Term

Some other significant projects, evaluated as part of this study but which fall outside the 12-Year CIP, are summarized below. It should be noted that it may be desirable for these projects to be implemented sooner in order improve the resiliency of the system.

- 36-inch Transfer Main from Vine Street Reservoir to "A" Street Reservoir (LT-1). The addition of another transfer main from Vine Street Reservoir to the "A" Street Reservoir is an improvement recommended in the previous master plan which the City requested Black & Veatch specifically analyze. Operational cost analyses performed by LWS have determined that water can be delivered more efficiently to "A" Street from Vine Street Reservoir, as opposed to being conveyed with the transfer pumps at 51st Street Pumping Station. The cost programmed into the previous master plan was over $16 million. Based on energy use, it would be very difficult to justify implementation of this project. If it is determined that the 36-inch from 51st Street to Vine is compromised, it may be justified from a redundancy standpoint. This improvement is proposed for Year 2033.
- Horizontal Collector Well No. 5 – Site 7 (LT-2). As noted in Chapter 4, additional raw water supply is needed no later than Year 2035. It may be desirable to advance this improvement in the CIP to provide greater drought resilience to the system. This improvement is proposed for Year 2033.
- Water Treatment Plant Expansion – Ozone and East Filters (LT-3). A comparison of expansion between the East Water Treatment Plant and West Water Treatment Plant was performed and summarized in Appendix D. Based upon this comparison, it is recommended that an expansion of 30 mgd to the East Water Treatment Plant be completed by Year 2037. This improvement should proceed by Year 2034.
- Horizontal Collector Well No. 6 – South Site (LT-4). Construction of the sixth horizontal collector well is recommended no later than Year 2048. Again, if greater drought resiliency is desired, this project should be advanced in the CIP.

- Long-Term Water Supply. As recommended in the *2014 Master Plan*, the City continues to set aside funding for their next source of water supply which will be necessary no later than Year 2048 based upon current supply and demand projects.
- Lead Service Line Replacement Program. LWS has been developing a LSL inventory as part of their LSL Identification Program. Through these efforts, LWS has identified approximately 4,000 potential LSLs for replacement. LWS is developing a proactive LSL replacement plan in advance of the proposed LCR revisions. The City currently has $24,500,000 budgeted for their LSL Replacement Program.

Table 8-2 Recommended Improvements – Phase I and Phase II

Recommended Improvements - Phase I and Phase II							
Year	CIP Tag	Description	Improvement Type	Construction Cost	Contingency	Engineering, Legal, Admin (ELA)	Total Capital Cost (FY 2020)
Phase I – Immediate Improvements							
2020	IM-1	Valve Replacement and Automation at 51st Street PS	Facility	$263,889	$52,778	$63,333	$380,000
2020	IM-2	NW 12th Street Pumping Station	Pumping	$3,200,000	$640,000	$768,000	$4,608,000
2020	IM-3	Vine Street Pumping Station East - Add Pump No. 8 w/ AFD	Pumping	$1,637,000	$327,400	$392,880	$2,357,000
2020	IM-4	Innovation Campus - Phase 1 - 16-inch Main	Distribution	$814,044	$162,809	$195,371	$1,172,000
2021	IM-5	I-80 & 56th Street Pumping Station - Supply Main and PS	Pumping	$4,000,000	$800,000	$960,000	$5,760,000
2021	IM-6	I-80 & 56th Street Pumping Station - Belmont Loop	Distribution	$3,894,000	$778,800	$934,560	$5,607,000
2021	IM-7	Arsenic/Atrazine Study and Preliminary Design	Treatment	$0	$0	$250,000	$250,000
2022	IM-8	Distribution Water Quality Improvements - Phase 1	Distribution	$2,092,390	$418,478	$502,174	$3,013,000
2022	IM-9	16-inch Main on NW 56th Street, "O" St. to Partridge Lane	Distribution	$996,600	$199,920	$239,904	$1,439,000
2022	IM-10	Decommission Merrill Street Pumping Station	Pumping	$212,739	$42,548	$51,057	$306,000
2022	IM-11	Rehabilitate Eddy Current Drive - Northeast #6	Pumping	$84,267	$16,853	$20,224	$121,000
2022	IM-12	West Water Treatment Plant Rehabilitation	Treatment	$1,587,040	$317,408	$380,890	$2,285,000

Recommended Improvements - Phase I and Phase II							
Year	CIP Tag	Description	Improvement Type	Construction Cost	Contingency	Engineering, Legal, Admin (ELA)	Total Capital Cost (FY 2020)
2023	IM-13	31st and Randolph Valve Vault Relocation to "A" street	Facility	$237,999	$47,600	$57,120	$343,000
2023	IM-14	Add 20.9 mgd WTP South Pumping Station Pump No. 13	Pumping	$1,254,000	$250,800	$300,960	$1,806,000
2023	IM-15	2023 Master Plan	System	$0	$0	$1,000,000	$1,000,000
2024	IM-16	Add AFD's at Pioneers Pumping Station	Pumping	$163,645	$32,729	$39,275	$236,000
2024	IM-17	Pressure Monitoring Stations	Distribution	$114,629	$22,926	$27,511	$165,000
2024	IM-18	East Plant Overall Rehab	Treatment	$464,800	$92,960	$111,552	$669,000
2024	IM-19	Decommission South 56th Street PS	Pumping	$208,333	$41,667	$50,000	$300,000
2024	IM-20	Condition Assessment of 36-inch Cast Iron from 51st to A Street	Condition	$155,000	$31,000	$37,200	$223,000
2024	IM-21	Condition Assessment of 48-inch PCCP from Ashland to NE	Condition	$215,000	$43,000	$51,600	$310,000
2024	IM-22	Condition Assessment of 54-inch PCCP from Northeast to Vine	Condition	$327,000	$65,400	$78,480	$471,000
2025	IM-23	Arsenic Treatment - Adsorber	Treatment	$28,267,008	$5,653,402	$6,784,082	$40,704,000
Phase II – Short-Term Improvements							
2026	ST-1	Northwest Reservoir (2 MG) and Pipeline	Storage	$4,109,920	$821,984	$986,381	$5,918,000
2026	ST-2	Belmont to Low PRV Station ("O" Street and N 12th Street)	Distribution	$125,000	$25,000	$30,000	$180,000
2027	ST-3	Decommission NW 12th Street Pumping Station	Pumping	$222,430	$44,486	$53,383	$320,000

Recommended Improvements - Phase I and Phase II							
Year	**CIP Tag**	**Description**	**Improvement Type**	**Construction Cost**	**Contingency**	**Engineering, Legal, Admin (ELA)**	**Total Capital Cost (FY 2020)**
2027	ST-4	Decommission Cheney Pumping Station	Pumping	$217,186	$43,437	$52,125	$313,000
2027	ST-5	Yankee Hill Pumping Station - Add 6 mgd Pump	Pumping	$360,000	$72,000	$86,400	$518,000
2027	ST-6	PRV Southeast SL to High SL - Vault near Southeast PS	Distribution	$125,000	$25,000	$30,000	$180,000
2027	ST-7	Innovation Campus - Phase 2 - 12-inch Main	Distribution	$506,000	$101,200	$121,440	$729,000
2028	ST-8	Distribution Water Quality Improvements - Phase 2 (Pioneers WQ)	Distribution	$960,000	$192,000	$230,400	$1,382,000
2030	ST-9	Adams Street Reservoir and Pipelines for HSL (5 MG)	Storage	$8,322,600	$1,664,520	$1,997,424	$11,985,000
2032	ST-10	54-inch Main from Northeast PS to 88th and Holdrege	Transmission	$18,538,000	$3,707,600	$4,449,120	$26,695,000
Long-term Improvements							
2033	LT-1	36-inch Transfer Main from Vine Street Reservoir to A Street Reser	Transmission	$12,165,429	$2,433,086	$2,919,703	$17,518,000
2033	LT-2	Horizontal Collector Well No. 5 - Site 7	Supply	$8,427,500	$1,685,500	$2,022,600	$12,136,000
2034	LT-3	Water Treatment Plant Expansion - Ozone and East Filters	Treatment	$17,225,040	$3,445,008	$4,134,010	$24,804,000
2041	LT-4	Horizontal Collector Well No. 6 - South Site	Supply	$8,278,688	$1,655,738	$1,986,885	$11,921,000

Recommended Improvements - Phase I and Phase II							
Year	**CIP Tag**	**Description**	**Improvement Type**	**Construction Cost**	**Contingency**	**Engineering, Legal, Admin (ELA)**	**Total Capital Cost (FY 2020)**
		New Source of Supply Reserve Fund	Supply	$22,000,000	$0	$0	$22,000,000
		Lead Service Line Replacement Program	Distribution	$24,500,000	$4,900,000	$5,880,000	$35,280,000
			Subtotal Construction Cost	**$176,275,568**		**Total Capital Cost**	**$245,405,000**

Table 8-3 Immediate Fire Flow Improvements

Fire Flow Improvements - Immediate						
CIP Tag	Description	Service Level	Length	Diameter	Unit Cost ($/ft)	Total Capital Costs
FF-1	Connection on Normal Blvd between S 62nd St and Park Crest Ct	Southeast	300	8	$152	$45,600
FF-2	Connection between Calvert St and S 58th St	High	500	8	$152	$76,000
FF-3	Connection on Kearney Ave between N 70th St and N 71st St	Low	400	8	$152	$60,800
FF-4	Connection on N 68th St between Seward Ave and Colfax Ave	Low	400	8	$152	$60,800
FF-5	Connection on N 66th St between Colfax Ave and Freemont St	Low	700	8	$152	$106,400
FF-6	Connection on N 38th St between Cleveland Ave and Madison	Low	400	8	$152	$60,800
FF-7	Connection on S 16th St between Woodsview St and Calvert St	High	2,100	8	$152	$319,200
FF-8	Connection on N 29th St between Q St and O St	Low	800	8	$152	$121,600
FF-9	Connection on N 31st St between P St and O St	Low	400	8	$152	$60,800
FF-10	Connection on NW 57th St between W Thatcher Ln and W Aurora St	Belmont	800	8	$152	$121,600
FF-11	Looping on 53rd, North of Huntington	Low	500	8	$152	$76,000
Total Capital Costs						**$1,110,000**

Table 8-4 Distribution Main Extensions – Immediate

Distribution Main Extensions - Immediate							
CIP Tag	LWS Tag	Description	Service Level	Length	Diameter	Unit Cost ($/ft)	Total Capital Costs
ExtI-1	P-1877	Holdrege St, N 98th St to Cessna Ln	Low	2,800	16	$211	$590,800
ExtI-2	FP-117	N 98th St, O St to the north	Belmont	300	16	$211	$63,300
ExtI-3	FP-196	Alvo Rd, N 48th St to the east	Northwest	500	12	$158	$79,000
ExtI-4	P-1967	Alvo Rd, N 14th St to N 16th St	High	1,100	12	$158	$173,800
ExtI-5	FP-133	Rokeby Rd, S 40th St to the east	Belmont	2,400	16	$211	$506,400
ExtI-6	FP-143	Rokeby Rd, S 27th St to the east	Belmont	1,700	16	$211	$358,700
ExtI-7	FP-173	S 1st St towards W Folsom	High	800	12	$158	$126,400
ExtI-8	P-2104	Arbor Rd, N 40th St to N 56th St	Belmont	4,200	24	$317	$1,331,400
ExtI-9	P-1996	NW 48th St, W Fletcher Ave to W Cuming St	High	2,900	16	$211	$611,900
ExtI-10	P-1997	W Cuming St extended past NW 53rd St	Low	700	16	$211	$147,700
ExtI-11	P-1998	W Superior St extended north	Belmont	2,200	16	$211	$464,200
ExtI-12	P-2034	W Pleasant Hill Rd (extended), SW 12th St to S Folsom St	Southeast	2,700	12	$158	$426,600
ExtI-13	P-2037	W Denton Rd, S Folsom St to S 1st St	High	2,000	16	$211	$422,000
ExtI-14	P-2102	N 48th St (extended)	Belmont	1,300	12	$158	$205,400
ExtI-15	FP-116	O St, N 98th to the east	Belmont	2,600	24	$317	$824,200
ExtI-16	FP-120	E Avon Ln, N 86th St to Linwood Ln	Belmont	2,700	12	$158	$426,600
ExtI-17	FP-157	S 1st St	Low	1,900	12	$158	$300,200
ExtI-18	FP-187	W Alvo Rd, east to NW 12th St	High	2,600	12	$158	$410,800
ExtI-19	FP-188	W Alvo Rd, east to N 14th St	Belmont	1,500	12	$158	$237,000
ExtI-20	FP-211	W Dan Dorn St, SW 33rd ST to S Coddington Ave	Belmont	5,500	16	$211	$1,160,500

Distribution Main Extensions - Immediate							
CIP Tag	LWS Tag	Description	Service Level	Length	Diameter	Unit Cost ($/ft)	Total Capital Costs
ExtI-21	FP-213	Havelock Ave, N 70th St to N 84th St	Belmont	5,300	16	$211	$1,118,300
ExtI-22	FP-219	SW 33rd St, south to W Van Dorn St	Low	2,700	12	$158	$426,600
ExtI-23	FP-245	E Avon Ln, to Sunny Slope Rd	Belmont	800	12	$158	$126,400
ExtI-24	FP-247	E Avon Ln, N 98th to the east	High	1,100	12	$158	$173,800
ExtI-25	FP-286	Wilderness Hills from existing 12" east of 40th to FP-287/FP-288	High	1,600	12	$158	$252,800
ExtI-26	FP-287	Yankee Hill Rd to the south	Southeast	2,500	12	$158	$395,000
ExtI-27	FP-288	FP-287 continued to Rokeby Rd	Southeast	2,700	12	$158	$426,600
ExtI-28	P100814	E Avon Ln to Anthony Ln	Southeast	1,900	12	$158	$300,200
ExtI-29	FP-242	Linwood Ln, Holdrege St to E Avon Ln	High	2,100	12	$158	$331,800
ExtI-30	FP-258	Leighton Ave to N 91st St	High	800	12	$158	$126,400
ExtI-31	FP-280	Rokeby Rd to Current 12" no street	High	1,200	12	$158	$189,600
ExtI-32	FP-283	Wilderness Hill Blvd to Whispering Wind Rd (Wilderness Hills to El Dorado)	Southeast	1,000	12	$158	$158,000
ExtI-33	FP-284	Whispering Wind Rd to S 40th St (Connection Point to Connection Point)	Southeast	2,600	16	$211	$548,600
ExtI-34	FP-285	Wilderness Hill Blvd to Rokeby Rd	Southeast	5,200	12	$158	$821,600
Total Capital Costs							**$14,263,000**

