Water Master Plan

Authorization No. 75

City of Smithville, Missouri

June 1, 2018

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1 Executive Summary

This Water Master Plan has been developed for the City of Smithville, Missouri. The Master Plan summarizes HDR's assessment of the City of Smithville's water supply system from the Smithville Lake through the Raw Water Pump Station, Water Treatment Plant and the Distribution System including transmission mains, booster pump stations and elevated storage tanks. HDR's findings and recommendations are summarized below.

1.1 Water Demand

Population growth and water demand projections are summarized in Table 1.1 below.

Table 1-1 Water System Demands and Population Growth

Smithville Water System Demand Projections					
Year	**MARC Population**	**Connections**	**Average Day Demand (gpm)**	**Max Day Demand (gpm)**	**Peak Hour Demand (gpm)**
2016	12,000*	4,138	833 (1.2 MGD)	1,417 (2.04 MGD)	2,479 (3.57 MGD)
2028	16,000	5,517	1,111 (1.6 MGD)	1,889 (2.72 MGD)	3,306 (4.7MGD)
2038	19,074	6,577	1,325 (1.91 MGD)	2,252 (3.24 MGD)	3,941 (5.67 MGD)

*Current population is approximately 10,000

The Raw Water Supply Facilities Water Treatment Plant and pipe distribution system must have sufficient capacity to provide Average Day and Maximum Day water Demands. Peak Hour Demands are met by the elevated water storage facilities in the Distribution System.

1.2 Raw Water Supply

The Raw Water supply system consists of water stored in Smithville Lake, the supply pipeline between the Lake and the Raw Water Pump Station, the Raw Water Pump Station, and the raw water transmission main between the raw water pump station and the water treatment plant.

1.2.1 Smithville Lake

The City of Smithville has a contract with the Federal Government for a supply of 2,000 Acre Feet (AF) per year which calculates to 652 Million Gallons per year. Based on Average Day Demand projections, the City will need to contract with the government for additional raw water supply before the year 2033 when average day demands are projected to exceed the supply. The actual amount of storage needed will need to be reviewed at the time but it will not likely be more than 2,000 AF, which will double the current supply. The Lake has ample storage allocated for water supply up to 92,500 AF and has only two customers at the present time with a total allocation of 13,500 AF.

1.2.2 Raw Water Supply Pipeline

The 12” raw water supply pipeline delivers water from the Lake intake to the Raw Water Pump Station. At a velocity of 7 feet per second (fps) the capacity of the raw water pipe is 2,467 gallons per minute (gpm) or 3.55 Million Gallons per Day (MGD). It is recommended that pressure and flow tests be conducted to confirm the capacity calculations. The calculated pipeline capacity is greater than the projected maximum day demand (MDD) of 2,252 gpm (3.24 MGD) for planning year 2038. The calculations indicate improvements to the raw water supply pipeline, upstream of the Raw Water Pump Station are not required.

The United States Army Corps of Engineers (USACE) has requested the City to provide inspection access to the outside of the existing raw water supply pipeline isolation valve, which is located within the toe of the dam, upstream of the Raw Water Pump Station. Due to complications associated with proximity to the dam, new piping, valves and vaults are recommended at an estimated cost of $366,000. This work will require an excavation in excess of 15 feet deep within the toe of the dam, requiring several meetings with USACE to design, coordinate, and construct the improvements.

1.2.3 Raw Water Pump Station

According to City staff, the duty vertical turbine pump at the Raw Water Pump Station is has a maximum available capacity of 1,450 gpm (2.09 MGD). This capacity is 286 gpm less than the Water Treatment Plant’s current design capacity of 2.5 MGD. Although the pumping capacity is limited, the duty vertical turbine pump provides enough capacity to meet the 2019 MDD of 1,450 gpm (2.09 MGD). The standby pump is a submersible pump that has not been operated in some time and operates at a lower head and flow range. It cannot be operated with the duty pump, and is unreliable. Therefore the firm capacity is 1450 gpm or 2.09 MGD.

The Raw Water Pump Station flooded two times in 2017, requiring replacement of some of the electrical equipment. Due to the very small physical size of the station, location within the flood plain, and the inadequate supply capacity, HDR recommends replacement of the pump station to a location above the flood plain. Replacement with a new raw water pump station is estimated at $1,659,000. It is further recommended that the City begin preparing plans for a new raw water pump station this year, given the fact that this pump station is the weakest link in the supply chain, and is not projected to have adequate capacity within the next year or two. An optional immediate improvement would be to replace the existing pump with a higher head pump capable of 1,800 gpm to extend the time frame for replacement of the raw water pump station building by three or four years.

Recently zebra mussels have been discovered on the pump intake. The mussels reduce the pump capacity and need to be controlled with chemical. The chemical needs to be injected upstream of the pump station. The estimated cost to inject chemicals upstream of the pump is $20,000.

The Raw Water Supply Improvements are outlined in Table 1-2 Raw Water Supply Improvements

1.2.4 Raw Water Transmission Main

The 12” raw water transmission main between the pump station and water treatment plant, is calculated to have a capacity of 2,467 gpm or 3.55 MGD which exceeds the projected MDD of 2,252 gpm or 3.24 MGD for the planning year 2038. Therefore, no improvements are planned.