Table 8-5 Distribution Main Extensions - Phase I

Distribution Main Extensions - Phase I							
CIP Tag	LWS Tag	Description	Service Level	Length	Diameter	Unit Cost ($/ft)	Total Capital Costs
Ext6-1	FP-308	N 40th St, Bluff Rd to the south	Belmont	2,900	16	$211	$611,900
Ext6-2	FP-310	N 40th to the east	Belmont	3,900	12	$158	$616,200
Ext6-3	P-2111	N 40th St, Superior St to the north	Belmont	3,200	16	$211	$675,200
Ext6-4	P-2112	Bluff Rd, N 0th St to Hwy 77	Belmont	3,900	16	$211	$822,900
Ext6-5	FP-309/P-2176	North of I-80 to Bluff Road	Belmont	6,100	16	$211	$1,287,100
Ext6-6	P-2000	W Holdrege St, NW 56th St to NW 48th St	Belmont	2,400	16	$211	$506,400
Ext6-7	FP-181	W Holdrege St and NW 40th St, NW 48th St to W Cavalry Ct	Belmont	1,300	12	$158	$205,400
Ext6-8	FP-136	S 70th St, Pine Lake Rd to Yankee Hill Rd	Southeast	5,200	16	$211	$1,097,200
Ext6-9	FP-206	S 98th St, Yankee Hill Rd to the north	Cheney	1,300	24	$317	$412,100
Ext6-10	FP-234	W Denton Rd, S Coddington Ave to SW 12th St	Belmont	2,600	16	$211	$548,600
Ext6-11	FP-238	Holdrege St, N 112th St to the west	High	2,500	16	$211	$527,500
Ext6-12	FP-243	S. 98th St, O St. to Sandalwood Dr.	High	2,100	16	$211	$443,100
Ext6-13	FP-248	N. 105th St, O Street to Vine St. (to Shorefront 12")	High	1,600	12	$158	$252,800
Ext6-14	FP-256	S. 105th St, Randolph St. to O Street	High	2,000	12	$158	$316,000
Ext6-15	FP-257	Leighton Ave, N 98th St to the west	High	1,000	12	$158	$158,000
Ext6-16	FP-264	Jerome & Betty Warner Expy, S 91st St to Yanke Hill Rd	Cheney	3,000	12	$158	$474,000

Distribution Main Extensions - Phase I							
CIP Tag	LWS Tag	Description	Service Level	Length	Diameter	Unit Cost ($/ft)	Total Capital Costs
Ext6-17	FP-282	Rokeby Rd, S 56th St to the west	Southeast	2,700	16	$211	$569,700
Ext6-18	FP-290	S 40th St, Rokeby Rd to the south	Southeast	2,300	16	$211	$485,300
Ext6-19	FP-292	S 40th St to Cromwell Dr	Southeast	2,600	12	$158	$410,800
Ext6-20	FP-311	Abbott Sports Complex, N 70th St to the west	Low	2,700	12	$158	$426,600
Ext6-21	FP-312	Abbott Sports Complex to Arbor Rd	Low	4,000	12	$158	$632,000
Ext6-22	P-1868	N 98th St, Leighton Ave to Adams St	High	2,600	16	$211	$548,600
Ext6-23	P-1869	N 98th St, Holdrege St to Leighton Ave	High	2,600	16	$211	$548,600
Ext6-24	P-1940	S 56th St, Rokeby Rd to Yanke Hill Rd	Southeast	5,200	16	$211	$1,097,200
Ext6-25	P-1941	Rokeby Rd, S 56th St to S 70th St	Southeast	5,300	16	$211	$1,118,300
Ext6-26	P-1958	S 98th St, Yankee Hill Rd to Breagan Rd	Cheney	2,300	24	$317	$729,100
Ext6-27	P-1970	N 7th St, Alvo Rd to Humphrey Ave	Belmont	1,300	16	$211	$274,300
Ext6-28	P-1971	N 7th St, Humphrey Ave to Fletcher Ave	Belmont	2,600	16	$211	$548,600
Ext6-29	P-2002	I-80, NW 56th St to the west	Belmont	5,200	16	$211	$1,097,200
Ext6-30	P-2035	W Denton Rd, SW 12th St to S Folsom St	Belmont	2,600	16	$211	$548,600
Ext6-31	P-2036	SW 12th St, W Pleasant Hill Rd to W Denton Rd	Belmont	2,700	12	$158	$426,600
Ext6-32	P-2107	Arbor Rd, N 70th St to east	Low	2,100	12	$158	$331,800
Ext6-33	FP-144	Approx. S. 36th St, 880ft South of Rokeby Rd to 1/2 mile south of Rokeby Rd (Rokeby to no street name)	High	2,600	16	$211	$548,600

Distribution Main Extensions - Phase I							
CIP Tag	LWS Tag	Description	Service Level	Length	Diameter	Unit Cost ($/ft)	Total Capital Costs
Ext6-34	FP-246	E Avon Ln, Linwood Ln to N 98th (Eastview from Linwood to 98th stub out)	High	1,400	12	$158	$221,200
Ext6-35	FP-249	N. 105th St, Vine St. to Holdrege St. (Shorefront to 12" south of Holdredge at Cessna)	High	3,000	12	$158	$474,000
Ext6-36	FP-251	E Hillcrest Dr, O St to Anthony Ln (to west of tennis courts)	High	4,600	12	$158	$726,800
Ext6-37	FP-252	Randolph St, S. 92nd & E. Hillcrest Dr. to S. 98th St. (From FP-251 to FP-243/253)	High	4,700	12	$158	$742,600
Ext6-38	FP-253	Randolph St, S. 98th to S. 105th St. (from FP-256 to 243/252)	High	2,700	12	$158	$426,600
Ext6-39	FP-259	N 91st St to Holdrege St	High	1,700	12	$158	$268,600
Ext6-40	FP-268	Rokeby Rd, S 84th St to the west	Cheney	1,700	16	$211	$358,700
Ext6-41	FP-271	Mohave Dr, Boone Trail to Yankee Hill Rd	Cheney	2,600	12	$158	$410,800
Ext6-42	FP-273	Boone to Renatta	Cheney	600	12	$158	$94,800
Ext6-43	FP-275	S 78th Rd to S 84th St	Cheney	1,300	12	$158	$205,400
Ext6-44	FP-276	Unnamed Street, 1/2 mile south of Yankee Hill Rd.; from S. 84th St. to S. 92nd. St.	Cheney	3,600	12	$158	$568,800
Ext6-45	FP-277	S. 92nd St, Rokeby Rd. to Unnamed Street 1/2 mile south of Yankee Hill Rd. close to Breagan Rd.	Cheney	2,700	12	$158	$426,600
Ext6-46	FP-278	Unnamed Street from Breagan Rd. and Showers St. to Breagan Rd. for 1785ft SW to S. 92nd Street.	Cheney	1,300	12	$158	$205,400

Distribution Main Extensions - Phase I							
CIP Tag	LWS Tag	Description	Service Level	Length	Diameter	Unit Cost ($/ft)	Total Capital Costs
Ext6-47	FP-279	Rokeby extension east from 30th	High	1,500	16	$211	$316,500
Ext6-48	FP-289	S. 48th St from Rokeby Rd to 1/2 mile South of Rokeby Rd.	Southeast	2,600	12	$158	$410,800
Ext6-49	FP-297	S 27th St to the east	High	3,200	12	$158	$505,600
Ext6-50	FP-299	Unnamed Street 1/2 mile south of Rokeby Rd.; from S 33rd St. to S 36th St Pipe FP-144.	High	2,000	12	$158	$316,000
Ext6-51	FP-305	Folkways Cir to the NE	Low	1,900	12	$158	$300,200
Ext6-52	P100936	S 70th St, Yankee Hill to Rokeby Rd	Southeast	5,400	12	$158	$853,200
Ext6-53	P-1915	Sandalwood Dr to E Hillcrest Dr	Southeast	2,500	12	$158	$395,000
Ext6-54	P-2097	N 40th St, Superior St to the north	Low	2,800	12	$158	$442,400
Total Capital Costs							**$27,966,000**

Table 8-6 Distribution Main Extensions - Phase II

Distribution Main Extensions - Phase II							
CIP Tag	LWS Tag	Item	Service Level	Length	Diameter	Unit Cost ($/ft)	Total Capital Costs
Ext12-1	P-1895	N 112th St, Holdrege to the south	High	2,600	16	$211	$548,600
Ext12-2	P-1896	O St, N112th St to S 120th St	High	2,300	16	$211	$485,300
Ext12-3	P-1898	S 120th St, O St to A St	High	5,300	16	$211	$1,118,300
Ext12-4	P-1899	S 120th St, A St to Seabiscuit Dr	High	2,800	16	$211	$590,800
Ext12-5	P-1900	S 112th St, A St to Secretariat Dr	High	2,900	16	$211	$611,900
Ext12-6	P-1901	A St, S 112th St to S 120th St	High	2,400	16	$211	$506,400
Ext12-7	P-1902	Secretariat Dr, S 112th St to S 120th St	High	3,000	16	$211	$633,000
Ext12-8	P-1903	A St, S 98th St to S 105th St	High	2,700	16	$211	$569,700
Ext12-9	P-1904	A St, S 105th St to S 112th St	High	2,600	16	$211	$548,600
Ext12-10	P-1905	S 105th St, A St to the south	High	1,900	16	$211	$400,900
Ext12-11	P-1912	S 112th St, O St to the south	High	2,500	16	$211	$527,500
Ext12-12	P-1922	Calvert St, Firethron Ln to S 98th St	Southeast	600	12	$158	$94,800
Ext12-13	P-1923	S 98th St, Calvert St to Pioneers Blvd	Southeast	2,700	16	$211	$569,700
Ext12-14	P-1942	S 70th St, Rokeby Rd to Saltillo Rd	Southeast	5,300	24	$317	$1,680,100
Ext12-15	P-1943	Saltillo Rd, S 68th St to S 70th St	Southeast	1,100	24	$317	$348,700
Ext12-16	P-1944	S 56th St, Rokeby Rd to Saltillo Rd	Southeast	4,300	16	$211	$907,300
Ext12-17	P-1945	S 56th St, Southdale Ln to Saltillo Rd	Southeast	1,000	16	$211	$211,000
Ext12-18	P-1946	Saltillo Rd, S 54th St to S 56th St	Southeast	1,100	24	$317	$348,700

Distribution Main Extensions - Phase II							
CIP Tag	LWS Tag	Item	Service Level	Length	Diameter	Unit Cost ($/ft)	Total Capital Costs
Ext12-19	P-1948	Saltillo Rd, S 56th St to the east	Southeast	700	24	$317	$221,900
Ext12-20	P-1962	Saltillo Rd, S 27th St to the east	High	1,900	24	$317	$602,300
Ext12-21	P-1973	NW 27th St, W Alvo Rd to O St	Belmont	2,200	12	$158	$347,600
Ext12-22	P-2013	SW 40th St, W A St to W Peach St	Belmont	1,800	16	$211	$379,800
Ext12-23	P-2014	SW 40th St, W Peach St to W Van Dorn St	Belmont	3,500	16	$211	$738,500
Ext12-24	P-2027	W Calvert St, S Coddington Ave to SW 15th St	Belmont	2,000	12	$158	$316,000
Ext12-25	P-2030	SW 12th St, W Claire Ave to W Old Cheney Rd	Belmont	2,700	12	$158	$426,600
Ext12-26	P-2031	W Old Cheney Rd, SW 12th St to the east	Belmont	1,400	16	$211	$295,400
Ext12-27	P-2033	W Old Cheney Rd, S Folsom St to the west	Belmont	1,300	16	$211	$274,300
Ext12-28	P-2047	SW 12th St, W Old Cheney Rd to W Pleasant Hill Rd	Belmont	2,700	12	$158	$426,600
Ext12-29	P-2093	Pioneers Blvd, S 1st St to S 8th St	Belmont	3,000	12	$158	$474,000
Ext12-30	P-2095	S 1st St, Pioneers Blvd to the south	Belmont	2,500	12	$158	$395,000
Ext12-31	P-2096	N 48th St, Fletcher Ave to Morton St	Low	2,800	12	$158	$442,400
Ext12-32	P-2098	N. 48th St. & Fletcher Ave. SW 1/2 mile to N. 36th St. & Folkways Blvd.	Low	3,000	12	$158	$474,000
Ext12-33	P-2101	N 40th St to Alvo Rd	Low	2,600	12	$158	$410,800
Ext12-34	P-2103	N 40th St, Arbor Rd to the south	Low	2,600	16	$211	$548,600
Ext12-35	P-2147	W Holdrege St, NW 56th St to the west	Belmont	2,600	12	$158	$410,800
Ext12-36	P-2192	S. 98th St, A St. to 300 ft South of South St.	High	2,900	16	$211	$611,900

Distribution Main Extensions - Phase II							
CIP Tag	LWS Tag	Item	Service Level	Length	Diameter	Unit Cost ($/ft)	Total Capital Costs
Ext12-37	FP-129	S 120th St, Seabiscuit Dr to Van Dorn St	High	2,400	16	$211	$506,400
Ext12-38	FP-134	Saltillo Rd, S 54th St to the west	Southeast	1,600	16	$211	$337,600
Ext12-39	FP-135	S 40th St, Saltillo Rd to the north	Southeast	2,900	16	$211	$611,900
Ext12-40	FP-138	Van Dorn St, S 91st St to S 98th St	Southeast	2,300	16	$211	$485,300
Ext12-41	FP-139	S 98th St, Van Dorn St to Calvert St	Southeast	2,600	16	$211	$548,600
Ext12-42	FP-142	S 98th St, A St to the north	High	2,700	16	$211	$569,700
Ext12-43	FP-152	Homestead Expy, W Old Cheney Rd to Warlick Blvd	Belmont	3,600	12	$158	$568,800
Ext12-44	FP-174	Old Cheney Rd, Hunt Drive to S. Folson St	Belmont	4,900	12	$158	$774,200
Ext12-45	FP-198	South St to S 98th St	High	5,900	24	$317	$1,870,300
Ext12-46	FP-199	S 98th St, Van Dorn St to the north	High	2,300	16	$211	$485,300
Ext12-47	FP-200	Van Dorn St, S 98th St to S 112th St	High	5,300	16	$211	$1,118,300
Ext12-48	FP-201	S 112th St, Secretariat Dr to Van Dorn St	High	2,300	16	$211	$485,300
Ext12-49	FP-202	Van Dorn St, S 112th St to S 120th St	High	2,500	16	$211	$527,500
Ext12-50	FP-208	W Calvert St, SW 15th St to Lincoln Regional Center	Belmont	2,800	12	$158	$442,400
Ext12-51	FP-209	SW 12th St, W Burnham St to the south	Belmont	1,400	12	$158	$221,200
Ext12-52	FP-210	SW 15th St, W Calvert to W Burnham St & SW 12th St	Belmont	1,900	12	$158	$300,200
Ext12-53	FP-218	W Van Dorn, SW 40th St to Pioneers Golf Course	Belmont	2,500	16	$211	$527,500

Distribution Main Extensions - Phase II							
CIP Tag	LWS Tag	Item	Service Level	Length	Diameter	Unit Cost ($/ft)	Total Capital Costs
Ext12-54	FP-224	SW 12th St, W Claire Ave to the north	Belmont	2,600	12	$158	$410,800
Ext12-55	FP-226	W Pioneers Blvd, SW 12th St to S Folsom St	Belmont	2,600	16	$211	$548,600
Ext12-56	FP-227	S 1st St, Old Cheney Rd to the north	Belmont	2,800	12	$158	$442,400
Ext12-57	FP-228	W Claire Ave, SW 12th St to S Folsom St	Belmont	2,700	12	$158	$426,600
Ext12-58	FP-229	S 1st St to Radcliff St	Belmont	4,700	12	$158	$742,600
Ext12-59	FP-237	S 98th St, A St to Van Dorn St	Southeast	7,100	16	$211	$1,498,100
Ext12-60	FP-239	N 112th St, O St to the north	High	2,800	16	$211	$590,800
Ext12-61	FP-240	O St, N 112th to the west	High	2,600	24	$317	$824,200
Ext12-62	FP-244	S 112th St, A St to the north	High	2,700	16	$211	$569,700
Ext12-63	FP-250	Vine St,N. 105th St. to N. 112th. St.	High	2,600	12	$158	$410,800
Ext12-64	FP-254	S 105th St, A St to the north	High	2,900	12	$158	$458,200
Ext12-65	FP-255	Randolph St, S. 105th to S. 112th St.	High	2,700	12	$158	$426,600
Ext12-66	FP-291	Saltillo Rd, S 40st St to the east	Southeast	2,600	16	$211	$548,600
Ext12-67	FP-293	S. 48th St from Saltillo Rd to 1/2 mile South of Rokeby Rd.	Southeast	2,600	12	$158	$410,800
Ext12-68	FP-294	S 27th St, Saltillo Rd to the north	High	2,800	24	$317	$887,600
Ext12-69	FP-295	S 38th St, Saltillo Rd to the north	High	2,700	16	$211	$569,700
Ext12-70	FP-296	Saltillo Rd, S 38 to the west	High	1,800	24	$317	$570,600
Ext12-71	FP-300	S. 33rd St from Saltillo Rd to 1/2 mile North of Saltillo Rd.	High	2,700	12	$158	$426,600

Distribution Main Extensions - Phase II							
CIP Tag	LWS Tag	Item	Service Level	Length	Diameter	Unit Cost ($/ft)	Total Capital Costs
Ext12-72	FP-301	Pioneers Blvd, Thorn Ct to S 98th St	Southeast	400	16	$211	$84,400
Ext12-73	FP-349	Saltillo Rd, S 56th St to S 68th St	Southeast	3,500	24	$317	$1,109,500
Ext12-74	FP-186	O St to W Alvo Rd	Belmont	2,400	12	$158	$379,200
Total Capital Costs							**$41,215,000**

Figure 8-1 Recommended Phased Improvements for the Distribution System

Appendix A. Climate Change Assessment

BLACK & VEATCH CORPORATION
Lincoln Water System **B&V Project Number 401472**
2020 Facilities Master Plan Update **January 31, 2020**
Climate Change Projections

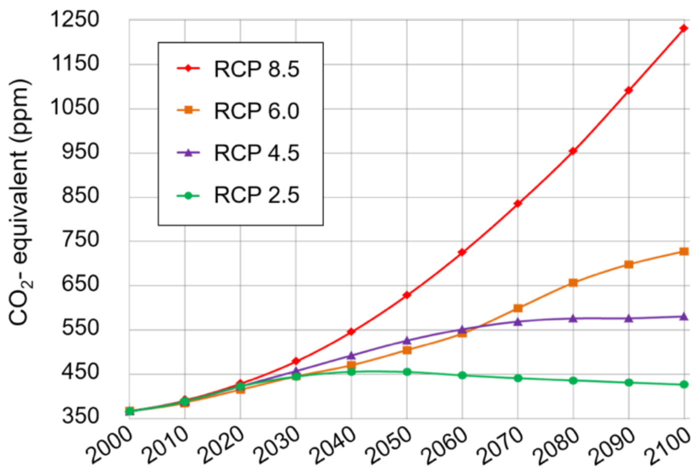
To:	File
From:	Martha Shulski, Nebraska State Climatologist
Reviewed By:	Andrew Hansen, Ben Day

Central and Eastern Nebraska Mid-century Climate Projections (2041-2070)

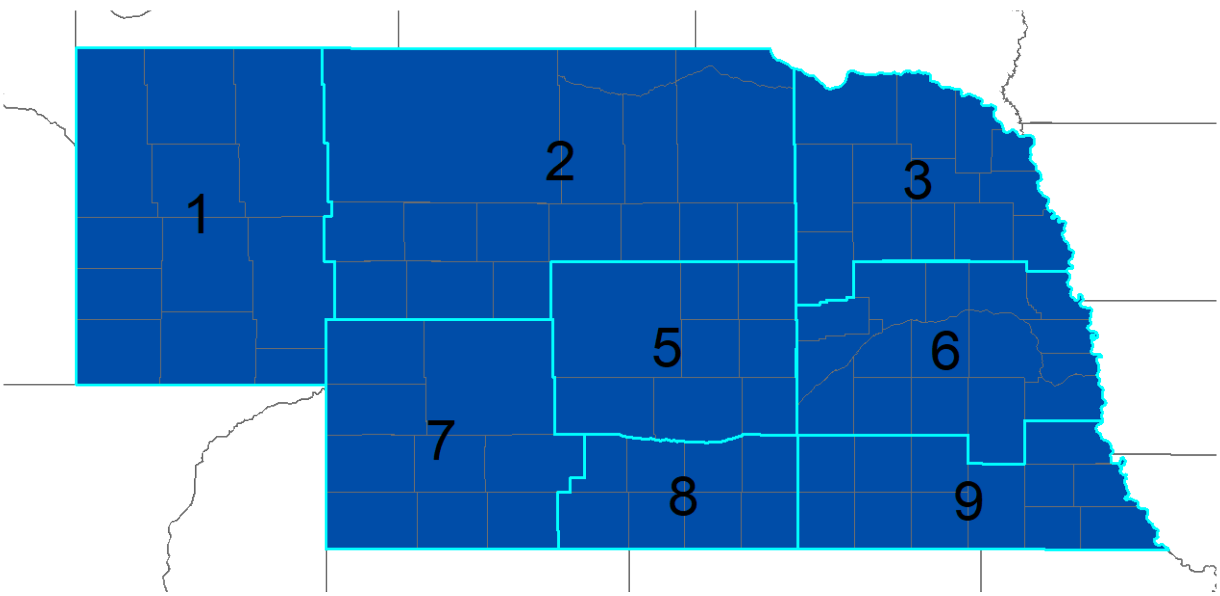
A set of global climate models are used to predict the influence of future atmospheric carbon dioxide (CO_2) concentrations on temperature and precipitation patterns. The model ensemble CMIP5 and the RCP 8.5 scenario were used in this project to assess climate change impacts to central and eastern Nebraska. CMIP5 is the Coupled Model Intercomparison Project Phase 5. Its main objectives are to analyze how realistic the models are in simulating the past climate, provide projections for mid and late twenty-first century, and to better understand the components responsible for differences in model projections like feedback between the clouds and the carbon cycle. The CMIP5 facilitates the strengths and weaknesses of climate models to enhance the focus of the development of future models. RCP is the Representative Concentration Pathway, which is a greenhouse gas concentration future trajectory. It has a wide range of outcomes based on four different model simulations (see figure A-1).

The RCP 8.5 scenario, used in the images and data tables, has the highest increase in CO_2 emissions and therefore has the most extreme climate projections compared to the other RCP model runs. If energy generation does not change and business as usual continues, the future CO_2 concentrations could look like the RCP 8.5 If actions are taken towards a cleaner and more renewable energy source, one of the less extreme models runs could occur causing the future climate changes to be less extreme. The hatched areas on the below images represent areas with more than 50 percent of the models showing a significant change and more the 67 percent agreeing on the sign change. These scenarios are widely accepted and utilized in international and national climate assessment reports, such as the recent U.S. National Climate Assessment (www.globalchange.gov/nca4).

This report outlines annual and seasonal changes in temperature and precipitation for Nebraska. The mid-century projection represents an average over the 30-year timeframe from 2041 to 2070 minus the 1971 – 2000 climate normal average. Changes are provided in degrees F for temperature and percentage for precipitation. The eight climate divisions in the state (figure A-2) are identified in the graphics and tables.



Global carbon dioxide concentration emissions scenarios used in the CMIP5 climate model projections. RCP 8.5 is termed the 'business as usual' approach and was the one chosen for this project. It assumes little to no mitigative action and follows the current rate of CO_2 increase.



Represented here are the eight climate divisions in Nebraska; 1 – Panhandle, 2 – Northcentral, 3 – Northeast, 5 – Central, 6 – East central, 7 – Southwest, 8 – Southcentral, 9 - Southeast. Seasonal climate projections are summarized based on these regions.

Winter

The average winter (Dec-Jan-Feb) temperature shows a projected increase (compared to the 1971 – 2000 average) by mid-century with high confidence. This increase has not always been a trend historically. The most recent 30-years (1987 - 2016) shows a decrease of 0.09-1.5°F while the 100-year trend (1895 - 2016) shows an increase of 2.1-3.6°F. The increase of 4-5°F by mid-century is a rate greater than previously experienced.

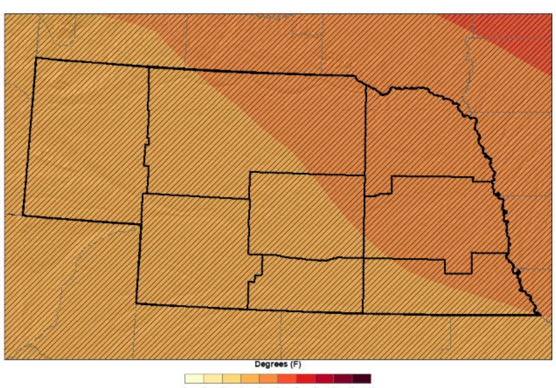
Winter shows the largest projected increase, in percent change, in precipitation when compared to the other seasons. Conversely, it is the smallest increase in liquid-equivalent precipitation out of the four seasons. The eastern part of the state shows a smaller percent increase along with lower confidence. Although, the eastern part of the state receives the most precipitation during the winter the increase in liquid-equivalent precipitation will be the greatest in amount. On average, winter precipitation averages are 1.6-2.5 inches, with the lower totals found for the central climate divisions.

Table A-1 Projected Temperature and Precipitation Change by Mid-Century

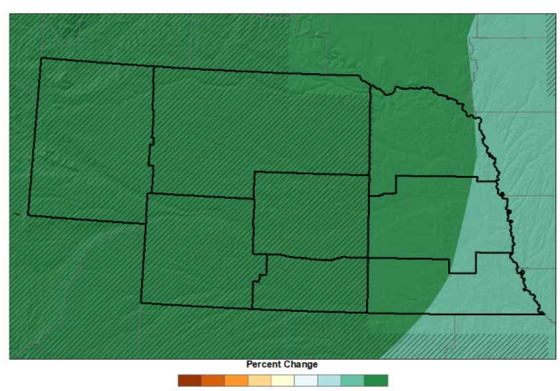
Climate Division	Temperature Increase
Northcentral	5°F
Northeast	5°F
Central	4°F
East Central	5°F
Southcentral	4°F
Southeast	4-5°F

Climate Division	Precipitation Increase	
Northcentral	15-20%	0.24-0.32”
Northeast	15-20%	0.33-0.44”
Central	15-20%	0.26-0.34”
East Central	10-20%	0.23-0.46”
Southcentral	15-20%	0.26-0.34”
Southeast	10-20%	0.25-0.5”

PROJECTED TEMPERATURE CHANGE BY SEASON
Winter, 2041-2070, RCP 8.5



PROJECTED PRECIPITATION CHANGE BY SEASON
Winter, 2041-2070, RCP 8.5



Spring

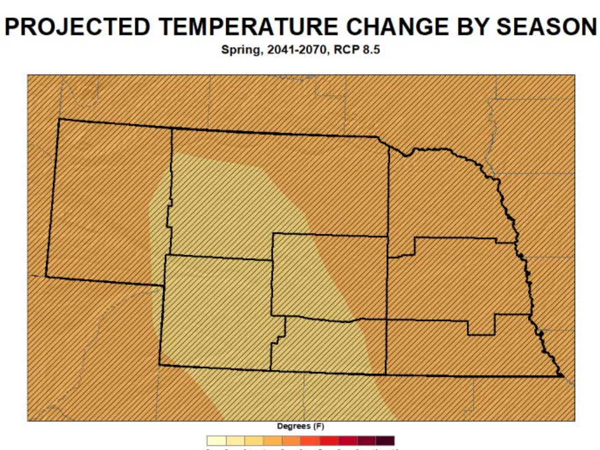
The average spring (Mar-Apr-May) temperature shows a projected increase (compared to the 1971 – 2000 average) by mid-century with high confidence. The increase continues the general historic trend of warming. The most recent 30-years (1987 - 2016) shows an increase of 0.39-0.72°F while the 100-year trend (1895 - 2016) shows an increase of 1.5-2.5°F. The coming decades could experience a 3-4°F increase in temperature, which is double the historic 100-year trend.

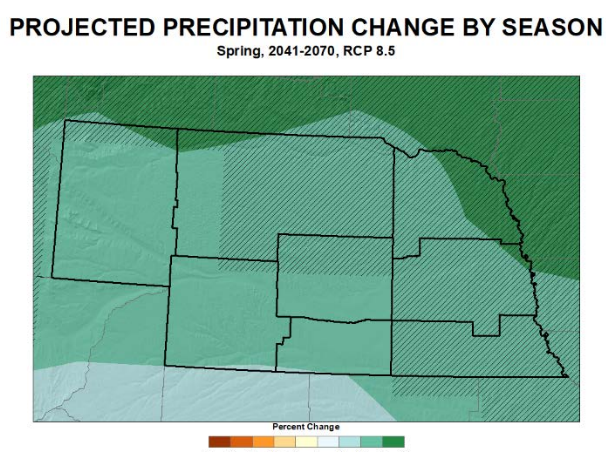
Spring shows the second largest projected increase, in percent change, in precipitation when compared to the other seasons. However, spring will have the largest increase (in terms of amount) in liquid equivalent precipitation. The northeast part of the state will experience the greatest percent change. The eastern part of the state will experience that largest increase in liquid equivalent precipitation with high confidence. On average, spring precipitation totals are 6.9-8.7 inches, with the lower totals found for the central climate divisions. The 30-year historic trends range from a 2-17 percent increase with the higher totals coming from the southeastern climate divisions. The historic 100-year trend ranges from a 13-25 percent increase with the increase coming from the southern climate divisions.

Table A-2 Projected Temperature and Precipitation Change by Mid-Century

Climate Division	Temperature Increase
Northcentral	4°F
Northeast	4°F
Central	3-4°F
East Central	4°F
Southcentral	3°F
Southeast	4°F

Climate Division	Precipitation Increase	
Northcentral	10-15%	0.68-1.0”
Northeast	10-20%	0.8-1.6”
Central	10-15%	0.74-1.1”
East Central	10-15%	0.84-1.25”
Southcentral	10-15%	0.74-1.1”
Southeast	10-15%	0.87-1.3”





Summer

The average summer (June-July-Aug) temperature shows a projected increase by mid-century with high confidence. The increase has been a historic trend. The 30-year (1987 - 2016) historic trend shows an increase of 0.28-1.12°F while the historic 100-year (1895 - 2016) trend shows and increase of .03-0.96°F. The increase of 5°F by mid-century is several degrees higher than previously experienced in past climate.

Summer is the only season that has a projected decrease in precipitation. The whole state could experience a 5-10 percent decrease in precipitation by mid-century. Even though it is a small percentage decrease, summer has the highest precipitation totals, on average, compared to other seasons. This will mostly affect the eastern and southern parts of the state. On average, summer precipitation totals are 8.8-11.8 inches, with the lower totals found for the central climate divisions. The 30-year historic trends range from a 10 percent decrease to an 11 percent increase with the higher totals found for the northern climate divisions. The historic 100-year trend ranges from a 5 percent decrease to a 13 percent increase with the increase occurring for the northcentral climate division.

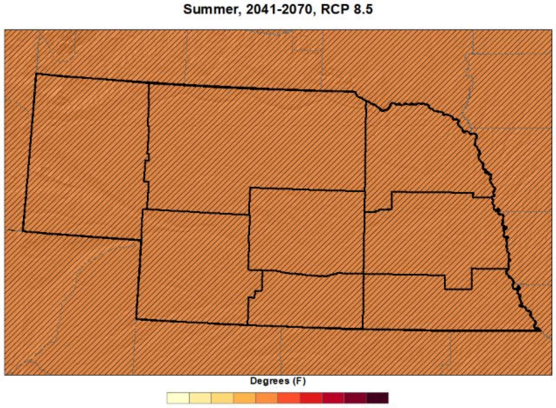
Table A-3 Projected Temperature and Precipitation Change by Mid-Century

Climate Division	Temperature Increase
Northcentral	5°F
Northeast	5°F
Central	5°F
East Central	5°F
Southcentral	5°F
Southeast	5°F

Climate Division	Precipitation Decrease	
Northcentral	15-20%	0.44-0.88”
Northeast	15-20%	0.53-1”
Central	15-20%	0.5-1”
East Central	10-20%	0.57-1.13”
Southcentral	15-20%	0.5-1”
Southeast	10-20%	0.6-1.2”

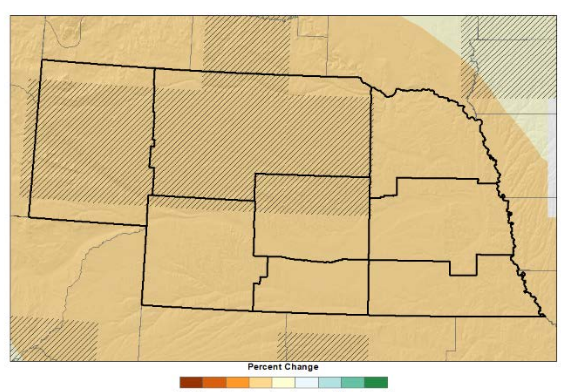
PROJECTED TEMPERATURE CHANGE BY SEASON

Summer, 2041-2070, RCP 8.5



PROJECTED PRECIPITATION CHANGE BY SEASON

Summer, 2041-2070, RCP 8.5



Fall

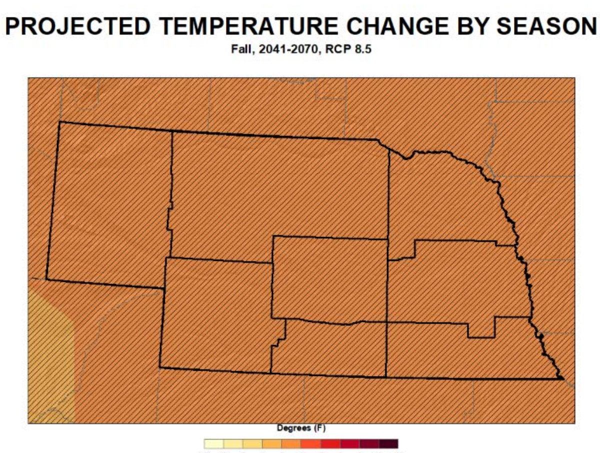
The average fall (Sep-Oct-Nov) temperature shows a projected increase by mid-century with high confidence. The increase has been a historic trend especially in the recent decades. The 30-year (1987 - 2016) historic trend shows an increase of 2.5-3°F while the historic 100-year (1895 - 2016) trend shows and increase of 0.2-0.8°F. The next few decades could experience a 5°F increase in temperature which is double the 100-year trend.

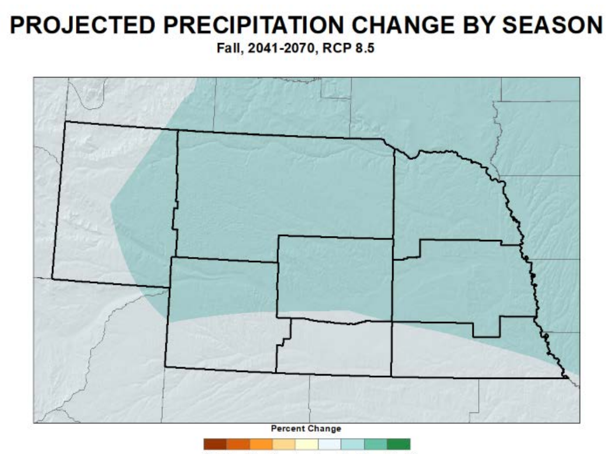
Fall shows a small projected increase, in percent change, in precipitation when compared to the other seasons. The greatest increase in liquid equivalent precipitation will occur in the northern climate divisions. There is lower confidence on the changes in the fall compared to the other seasons. On average, fall precipitation averages are 3.85-6.65 inches, with the lower totals found for the central climate divisions. The 30-year historic trends range from a 12 percent decrease to a 6 percent increase with the higher totals coming from the south and central climate divisions. The historic 100-year trends range from a 1 percent decrease to a 30 percent increase with the increase coming from the northern climate divisions.

Table A-4 Projected Temperature and Precipitation Change by Mid-Century

Climate Division	Temperature Increase
Northcentral	5°F
Northeast	5°F
Central	5°F
East Central	5°F
Southcentral	5°F
Southeast	5°F

Climate Division	Precipitation Increase	
Northcentral	5-10%	0.2-0.38”
Northeast	5-10%	0.28-0.56”
Central	5-10%	0.23-0.46”
East Central	5-10%	0.3-0.6”
Southcentral	0-5%	0-0.24”
Southeast	0-5%	0-0.33”

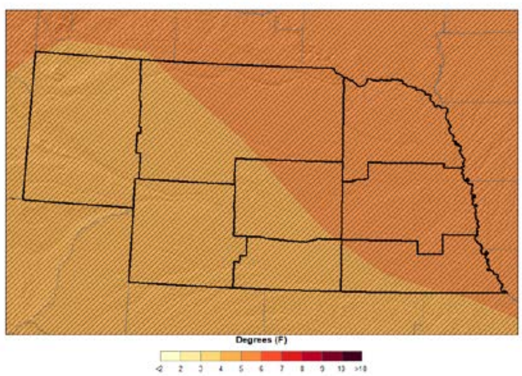




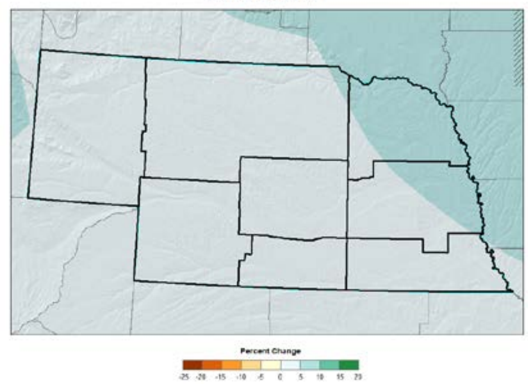
Annual

The annual average temperature shows a projected increase by mid-century with high confidence. The magnitude of the increase is in the 4° to 5°F range for central and eastern Nebraska, when compared to the 1971 – 2000 average. Historically, temperatures have increased in the state by approximately 1.5°F over the past 100 years (1895 – 2016). The rate of increase has at least doubled (eastern climate divisions), and in the central divisions has increased by a factor of four. Annual total precipitation is expected to increase by approximately 5 percent, compared to the 1971 – 2000 average. Historically, precipitation on an annual basis has increased over the long-term (by 5 percent - 10 percent). That trend has accelerated in recent decades (1987 – 2016), particularly for the central and northern divisions.

PROJECTED ANNUALLY-AVERAGED TEMPERATURE CHANGE
2041-2070 RCP 8.5



PROJECTED ANNUALLY-AVERAGED PRECIPITATION CHANGE
2041-2070 RCP 8.5



Appendix B. 2020 Lincoln Wellfield Groundwater Modeling

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TECHNICAL MEMORANDUM

To: Andrew Hansen

From: Travis Zielke, CGWP

Date: 5/12/2020

Re: 2019 Lincoln Well Field Groundwater Modeling

CC:

Project No.: 0219047

PURPOSE

This memo summarizes groundwater modeling work conducted in support of the 2020 Lincoln Water Facilities Master Plan Update. This modeling effort was a continuation of prior modeling referred to as the Ashland Well Field Model and used in previous planning reports submitted to the City of Lincoln. Changes incorporated in the 2019 model include revisions to the precipitation recharge based on climate modeling by Martha Shulski, Nebraska State Climatologist, adjustments to the location of the Platte River at low flow rates, and the inclusion of two new wells proposed to be constructed sometime in the future as Lincoln's demands increase.

MODEL DESCRIPTION

The Ashland Well Field model encompasses a 34 square mile area centered on the US Highway 6 Bridge over the Platte River, just northeast of Ashland. The modeled well field includes 40 vertical wells in the North and South well fields, the four existing horizontal wells, and the two horizontal wells proposed for drilling at a future date. These well locations are shown on Figure 1.

The Ashland Well Field model was originally created in 1987 for the purposes of evaluating proposed well field expansion alternatives. The model has been updated a number of times since then, most recently in 2014 when the model was rebuilt to run in Groundwater Vistas with an enhanced level of detail. This work was described in reports and memos from TZA Water Engineers and others in 1987, 1989, 1994, 2004, 2013, and 2014.

MODEL REFINEMENT

For this modeling effort, two refinements were made to the model used in previous studies. One of these changes was made based on climate modeling conducted by Martha Shulski, which indicated that on average in the future, fall through spring would tend to be 15% wetter, and summers would be 12.5% dryer. These results were incorporated into the model by adjusting the precipitation recharge. The 15% wetter fall through springs were included by increasing the recharge during the Antecedent period by 15%. The dryer summers were included by reducing recharge during the Dry Spring antecedent condition and during each drought scenario by 12.5%. The scenario descriptions provide more details on antecedent condition modeling.

The second refinement made was regarding the location of the Platte River in low flow conditions. Observations made in support of previous modeling indicated that at flow rates less than 3000 cfs, the Platte River is no longer running bank-to-bank, and instead runs in smaller channels inside the river bed. In previous work, the river was modeled as running along the west bank north and south of Ashland Island, and east around the island. For this study, an analysis was conducted comparing results where the river was run fully along the east bank versus those where the river was running fully along the west bank. The river cells are shown for the two runs on Figure 1. It was found that the river running along the west bank resulted in lower sustainable production rates as compared to if the river ran along the east bank. To ensure a conservative analysis, the west bank configuration was used in this study. See the River Configuration section for more details.

Other model parameters are summarized on Table 1. Detailed descriptions of these parameters can be found in the report dated September 1, 1987.

Sensitivity Analysis

These two refinements were analyzed to evaluate to how the model results would be impacted by their incorporation into drought planning scenarios. The changes in precipitation recharge had little effect on well field production. This is attributable to the drought scenarios having a short duration and the prior model assumption that precipitation recharge is only 5% of the annual total precipitation during a drought. These combine to add very little water to simulation, and a reduction in that water supply had minimal impact on well field yields.

Placing the river to the west of Ashland Island had significant impact on yields compared to previous modeling. This is attributable to the increased distance between the horizontal wells and their water source. As the river moves further away from the horizontal wells, sustainable production from the wells drops considerably. These impacts are quantified later in this report.

SIMULATIONS FOR DROUGHT PLANNING PURPOSES

TZA previously performed modeling for drought planning purposes in response to the droughts and resulting river flow conditions that occurred in the summers of 2002 and 2012. Data gathered during those droughts was included in comprehensive modeling conducted in 2013, which were summarized in the Technical Memorandum dated January 23, 2014. Relevant results from the 2014 memo are included here in Table 3 as a baseline of comparison for the results of this modeling effort.

The primary focus of this 2019 modeling was to evaluate the addition of the two new horizontal wells proposed to be drilled in 2024. These wells are shown as Future-1 and Future-2 on the attached Figure 1. Three different well field configurations were studied: current conditions, current conditions plus the addition of well Future-1, and current conditions plus the addition of wells Future-1 and Future-2. After the previously discussed model refinements were made, each of the well field configurations were evaluated in a series of nine different droughts. These droughts corresponded to river flows of 1500 cfs, 700 cfs, or 200 cfs for periods of 30, 60, and 90 days.

For each of these droughts, the antecedent conditions prior to the modeled drought period are critical in getting valid results. For the 1500 cfs and 700 cfs scenarios, a 5000 cfs steady-state period is run before starting the drought period. These runs include average spring/fall pumping requirements for Lincoln. This portion of the run represents the spring run-off season, and allows the model to start with water table elevations reasonable for the beginning of a drought.

For the 200 cfs scenarios, an additional "Dry Spring" period of 60 days is added after the 5000 cfs steady-state period. This period has an elevated demand half-way in between spring and fall levels, and serves to reduce water table elevations at the beginning of the extremely low flows modeled in the 200 cfs drought scenarios. The 200 cfs drought also experiences increased severity from a total loss of recharge to the model from precipitation. These drought scenarios are discussed in more detail in the January 23, 2014 Model Update Memo.

The scenario settings are summarized below:

<u>Scenarios Considered</u>

For each of the three well configurations, nine drought scenarios were conducted:

Riverflow (cfs)	Duration (Days) 30	60	90	
1500				Normal Antecedent (4.4% Precip Recharge)
700				
200				Dry Spring Antecedent (Zero Precip Recharge)

<u>Well Capacities/Pumping Limitations</u>

Current well capacities were considered to be as shown on Table 2. The North and South Well Field capacities (32-1A to 86-2) were determined from well testing performed by Lincoln Water System staff during September and October of 2012, a time of the year when ground water levels and thus pumping capacities are typically the lowest.

The pumping withdrawals used within MODFLOW are considered to occur on a continuous basis. In order to provide a margin of safety for circumstances such as down-time for pump repairs and decreases in well capacities which may occur during peak use periods, it was decided that pumping from individual wells should be limited to an amount less than the full capacity. In addition, Lincoln Water System staff have determined from experience that individual wells within the South Well Field often experience excessive drawdown if they are operated more than 50% of the time. Based upon these considerations, pumping from individual wells was limited as follows; for steady state simulations the North Well Field wells are limited to 70% of the maximum capacity and the South Well Field wells are limited to 50% of the maximum capacity; for transient simulations the North Well Field wells are limited to 85% of the maximum capacity and the South Well Field wells are limited to 75% of the maximum capacity. Because all of the horizontal wells will include a sufficient number of pumps and sufficient capacity to allow production at rates in excess of the modeled capacities of 12,000 gpm, no further constraints were applied to the horizontal wells.

At the end of a model scenario, the model cells which contain wells are evaluated to determine if water levels have exceeded the allowed drawdown for that well. The amount of allowed drawdown is 25% of saturated thickness in vertical wells, and 50% of saturated thickness in horizontal wells. These criteria were developed for previous model studies and are described in detail in the Report dated September 1, 1987.

<u>River Configurations</u>

For these planning scenarios, river configurations used in previous modeling were changed. In previous modeling, the location of the river was based on field observations in 1988 and 2012. These observations indicated that, downstream of U.S. Highway 6, as flow rates increased the river filled the bank from east to west and transitioned to bank-to-bank flows at 3000 cfs. Below 1500 cfs, the river was observed to flow entirely through a channel east of Ashland Island. In the 2002 drought, the river was observed to flow west around Ashland Island. The 2002 configuration had been used in drought modeling since 2012.

For this series of model scenarios, a 700 cfs/60-day scenario was examined wherein the river was modeled as filling from the west rather than filling from the east as in previous modeling. Filling from the west caused the sustainable yields to decline considerably in the horizontal wells, which resulted in an overall decrease in the Well Field's production. To ensure a conservative analysis, this modeling effort uses a west to east filling methodology described in detail below.

For the stream reach between the U.S. Highway 6 bridge and the Interstate-80 bridge:

1) at flow rates less than about 1500 cubic feet per second (cfs), the entire river is flowing through a channel located west of Ashland Island;
2) at flow rates between about 1500 and 3000 cfs, the river begins flowing through a small channel east of and adjacent to Ashland Island, and gradually spreads over most of the streambed as flow rates approached 3000 cfs; and,
3) at flow rates greater than 3000 cfs, the river was flowing bank-to-bank and the entire streambed is generally submerged.

For the stream reaches upstream of the U.S. Highway 6 bridge and downstream of the Interstate-80 bridge, it is assumed that as flow rates increase, the river will fill from west to east across its channel, until it reaches bank-to-bank conditions at a flow rate of 3000 cfs. Based upon these assumptions, river depth versus river width relationships were developed by application of Manning's Equation.

The wetted areas for each river cell under bank-to-bank flow conditions (considered to occur at streamflows greater than 3000 cfs) were determined by utilizing recent NAIP aerial photography. Wetted areas for flow conditions less than 3000 cfs were calculated based on channel widths determined by Manning's Equation. A river stage-discharge relationship was developed based on records from the USGS gauging station at the

Highway 6 Bridge. An average river gradient for the modeled river reach was determined from USGS topographic maps. River stages for each river cell under various flow rates were determined based on the Highway 6 gauge stage-discharge relationship, the river gradient, and Manning's Equation.

RESULTS

The results of the modeling analyses based on the current production wells are summarized on Table 4. The entries marked as "OK" on Tables 4 through Table 6 indicate that the specified pumping rate could be maintained for the specified time period without exceeding the drawdown criteria. The entries marked as "Fails" on Tables 4 through Table 6 indicate that the specified pumping rate could not be maintained for the specified time period without exceeding the drawdown criteria.

Assuming the river runs to the west of Ashland Island has a significant effect on pumping rates for the well field. Comparing scenarios from Table 3 to those on Table 4 indicate that yields decline between 5 and 10 MGD due to this change. This is mainly attributable to the reduction in sustainable yield from the horizontal wells. While running the river west of the island improves recharge to the North and South well field, this effect is outweighed by the reduced yield from the horizontal wells.

For the current well configuration, sustainable production varied between 90 MGD and 115 MGD, depending on the drought scenario. The addition of Future-1 changes the sustainable production to between 95 MGD and 120 MGD depending on the drought scenario. The addition of both Future wells changes the sustainable production to between 105 MGD and 125 MGD depending on the drought scenario.

Attachments: Figure 1
Tables 1-6

REFERENCES

Ashland Well Field Comprehensive Development Plan Modeling Study, GMI Specialized Engineering Services, September 1, 1987

Ashland Well Field Comprehensive Development Plan Updated Groundwater Modeling Study, TZA Water Engineers, June 1989

Ground Water Modeling Analyses For The City Of Lincoln's Permit To Appropriate Natural Flow For Induced Ground Water Recharge, TZA Water Engineers, February 1994

2003 Well Field Modeling – Methodology and Results, TZA Water Engineers, April 3, 2004

Well Field Modeling for Drought Planning Purposes, TZA Water Engineers, March 22, 2013
Update of Groundwater Model and Well Field Modeling for Drought Planning Purposes, TZA Water Engineers, January 23, 2014

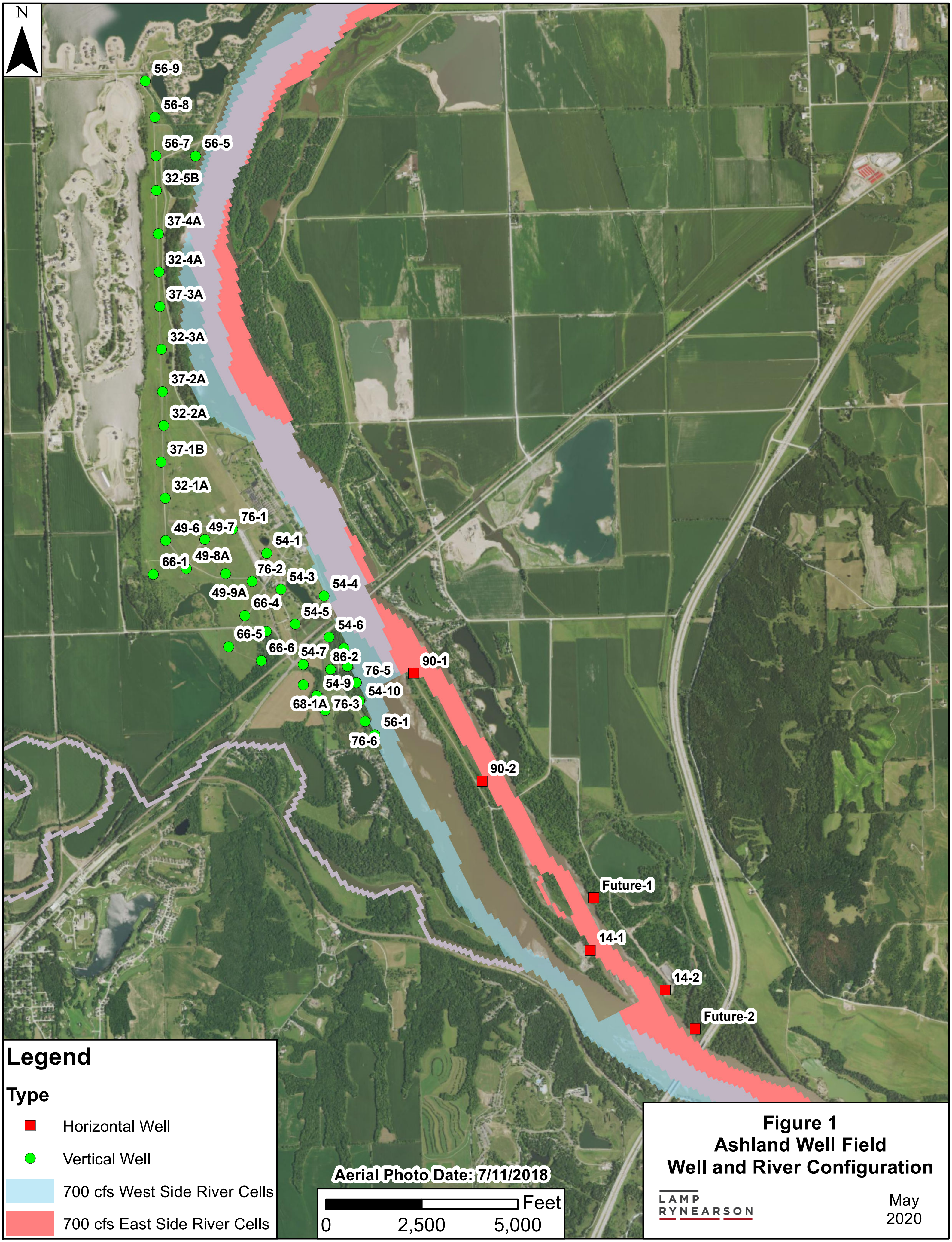

Figure 1
Ashland Well Field
Well and River Configuration

TABLE 1

SUMMARY OF INPUT DATA
ASHLAND WELL FIELD GROUND WATER MODEL

Parameter	Value	Units
Hydraulic Conductivity		
Sand/Gravel Aquifer	353	ft/day
Clay Unit	1.0×10^{-3}	ft/day
Storage Coefficient		
Sand/Gravel Aquifer	15	%
Clay Unit	5	%
Specific Storage for Confined Sand/Gravel Aquifer Conditions	3.0×10^{-5}	ft^{-1}
Hydraulic Conductivity of Riverbed Material/Thickness of Riverbed (K/M)		
Platte River	6	day^{-1}
Salt and Wahoo Creeks		
In Clay Unit	3.5×10^{-8}	day^{-1}
In Sand/Gravel Aquifer	0.014	day^{-1}
Drains		
In Clay Unit	10^{-3}	day^{-1}
In Sand/Gravel Aquifer	3.53	day^{-1}
Aquifer Recharge as a Percent of Total Precipitation		
In Clay Unit	1	%
In Sand/Gravel Aquifer	15	%
River/Creek Stage	Variable	ft
Well Pumpage	Variable	ft^3/day

Table 2
Modeled Well Capacities
Lincoln Ashland Well Fields

Well #	Maximum Modeled Well Capacity (gpm)	Well #	Maximum Modeled Well Capacity (gpm)
32-1A	2,500	66-4	2,500
32-2A	2,500	66-5	2,500
32-3A	2,500	66-6	2,500
32-4A	2,000	68-1	2,500
32-5B	2,000	76-1	3,000
37-1B	2,500	76-2	2,900
37-2A	2,600	76-3	2,500
37-3A	2,000	76-4	2,500
37-4A	2,000	76-5	2,500
49-6	2,500	76-6	2,500
49-7	2,500	86-1	2,500
49-8A	2,500	86-2	2,800
49-9	2,500	90-1	12,500
54-1	2,500	90-2	12,500
54-3	2,500	14-1	12,500
54-4	2,500	14-2	12,500
54-5	2,500	24-1	12,500
54-6	2,500	24-2	12,500
54-7	2,500		
54-8	2,500		
54-9	2,500		
54-10	2,500		
56-1	2,500		
56-5	2,000		
56-7	2,500		
56-8	2,500		
56-9	2,500		
66-1	2,500		
		Total (gpm)	173,800
		Total (mgd)	250.3

TABLE 3

Summary of Previous Ground Water Modeling Scenarios for Drought Planning Purposes

City of Lincoln Ashland Well Fields - Current 2019 Capacity

From 2014 Technical Memo - Previous River Configuration

River Flow: 1500 cfs

Simulation	Pumping Rate			
Period	**110 mgd**	**115 mgd**	**120 mgd**	**125 mgd**
30 Days	OK	OK	OK	Fails
60 Days	OK	OK	Fails	Fails
90 Days	OK	Fails	Fails	Fails

River Flow: 700 cfs

Simulation	Pumping Rate			
Period	**100 mgd**	**105 mgd**	**110 mgd**	**115 mgd**
30 Days	OK	OK	OK	Fails
60 Days	OK	OK	Fails	Fails
90 Days	OK	OK	Fails	Fails

River Flow: 200 cfs

Simulation	Pumping Rate			
Period	**90 mgd**	**95 mgd**	**100 mgd**	**105 mgd**
30 Days	OK	OK	OK	Fails
60 Days	OK	OK	Fails	Fails
90 Days	OK	Fails	Fails	Fails

TABLE 4
Summary of Transient Ground Water Modeling Scenarios for Drought Planning Purposes
City of Lincoln Ashland Well Fields - Current 2019 Capacity

River Flow: 1500 cfs

Simulation Period	Pumping Rate			
	105 mgd	**110 mgd**	**115 mgd**	**120 mgd**
30 Days	OK	OK	OK	Fails
60 Days	OK	OK	Fails	Fails
90 Days	OK	Fails	Fails	Fails

River Flow: 700 cfs

Simulation Period	Pumping Rate			
	90 mgd	**95 mgd**	**100 mgd**	**105 mgd**
30 Days	OK	OK	OK	Fails
60 Days	OK	OK	Fails	Fails
90 Days	OK	OK	Fails	Fails

River Flow: 200 cfs

Simulation Period	Pumping Rate			
	85 mgd	**90 mgd**	**95 mgd**	**100 mgd**
30 Days	OK	OK	OK	Fails
60 Days	OK	OK	Fails	Fails
90 Days	OK	OK	Fails	Fails

TABLE 5

Summary of Transient Ground Water Modeling Scenarios for Drought Planning Purposes

City of Lincoln Ashland Well Fields - Current 2019 Capacity plus Future-1

River Flow: 1500 cfs

Simulation Period	Pumping Rate			
	110 mgd	**115 mgd**	**120 mgd**	**125 mgd**
30 Days	OK	OK	OK	Fails
60 Days	OK	OK	Fails	Fails
90 Days	OK	Fails	Fails	Fails

River Flow: 700 cfs

Simulation Period	Pumping Rate			
	105 mgd	**110 mgd**	**115 mgd**	**120 mgd**
30 Days	OK	OK	OK	Fails
60 Days	OK	OK	Fails	Fails
90 Days	OK	Fails	Fails	Fails

River Flow: 200 cfs

Simulation Period	Pumping Rate			
	95 mgd	**100 mgd**	**105 mgd**	**110 mgd**
30 Days	OK	OK	OK	OK
60 Days	OK	OK	Fails	Fails
90 Days	OK	Fails	Fails	Fails

TABLE 6
Summary of Transient Ground Water Modeling Scenarios for Drought Planning Purposes
City of Lincoln Ashland Well Fields - Current 2019 Capacity plus Future-1 and Future-2

River Flow: 1500 cfs

Simulation	Pumping Rate			
Period	**115 mgd**	**120 mgd**	**125 mgd**	**130 mgd**
30 Days	OK	OK	OK	Fails
60 Days	OK	OK	Fails	Fails
90 Days	OK	Fails	Fails	Fails

River Flow: 700 cfs

Simulation	Pumping Rate			
Period	**110 mgd**	**115 mgd**	**120 mgd**	**125 mgd**
30 Days	OK	OK	OK	Fails
60 Days	OK	OK	Fails	Fails
90 Days	OK	Fails	Fails	Fails

River Flow: 200 cfs

Simulation	Pumping Rate			
Period	**100 mgd**	**105 mgd**	**110 mgd**	**115 mgd**
30 Days	OK	OK	OK	OK
60 Days	OK	OK	Fails	Fails
90 Days	OK	OK	Fails	Fails

Appendix C. Transmission Main Condition Assessment

BLACK & VEATCH CORPORATION
Lincoln Water System **B&V Project Number 401472**
2020 Facilities Master Plan Update **January 31, 2020**
Transmission Main Condition Assessment

To:	File
From:	Bryon Livingston, Black & Veatch
Reviewed By:	Joe Nease, Black & Veatch

Condition Assessment of Water Transmission Mains

The goal of a condition assessment is to gather information using non-destructive testing methods to evaluate the current condition of the pipe. The results of the inspection are analyzed and evaluated to determine if repair or rehabilitation is needed and cost effective. The key to condition assessment is in the understanding and implementation of the inspection technologies used to gather the information needed. The EPA defines condition assessment as "The collection of data and information through direct and/or indirect methods, followed by analysis of the data and information, to make a determination of the current and/or future structural, water quality, and hydraulic status of the pipeline" (EPA/600/X-09/003 April 2007). The primary emphasis in this project is structural condition assessment, as opposed to hydraulic or water quality condition assessment.

The critical transmission mains for providing water service to the City of Lincoln are the three transmission mains that provide water from the Ashland Treatment Plant near the Platte River to the City of Lincoln located approximately 19 miles away. We will examine the condition assessment approach for each of these four pipelines based upon material, age, and criticality to operations:

- 36-inch Cast Iron Transmission Main
- 48-inch Prestressed Concrete Cylinder Pipe (PCCP) Transmission Main
- 48-inch/54-inch PCCP from Northeast Pump Station to Vine Street Pumping
- 54-inch Welded Steel Transmission Main

Understanding how a given pipe material fails is critical to being able to assess the condition based on the data collected from the inspection. The major factors, shown in Figure C-1, include:

- Manufacturing defects
- Improper design/construction
- Pressure (operating and surges)
- Temperature changes
- External loads
- Internal and external corrosion
- Third party damage

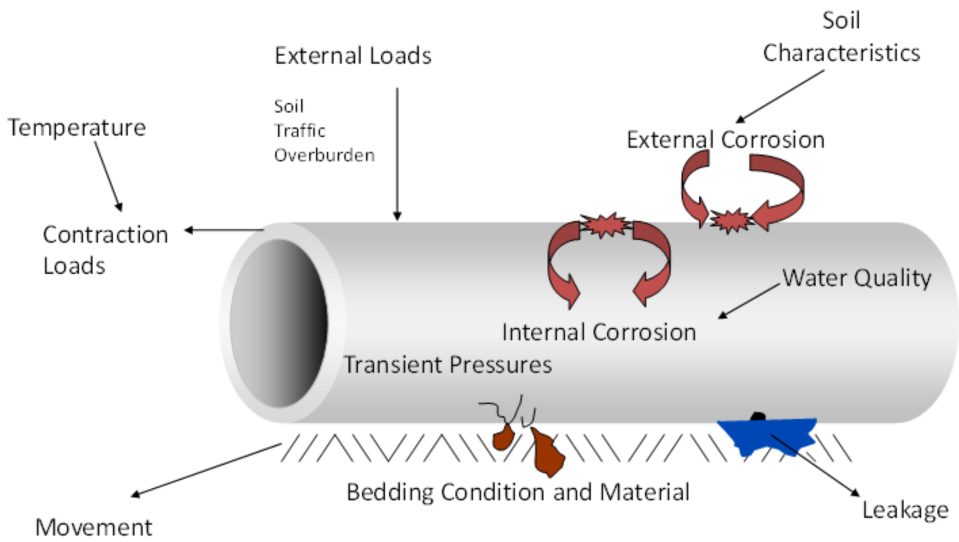

Figure C-1 Factors Affecting Pipe Failure

The decision to rehabilitate or replace a pipeline should be based upon how the pipeline meets the level of service expected. The failure of a pipeline is described as the pipe not being able to provide this level of service. A pipeline with redundancy and which is not critical for providing flows could have a level of service that a leak once a year is acceptable. Other pipelines that provide a majority of the required flows have a higher level of service and any leaks would create an impact on customer service. When evaluating pipelines, it is necessary to consider that similar pipe in different operating conditions will not fail at the same time. The facts show that not all pipe installed in the same year fail at the same time. The deterioration of a pipe is not necessarily a function of the age of the material but rather the cumulative effect of the external and internal forces acting on it.

36-inch Cast Iron Transmission Main

This pipeline is assumed to be cast iron and was installed in the mid-1930's when the Ashland WTP was built. The pipeline runs about 20 miles from the Ashland WTP to the 51st Street Pumping Station and then about 5 miles through the City to the "A" Street Pumping Station for a total length of about 25 miles. In a previous study it was determined the pipe is AWWA standard 1927 Class "C" pipe with a 1.36" wall thickness. The grade of iron used could either be 18/40 or 21/45.

The first cast iron pipe manufacturing process in the 1900's consisted of pouring molten iron into a sand mold, which stood on end in a pit in the ground. The pipe manufactured by this method is referred today as "pit" cast iron pipe. Due to potential for inconsistencies in the pipe wall thickness the pipe was designed with a wall thickness that was much greater than required for the anticipated loadings that the pipe would be subjected to. The pipe was installed using a rope and lead that was heated, poured into the joint and allowed to cool. This pipe normally did not have internal or external coatings but because of the wall thickness continues to be in service throughout the country.

The process was improved in the 1920's when the use of centrifugally casting pipe in a sand mold was introduced. The pipe manufactured with this process is referred to as "spun" or "centrifugal cast iron pipe. The centrifugal forces that are induced on the iron result in an increase in the tensile strength. The higher strength and lack of inconsistencies in the wall thickness resulted in thinner wall thickness than the pit cast pipe. Interior lining of the pipe with cement to prevent corrosion was available in the 1920's but did not become widely accepted until late in the 1930's. The improved tensile strength and reduction in wall thickness coupled with the lack of corrosion protection resulted in this pipe not having the long service life as the "pit" cast iron pipe.

Also, in the 1920's a plasticized sulfur cement compound, known as "leadite" was developed as an alternative to lead for sealing the pipe joints in construction. The use of leadite to seal the joints has proven to be inferior to lead and affects the service life of the joints and therefore the pipeline. The leadite has a different thermal expansion than cast iron and results in additional internal stresses that can lead to longitudinal splits in the pipe bell. Also, the sulfur can facilitate pitting corrosion resulting in circumferential breaks on the spigot end of the pipe. EPA has reported the failure rate in the industry for leadite joint pipe is significantly higher than for lead joint pipe even though the pipe may not be as old.

The metallurgical make up of cast iron is susceptible to a "graphitic" corrosion where an electrochemical reaction occurs between the cathodic graphite component (flakes) and the anodic iron matrix causing a metal loss.

In locations where the 36-inch Transmission Main has recently been exposed, it is our understanding that any leakage is occurring at the joints. This could be an indicator that leadite joints were used for construction and it would be consistent with that time period. The reported observations of the pipe at the leaks indicate the pipe wall is in good condition.

Inspection Plan for 36-inch Cast Iron Transmission Main

The implementation of an inspection plan allows the City of Lincoln to develop a realistic infrastructure management plan based on actual data. With accurate data, utility managers can make informed decisions on pipe replacement or repair instead of relying on guesswork. By identifying and phasing these activities, condition assessments frequently result in significant capital savings to utilities that would otherwise have replaced an entire pipeline.

A phased approach for data collection allows utilities to begin with the basic information and then select the next step based upon the results of the first. The cost of condition assessment increases with the amount of data collected, but increased data provides potentially more guidance for decisions about rehabilitation or replacement.

The proposed plan for the 36-inch Transmission Main is based upon the historical information regarding the leaks at the joints and the reported good condition of the pipe.

- Although inspection of the entire 36-inch may eliminate concerns about totality of the system, the most critical segment with respect to reliability/redundancy is the segment from 51st Street to the A Street Pumping Station. Therefore, to keep cost in check, we would recommend only this segment at this time.
- Based upon the leak detection additional testing may be required.
- If the initial inspection, and subsequent additional testing, yield concerning results, the City should then consider inspection of the 20-mile segment between Ashland and 51st Street Pumping Station.

The typical failure of cast iron with leadite joints is leaks at the joints and leaks typically occur before breaks or splits in the pipe. The recommended method for leak detection would be an in-line free swimming tool capable of detecting and locating small leaks. The purpose of the leak detection would be to determine if there are undetected leaks along the alignment indicating the current condition of the pipe.

The number, location and size of the leaks would be evaluated to determine the recommended next steps. If there are multiple leaks detected, the recommendation would likely be that the pipeline has failed, and rehabilitation is recommended. If no leaks are detected the results indicate the pipeline has a remaining service life, and because the pipeline is critical to operations, additional testing only is recommended in the future.

The potential to rehabilitate or replace this segment of the pipeline would be evaluated based upon the risk associated with failure. The need for soil corrosion potential analyses or examination of the external pipe condition is currently not recommended based on the wall thickness of the cast iron and the reported good condition. During any future repair of leaks the pipe should be examined for pitting and the next step re-evaluated.

There are three in-line leak detection systems currently available:

- Pure Technologies (SmartBall),
- Hydromax (Nautilus)
- PICA (RECON+). The PICA (RECON+) system does not have a tracking system to locate the tool along the alignment, so we would not recommend it for this inspection.

The details for an inspection with these tools should be prepared prior to the work being conducted and an inspection plan developed. The inspection plan would identify access and retrieval locations, tracking sites along the alignment, and other details required for a successful inspection.

Pure Technology SmartBall®

The SmartBall technology has been used in the United States for over 10 years and there are several case studies showing the advantages and disadvantages of this tool. The improved tracking of the tool with sensors spaced about 2,000 feet apart has reduced the potential to "lose" the ball during the inspection. The SmartBall has an inner core with the sensors protected by a foam outer layer as shown in Figure C-2. The sensor can detect very small leaks and air pockets since it is inside the pipe.

The SmartBall is inserted through a 4-inch tap and retrieved with a net that is inserted in the pipeline. The battery life is an estimated 15 hours and provides approximately 15 miles of inspection per insertion. The recommended segment of the 36-inch Transmission Main could be inspected with one insertion and retrieval.

Figure C-2 SmartBall Components

The preliminary cost for conducting a leak detection by Pure Technologies (received November 2019) on the proposed 5-mile segment of the 36-inch Transmission Main is described in Table C-1. The estimated cost includes a 20 percent contingency and 20 percent engineering and administrative costs. The construction estimated cost includes the cost to install a 4-inch tap for insertion and retrieval ($15,000 each) and 15 sensor locations ($1,000 each) along the alignment.

Table C-1 Estimated Cost for SmartBall Leak Detection - 51st Street to "A" Street

Description	Units	Unit Cost	Total Cost
Site visit, planning, data review	Each	$10,000	$10,000
Mobilization of Equipment	Each	$16,250	$16,250
Leak detection inspection	5 miles	$16,538	$82,690
Report of Results	Each	$10,500	$10,500
Total Estimated Inspection Cost			$119,440
Contingency	20%	$23,888	$23,888
Engineering & Administrative	20%	$28,665	$28,665
Construction Estimated Cost	2 Taps and 15 Sensors	$45,000	$45,000
Total Estimated Cost			**$216,993**

Hydromax Nautilus

The Nautilus system is new to the United States and has been available for less than 5 years. The technology was developed in Spain and is similar to the Smart Ball. Nautilus is an in-line, free swimming leak and air pocket detection tool for larger diameter distribution and transmission mains. The Nautilus is different from the Smart Ball because it is neutrally buoyant and floats instead of rolling along the bottom.

The Nautilus is inserted and retrieved through a 4-inch or larger tap. The system is tracked using synchronizers and detectors attached to the pipeline along the alignment about every 2,000 feet as shown in Figure C-5. The detectors and synchronizers track the system but are also used to help determine the location of any leaks identified by the Nautilus.

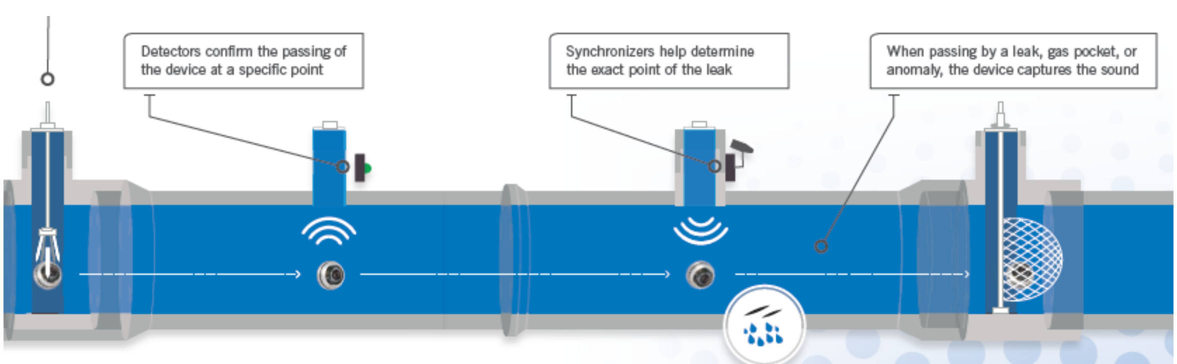

Figure C-5 Nautilus Inspection Layout

The preliminary cost for conducting a leak detection by Hydromax (received November 2019) on the proposed segment of the 36-inch Transmission Main is described in Table C-2. The estimated cost includes a 20 percent contingency and 20 percent engineering and administrative costs. The construction estimated cost includes the cost to install a 4-inch tap for insertion and retrieval ($15,000 each) and 15 sensor locations ($1,000 each) along the alignment.

Table C-2 Estimated Cost for Nautilus Leak Detection - 51st Street to "A" Street

Description	Units	Unit Cost	Total Cost
Site visit, planning, data review	Each	$0	$0
Mobilization of Equipment	Each	$11,000	$11,000
Leak detection inspection	5 miles	$7,125	$35,625
Report of Results	Each	$0	$0
Total Estimated Inspection Cost			$46,625
Contingency	20%	$9,325	$9,325
Engineering & Administrative	20%	$11,190	$11,190
Construction Estimated Cost	2 Taps and 15 Sensors	$45,000	$45,000
Total Estimated Cost			**$112,140**

PCCP Transmission Mains

There are two PCCP transmission mains included in this evaluation. The first is the 48-inch pipeline that was installed in about 1950 that starts out as ductile iron leaving the Ashland Water Treatment Plant but transitions to PCCP within the first mile. The pipeline runs about 16 miles from the WTP to the Northeast Pump Station. The second pipeline to be evaluated was installed in the 1970's and is 48-inch and 54-inch and runs about 5 miles from the Northeast Pumping Station to Vine Street Pumping Station.

PCCP is a common material for water transmission mains and has been used since the 1940's. PCCP was introduced during World War II to minimize the use of steel and by the 1960's was used throughout the United States and Canada. The manufacturing standards for PCCP were modified from 1964 to 1992 to allow for the use of thinner, high strength prestressing wires which is susceptible to failure from hydrogen embrittlement.

PCCP consists of a concrete core cast inside a steel cylinder that serves as a watertight membrane. High-tensile strength steel wire is wrapped directly on the steel cylinder, providing the strength to support the internal loads from the pipe operation. Wires are embedded in a cement mortar to protect the wire from corrosion. A cross section of PCCP identifying the components is shown in Figure C-6. The pipe design effectively utilizes the compressive strength of concrete and the high-tensile strength of steel in the wires. Manufacture of PCCP is covered by American Water Works Association (AWWA) standard C301 and the design is covered in AWWA C304.

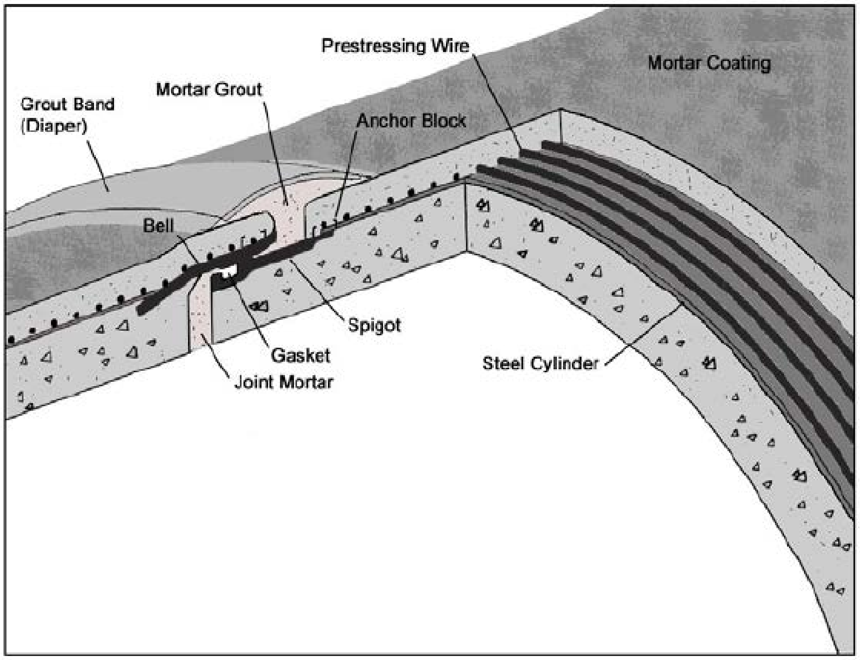

Figure C-6 PCCP Components

There are many reasons why PCCP can fail, but the most common are circumferential and longitudinal cracking of the mortar. Failure In the circumferential mode is typically in the form of prestress loss in the core caused by wire breaks. This is the most common failure mode of PCCP. That this failure mode is significant is supported by the fact that prestressing wires are known to be the primary structural component of PCCP, and breakage of the prestressing wires can result in sudden failure of the pipe. When wires break, the loads are transferred to the concrete core causing them to crack and exposing the steel cylinder to soil and ground water. Eventually, the steel cylinder corrodes and fails. Wire breaks can be caused by corrosion or hydrogen embrittlement. Causes of this failure mode may be related to design, manufacturer, installation, operation, or aggressive environment.

The failure process of hydrogen embrittlement is when elemental hydrogen diffuses into the steel causing it to become brittle and fail at a tensile stress below the normal yield stress of steel. The prestressed concrete pipe is made using wire with residual stress. When the cement coating breaks down or cracks, water and the accompanying hydrogen comes into contact with the wires. Over time, as the hydrogen affects the steel in the wires, they become brittle and break. Hydrogen embrittlement can also be caused by stray currents or over use of cathodic protection currents.

Design and manufacturing standards for PCCP have changed over time. One time period in particular has been associated with higher damage rates due to deficient wire and/or coating standards. In the period 1964 to 1992, the prevailing standard allowed the use of Class IV wire, which had no maximum tensile strength limit. During the manufacturing process, this wire was sometimes over-heated during drawing, leading to dynamic strain aging. This process resulted in less ductility and increased susceptibility to damage from hydrogen embrittlement. Also, during this same time frame, porous or thin mortar coating was applied over the prestressing wires. Low moisture in the mix increased permeability, allowing the coating to absorb chlorine ions exposing the steel to a corrosive environment.

In the longitudinal mode, PCCP may fail as a result of pipe movement caused by differential settlement, inadequate hydraulic thrust restraint, Poisson's effect of pressure, thermal loads, nearby blast or vibration loads, or seismic loads. The failure process may involve opening of joints or cracking of the concrete core or tearing of the steel cylinder with or without corrosion, and failure with or without prior leakage (AWWA M77, 2019).

Inspection Plan for 48-inch PCCP from Ashland WTP

There have not been any reported leaks or failures on the 48-inch Transmission Main from the Ashland WTP to Northeast Pumping Station. This may be an indication that the soil is not corrosive to the concrete pipe and conducting a field survey to collect soil data would likely not provide useful data. The gold standard for inspection of the PCCP would be to utilize electromagnetic (EM) technology mounted on a robot crawler to assess the condition of the main. However, that would be cost prohibitive for 16 miles of main when considering the main has not shown indications of degradation. Therefore, the recommended inspection plan is to evaluate the condition of the pipeline with an internal leak detection using the SmartBall or Nautilus technology.

The need for additional testing for broken prestressing wires will be evaluated based upon the number of leaks and the location of the leaks. Multiple leaks would be an indication that the pipe is deteriorating, and these areas could be further analyzed by EM or visual inspection. If any leaks are deemed significant, they should be repaired which would provide a good opportunity for additional inspection of the main, either with manned entry or with the electromagnetic inspection for broken wires.

The preliminary cost for conducting a leak detection by Pure Technologies on the 16 miles of 48-inch PCCP Transmission Main is described in Table C-3. The estimated cost includes a 20 percent contingency and 20 percent engineering and administrative costs. The construction estimated cost includes the cost to install 4-inch taps ($30,000 each) for insertion and retrieval and 40 sensor locations ($1,000 each) along the alignment.

Table C-3 Estimated Cost for SmartBall Leak Detection - Ashland to Northeast

Description	Units	Unit Cost	Total Cost
Site visit, planning, data review	Each	$10,000	$10,000
Mobilization of Equipment	Each	$16,250	$16,250
Leak detection inspection	5 miles	$16,538	$82,690
	Next 10 miles	$12,075	$120,750
	Over 15 miles (1 mile)	$7,350	$7,350
Report of Results	Each	$10,500	$10,500
Total Estimated Inspection Cost			$247,540
Contingency	20%	$56,858	$49,508
Engineering & Administrative	20%	$59,409	$59,409
Construction Estimated Cost	2 Taps and 40 Sensors	$100,000	$100,000
Total Estimated Cost			**$456,457**

The preliminary cost for conducting a leak detection by Hydromax on the 16 miles of 48-inch PCCP Transmission Main is described in Table C-4. The estimated cost includes a 20 percent contingency and 20 percent engineering and administrative costs. The construction estimated cost includes the cost to install 4-inch taps ($30,000 each) for insertion and retrieval and 40 sensor locations ($1,000 each) along the alignment.

Table C-4 Estimated Cost for Nautilus Leak Detection - Ashland to Northeast

Description	Units	Unit Cost	Total Cost
Site visit, planning, data review	Each	$0	$0
Mobilization of Equipment	Each	$11,000	$11,000
Leak detection inspection	16 miles	$6,431	$102,896
Report of Results	Each	$0	$0
Total Estimated Inspection Cost			$113,896
Contingency	20%	$22,779	$22,779
Engineering & Administrative	20%	$27,335	$27,335
Construction Estimated Cost	2 Taps and 40 Sensors	$100,000	$100,000
Total Estimated Cost			**$264,010**

Inspection Plan for 48-inch/54-inch PCCP from Northeast Pumping Station to Vine Street Pumping Station

There has not been any reported leaks or failures on the 5 miles of 48"/54" Transmission Main from the Northeast Pumping Station to Vine Street Pumping Station. This pipeline was constructed during the time period (early 1970's) that the standards for PCCP had been modified. In addition, this pipeline is one of the most critical in the Lincoln distribution system for providing water service to customers. Therefore, due to the criticality and comparatively shorter length, the recommended inspection plan for this PCCP pipeline is to conduct EM inspection on a majority of the pipeline.

Pure Technologies is currently the only contractor with the patent for the EM technology to inspect PCCP. Pure has two platforms for inspection of PCCP pipe for wire breaks: free-swimming or robotic crawler platform. The free-swimming platform does not require the pipe to be out of service and can be inserted through a 12" tap and retrieved with a net inserted into the pipe. The crawler platform requires the pipe be out of service and an 18" tap for insertion. The crawler is tethered so it can go about 4,000 feet in either direction and provides CCTV during the inspection. The crawler platform is recommended for this inspection. The proposed inspection will collect data on about 16,000 feet through two insertions. The data obtained from this inspection will be used to determine if additional testing is required.

The preliminary cost estimate for conducting this EM crawler inspection by Pure Technologies (provided November 2019) on the 48-inch/54-inch PCCP Transmission Main is described in Table C-5. The estimated cost includes a 20 percent contingency and 20 percent engineering and administrative costs. The construction estimate includes the cost to install two 18-inch taps ($40,000 each) for insertion of the crawler.

Table C-5 Estimated Cost for EM Inspection of PCCP - Northeast to Vine

Description	Units	Unit Cost	Total Cost
Site visit, planning, data review	Each	$25,000	$25,000
Mobilization of Equipment	Each	$27,000	$27,000
EM inspection	16,000 feet	$10.45/ft	$167,200
Report of Results	Each	$15,000	$15,000
Total Estimated Inspection Cost			$234,200
Contingency	20%	$46,840	$46,840
Engineering & Administrative	20%	$56,208	$56,208
Construction Estimated Cost	2 Taps	$80,000	$80,000
Total Estimated Cost			**$417,248**

54-inch Welded Steel Pipe

This 54-inch transmission pipeline was constructed in two phases in the 1993 time frame under contracts FWC.2TM and FWC.3TM. Project 2TM was steel pipe installed by Garney and the pipe manufacturer was Thompson Pipe. There are two classes of pipe, Class 1 with 0.320" wall thickness, and Class 2 with 0.560" thickness. Project 3TM was steel pipe constructed by a contractor called Kenko and the pipe was manufactured by Thompson Pipe. Based on information available, the project used two pipe classes and thicknesses, 0.320" and 0.450". These pipelines were constructed with both rubber gasket, and welded joints in restraint areas, and polyethylene tape wrap coating. These two contracts of 54-inch pipeline from Ashland WTP to Greenwood (interconnect) total approximately 7.6 miles long.

Inspection Plan for Steel

The recommended inspection plan for this pipeline is:

- Inspection of the cathodic protection system to determine the condition of the anodes and if they need to be replaced.
- An in-house pressure test on the segment.
- If the pressure test indicates a leak may be present, then a leak detection technology could be employed to locate the leaks.

References

"Deteriorating Buried Infrastructure Management Challenges and Strategies", EPA, May 2002

EPA/600/X-09/003 – "Innovation and Research for Water Infrastructure for the 21st Century", April 2007

AWWA Manual M77 – "Condition Assessment of Water Mains", 2019

Appendix D. Water Treatment Plant Condition Assessment

BLACK & VEATCH CORPORATION
Lincoln Water System **B&V Project Number 401472**
2020 Facilities Master Plan Update **January 31, 2020**
Water Treatment Plant Condition Assessment

To:	File
From:	Tim Malcolm, Black & Veatch
Reviewed By:	Andrew Hansen, Black & Veatch

Water Treatment Plant Condition Assessment

Summary

A high level condition assessment was conducted on 10/10/19 and 10/11/19 to identify the current condition of existing facilities, determine improvements needed within the next 12 years, and further evaluate the feasibility of expanding the West Plant in comparison to expansion at the East Plant.

The *2014 Facilities Master Plan* had identified the next plant expansion to occur at the West Treatment Plant by means of filter rehabilitation with the hopes of increasing the filter loading rate from 5.0 gpm/st to 6.0 gpm/sf. Throughout the condition assessment activities, concerns were identified by staff, primarily regarding the ability to physically process over 70 mgd through the facility, based upon previous operational knowledge from the 1980's. Specifically, when the West WTP was pushed to rates around 70 mgd, a bypass was utilized which circumvented the entire treatment process including aeration, chlorine contact, and filtration. This operational practice was subsequently discontinued as the safe drinking water act (SDWA) was amended and the bypass has been removed.

In light of these restrictions, in order to expand the West WTP some other modifications would be required in addition to the filter rehabilitation. Other recommended improvements include replacement of the existing clearwell transfer pumps (which would increase capacity and simply CT calculation), addition of a fourth aerator and contact basin, chemical feed modifications, and an allowance for hydraulic improvements to ensure the facility could convey the flows. The total capital cost for expansion of the West WTP by 12 mgd is summarized in Table D-1. The planning level opinion of probable capital cost is $10,749,000 for a 12 mgd expansion, which equates to an expansion cost of $0.90/gallon.

In addition to the hydraulic concerns, there is considerable apprehension with respect to filter performance when the media is replaced. It is believed that all the filter media within the facility is original. An alternative concept to replacing all filters would be to replace media in only two filters. Pilot testing should be performed in advance to compare alternative media configurations and confirm manganese removal. Given our experience at the facility, and the need to develop a manganese oxide coating on the media, it may be determined that media will need to be "pre-treated" prior to installation.

Table D-1 West Water Treatment Plant Expansion Opinion of Probable Cost

West Plant Expansion - 12 MGD				
Item	**Quantity**	**Unit**	**Unit Cost**	**Total Costs**
West Water Treatment Plant Rehab				
Rehab Fourteen Filters with Dual Media				
Media Removal	23400	CF	$20	$468,000
Filter Coatings	14	EA	$25,000	$350,000
Filter Underdrain Removal and Replacement	5200	SF	$150	$780,000
Media Installation	23400	CF	$30	$702,000
Rehab Surface Wash	14	EA	$7,500	$105,000
Skim, Test, Disinfect	14	EA	$15,000	$210,000
Trough and Crack Repair	1	LS	$100,000	$100,000
Replace Transfer Pumps to Increase Capacity (Wetwell to N. Res)	2	EA	$750,000	$1,500,000
Add 4th Aerator	1	LS	$500,000	$500,000
Add Contact Basin and Yard Piping	1	LS	$1,000,000	$1,000,000
Chemical Feed (Chlorinator)	1	LS	$150,000	$150,000
Hydraulic Improvements (Allowance)	1	LS	$500,000	$500,000
Piloting and Testing	1	LS	$50,000	$50,000
General Allowance	1	LS	$250,000	$250,000
Subtotal				$6,665,000
General Requirements		12%		$799,800
Subtotal				$7,464,800
Contingency		20%		$1,492,960
TOTAL CONSTRUCTION COSTS				**$8,957,760**
Engineering, Legal, Administration		20%		$1,791,552
TOTAL CAPITAL COSTS				**$10,749,000**

The other alternative for plant expansion is to expand the filtration capacity at the East WTP. The East WTP currently has a capacity of 60 mgd (originally 50 mgd prior to filter re-rating). The plant was configured such that 16 additional filters can be added to provide additional capacity of 120 mgd. As part of the study B&V provided costing analysis of adding either two filters (15 mgd) or four filters (30 mgd). The cost to add only two filters was not deemed to be in the City's best interest as it would be inefficient with respect to building walls, foundations, ozone system expansion, etc. Therefore, we would recommend that the next expansion of the East Water Treatment Plant should be 30 mgd. The planning level opinion of probable capital cost for this expansion would be $24,804,000 which equates to $0.83/gallon. Expansion of the East WTP would also be more beneficial from a treatment perspective as the City will add one or two more collector wells in the interim, increasing their reliance on water which is under the influence of surface water.

Table D-2 East Water Treatment Plant Expansion Opinion of Probable Cost

East Filtration Expansion and Ozone Expansion (30 mgd)				
Item	**Quantity**	**Unit**	**Unit Cost**	**Total Costs**
Filtration Expansion - East Plant				
Sitework	1	LS	$331,500	$331,500
Site Electrical	1	LS	$37,000	$37,000
Filter Expansion	1	LS	$9,650,000	$9,650,000
Ozone Contactor	1	LS	$2,251,000	$2,251,000
LOX Storage and Feed System	1	LS	$2,960,000	$2,960,000
Chlorine Feeder	1	LS	$150,000	$150,000
Subtotal				**$15,379,500**
General Requirements		12%		$1,845,540
Subtotal				**$17,225,040**
Contingency		20%		$3,445,008
TOTAL CONSTRUCTION COSTS				**$20,670,048**
Engineering, Legal, Administration		20%		$4,134,010
TOTAL CAPITAL COSTS				**$24,804,000**

In addition to the plant expansion, the condition assessment activities determined that both the East and West Water Treatment Plants are in need of repairs. The last major work at these facilities was in the late 1980's/early 1990's under the Lincoln Water Consortium. A listing and cost of rehabilitation needs as determined from the condition assessment reviews are as follows in Table D-3 and Table D-4.

Table D-3 General East Plant Rehabilitation Opinion of Probable Cost

East Water Treatment Plant Rehab				
Description	**Quantity**	**Unit**	**Unit Cost**	**Total Costs**
East Water Treatment Plant Rehab				
Replace Ambient Ozone Analyzers	10	EA	$10,000	$100,000
Replace Ozone Basin Drain Valves	2	EA	$20,000	$40,000
Filter Pipe Gallery - Clean Corrosion and Overcoat all pipe	1	LS	$150,000	$150,000
Exterior Maintenance of expansion/contraction joints & flashing	1	LS	$25,000	$25,000
Rehab roof drains in filter - Coat or cover with insulation	1	LS	$50,000	$50,000
General Allowance	1	LS	$50,000	$50,000
Subtotal				**$415,000**
General Requirements		12%		$49,800
Subtotal				**$464,800**
Contingency		20%		$92,960
TOTAL CONSTRUCTION COSTS				**$557,760**
Engineering, Legal, Administration		20%		$111,552
TOTAL CAPITAL COSTS				**$669,000**

Table D-4 General West Plant Rehabilitation Opinion of Probable Cost

West Water Treatment Plant Rehab plus Rehab of Two Filters w/ Dual Media				
Description	**Quantity**	**Unit**	**Unit Cost**	**Total Costs**
West Water Treatment Plant Rehab				
Rehab Two Filters with Dual Media for Full Scale Pilot				
Media Removal	2	EA	$36,000	$72,000
Filter Coatings	2	EA	$25,000	$50,000
Filter Underdrain Removal and Replacement	2	EA	$80,000	$160,000
Media Installation	2	EA	$40,000	$80,000
Rehab Surface Wash	2	EA	$7,500	$15,000
Skim, Test, Disinfect	2	EA	$15,000	$30,000
Replace Air Compressors (2)	2	LS	$25,000	$50,000
Miscellaneous Air Piping	1	LS	$5,000	$5,000
Remove Shroud from around HVAC Duct - Protect Filters	1	LS	$30,000	$30,000
Coating Rehab - Remove from masonry, clean and recoat metals	1	LS	$300,000	$300,000
Window glazing, weatherproof louvers	1	LS	$15,000	$15,000
Replacement/Maintenance HVAC Equipment (Allowance)	1	LS	$100,000	$100,000
Replace Surface Wash Piping Filters 11-14	4	EA	$20,000	$80,000
Selective Coating/Touchup Coatings in Pipe Gallery (Allowance)	1	LS	$100,000	$100,000
Concrete Repair Filter Influent Flume	1	LS	$30,000	$30,000
Selective Valve Replacement (Allowance)	1	LS	$100,000	$100,000
Pilot Testing (Optional)	1	LS	$50,000	$50,000
Service Water Pump Replacement (West PS)	2	EA	$25,000	$50,000
General Allowance	1	LS	$100,000	$100,000
Subtotal				**$1,417,000**

West Water Treatment Plant Rehab plus Rehab of Two Filters w/ Dual Media				
Description	Quantity	Unit	Unit Cost	Total Costs
General Requirements		12%		$170,040
Subtotal				**$1,587,040**
Contingency		20%		$317,408
TOTAL CONSTRUCTION COSTS				**$1,904,448**
Engineering, Legal, Administration		20%		$380,890
TOTAL CAPITAL COSTS				**$2,285,000**

Site Condition Assessment Field Notes

Pre-walk Discussion:

1. Rick has seen 75 MGD through the West Plant with the "bypass" open. The "bypass" completely bypasses all treatment and used to be done on a regular basis up to the late 1980's when flows reached approximately 70 MGD. This obviously is no longer an operational procedure that is practiced (due to regulations) and the bypass was physically removed after the East Plant was constructed. This data point is of significance as expanding the west plant to 72 mgd would require modifications to hydraulic capacity not previously accounted for in previous studies.
2. If LWS were to treat 72 MGD through West Plant, they would set a new energy demand or have to use the West Pump Station. The West Plant transfer pumps would have to be replaced to get the capacity needed.
3. If they did this without running West Pump Station, they'd have to run HS Pumps 5, 6, or 7 and would add another 2.5 MW of power.
4. In addition, it may be difficult to get 72MGD of groundwater to the West Plant; in 1973, they had 44 vertical wells which allowed them to get this capacity, but now they only have 40 and are rehabilitating 17 vertical wells.
5. From historical operational experience Staff believes a new aerator will be required to hydraulically get from 60 MGD to 72 MGD, or else they may have to bypass a portion of the flow around the existing aerators.
6. Running between 60-72 MGD would make it very difficult to backwash any filters.
7. Running the vertical wells at rates over 60 mgd would only work for about 4 weeks as the vertical well production would fall off quickly.
8. Staff would like to have a West Plant surface water CT calculator developed in case it is needed in an emergency. Existing groundwater CT calculator is only setup for CT through contact basins. New surface water CT calculator would have to go through filters and clearwells which may be complicated with how water flows through that plant on both ends.
9. West Plant filters have original media and are good at removing iron and manganese (1930-1956)

10. Filters 1-10 are probably clay tile underdrains, 1956 filters are probably pipe lateral underdrains.
11. None of the filters have air scour. Filters 3-10 do not have surface wash, but the rest of the filters do.
12. All filter valve actuators are pneumatic; drain valves are getting replaced with new vane style actuators.
13. Compressors were new in 1995, overhauled in 2005, overhauled again in 2015, need replaced in 2028. The overhauls cost about $6k/compressor.
14. New receivers were installed in 2017
15. All air piping is original
16. West Plant HVAC is all natural gas, Rick tends to think that is a weakness, but if electric, that would add to demand.
17. The last few valves/actuators were installed in 1980.
18. All electric lineups were 2400v and 480v MCCs have been replaced in 1995.
19. Only one surface wash pump was replaced in 1995.

West Plant Walkthrough:

1. Filters 1-10: LWS would like to close off filter chambers/observation flood from the gallery to help prevent the wet chlorine vapor from corroding elsewhere.
2. All new coatings are needed for ferrous surfaces, floors should probably just have coatings removed as they have a base of red wax paint that they cannot get any coatings to adhere to very well.
3. A few windows/frames/louvers need reglazing or weatherproofing as several leak.
4. The filter area's HVAC had new makeup air units installed in 1995, most gas heaters have been replaced, dehumidification was added in 2010 and has improved the conditioning of the space.
5. Filters 11-14 filter surface wash pipe needs to be replaced.
6. Filters 11-14 filters hit headloss limit faster than F1-10 during the summer (probably because more water run through them as they are closer to the influent hydraulically).
7. For the most part, filter piping is original. It was recoated about 7-8 years ago, but could use an overcoat soon to prevent a total recoat.
8. Filters 11-14 flume had experienced cracking previously and was sealed. A few of the cracks are leaking again and the bottom side of the concrete (exposed in the pipe gallery) is exhibiting spalling in several locations. Rebar is being exposed at those spalling locations.
9. Chlorine analyzers in pipe gallery were replaced two years ago. At that time, they just needed spare parts, but Hach said they no longer serviced the older units and LWS was forced to buy new units.
10. North High Service (NHS) pump 3 was replaced about 8 years ago and had an AFD added at that time.
11. NHS pump 4-6 are original to 1956 installation, motors are also original. The bearings were replaced when they reached 30-40 years of age.

12. NHS discharge pipe and ball valves on pump 3-6 discharge were replaced 10 years ago, the actuators were replaced at the same time.
13. NHS pump 2 has original (1995) valve and actuator.
14. NHS pump 1 has new valve, but original valve actuator.
15. The filter operating floor, transfer pump room, and NHS pump room all have separate air handlers that were new in 1995.
16. The NHS air handler has heat, but no cooling capacity, but the room has additional fans/louvers to help cool if needed.
17. The overhead crane in the NHS pump room had a new crane installed on the trolley about three years ago.
18. Windows and doors in NHS pump room are in good condition.
19. All main electrical gear in NHS are was installed in 1995. New conduit and conductors were run to all existing loads at that point.

Ozone Building Walkthrough:

1. Both the ambient and high concentration ozone analyzers have had issues with degradation and then shutting down the generator.
2. Ozone contact basin drain valves have had issues.
3. The air compressor for the ozone system has had scrolls replaced, drier media replaced, and that has helped dry the air. LWS had been experiencing a lot of condensation drainage prior to that.
4. The destruct units have had the catalyst replaced.
5. Ozone analyzers near the destruct units should be replaced.
6. LWS needs to start running the cooling water a day or so prior to starting up the generator. They had previously modified the service water to the cooling water system to run at a lower flow rate as it had been wasting a lot of water. If the generators have been off for a period of time and try to restart when the cooling water tank is at ambient temperature, the service water cannot cool the cooling water tank fast enough and the generators will shutdown on over temperature.

East Plant Walkthrough:

1. Filter pipe should be touched up and overcoated within the next 5 years so a total blast and recoat isn't required in 10 years.
2. East Plant exterior needs control and expansion/contraction joints repaired. Maintenance has done some of them near the main entrance.
3. The roof drain piping through the filter operating area has condensation on it continually and has caused it to corrode.
4. A few of the hollow core roof panels appear to have leaked; it is suspected that there was moist air condensing inside of the hollow core and then leaking out and not an actual leak from outdoors to indoors.

South Pump Station/Reservoir

1. The pump discharge pipe runs through the wetwell and needs to be coated within the next 2 years (this is already in the CIP).
2. HVAC controls are no longer supported and many have been disconnected. Used to be more of an automatic system, but many controls have been removed and they are now using manual controls.
3. The displays on the electrical equipment were very difficult to read (may just need a brightness adjustment?).

West Transmission Pump Station:

1. Engines had new injectors installed 3 years ago, the emissions/catalytic converters had been gone through before.
2. The right-angle gears and pumps were replaced in 2004 on both engine driven units.
3. The electric powered pump is original to the facility.
4. The existing engine powered units average 100 hours of use per year and burn around 95 gal/hr of diesel.
5. There are several service water valves in the basement that have packing that are leaking. It is suspected that these valves are some API type and the parts are no longer available.
6. There is a large ball valve on the east engine powered pump discharge that has been known to cut its seat due to its construction. After the seat is cut, the valve continues to leak. No known issue currently, but parts availability is unknown.
7. Diesel storage tank is scheduled to be recoated.