Table 1-2 Raw Water Supply Improvements

Item No.	Description	Initiation Year	Completion Year	Estimated Cost *
1	Zebra Mussel Control	2018	2018	$20,000
2	Raw Water Pump Station Replacement	2018	2019	$1,659,000
3	Valve Box at Dam	Based on USACE Schedule		$366,000
4	Negotiate Additional Water Storage in Smithville Lake	2031	2033	$2,000,000
Total Raw Water Supply Improvements				**$4,045,000**

*Note: Estimate Cost is in 2018 $

1.3 Water Treatment Plant

The Water Treatment Plant (WTP) has a design capacity of 2.5 MGD. Recommended improvements have been separated into two categories, Maintenance Improvements and Capacity Improvements.

The Maintenance Improvements include a new lime feed system, miscellaneous piping and site Improvements, chemical feed building improvements, and removal of residuals. These improvements provide for employee safety, MDNR standards compliance, and to prevent damage to other facilities. Maintenance Improvements are listed in Table 1-3 Water Treatment Plant Maintenance Improvements.

Table 1-3 Water Treatment Plant Maintenance Improvements

No.	Description	Year Initiated	Year Complete	Estimated Cost *
1	Replace Lime Feeder	2018	2018	$100,000
2	Miscellaneous Plant Improvements	2020	2020	$101,000
3	Chemical Feed Building Improvements	2020	2021	$235,000
4	Residual Removal	2025	2026	$594,000
Total Water Treatment Plant Maintenance Improvements				**$1,030,000**

*Note: Estimate Cost is in 2018 $

Capacity Improvements are identified in Table 1-4 Water Treatment Plant Capacity Improvements and include new primary and secondary basins as well as new filters and high service pumping to bring the future capacity of the plant to 5 MGD. Project initiation and completion dates are also provided. These dates are based on scheduling implementation prior to demands exceeding available capacity and should be updated as demands change.

Table 1-4 Water Treatment Plant Capacity Improvements

No.	Description	Year Initiated	Year Complete	Estimated Cost *
1	New Primary and Secondary Settling Basins	2020	2023	$5,450,000
2	New Filter and High Service Building	2020	2023	$4,200,000
Total Water Treatment Plant Capacity Improvements				**$9,650,000**

*Note: Estimate Cost is in 2018 $

The future water demands will exceed the transfer pump capacity in 2023 and will exceed the Primary Basin, Filters and High Service Pump Capacities in 2025. As part of these improvements, new Rapid Mix Basins, Chemical Feed Equipment and Secondary Settling Basins will help control taste and odor issues.

1.4 Water Distribution System

A new hydraulic model was developed for the Water Distribution System, which includes Elevated Storage Tanks, Booster Pump Stations and Transmission Mains. The existing Distribution System is divided into three pressure zones due to elevation changes across the City. The Central Zone includes the water treatment plant and the down town area along the Little Platte River. The other two zones are the South and North Pressure

Zones. Figure 5-1 in Appendix C illustrates the distribution system and the various pressure zones.

Existing water demands and large water users were included in the water model and analyzed in order to determine the improvements needed to ensure adequate capacity, pressure and fire flow. City staff assisted in estimating where future growth is likely to occur over the next 10 and 20 years, as well as the magnitude of the associated new water demand. The model reflects these future demand projections. Table 1-5 WTP to North Tower Water Main Improvements, Table 1-6 Downtown Water Main Improvements, and Table 1-7 South Booster Station and Water Main Improvement south booster station and water main improvements define the distribution system improvements needed and the recommended implementation dates.

Table 1-5 WTP to North Tower Water Main Improvements

Map ID	Description	Year	Pipe Dia.	Length (feet)	Estimated Cost *
C1	River Crossing – Main St to 3rd St	2020	12"	2,560	$461,000
C2	Maple Ln, Highway F to Maple Ln	2021	12"	1,180	$212,400
C3	Helvely Park Dr, WTP to Liberty Rd	2026	12"	3,280	$590,400
C4	Hwy F, East Pope Ln to North Tower	2030	12"	3,650	$657,000
N1	188th St, Primrose to Wildflower St	2031	8"	700	$84,000
Total Water Treatment Plant to North Tower					**$1,263,800**

*Note: Estimate Cost is in 2018 $

Table 1-6 Downtown Water Main Improvements

Map ID	Description	Year	Pipe Dia.	Length (feet)	Estimated Cost *
C5	Main St, Bridge St to River Crossing	2027	8"	1,180	$141,600
C6	Main St, River Crossing to Liberty Rd	2025	12"	625	$112,500
Total Downtown Water Main Improvements					**$254,100**

*Note: Estimate Cost is in 2018 $

Table 1-7 South Booster Station and Water Main Improvement

Map ID	Description	Year	Pipe Dia.	Length (feet)	Estimated Cost *
C8	Hwy 92, Commercial Ave to 169 Hwy	2022	8"	1,230	$147,600
C9	169 Hwy, Highway 92 to Park Dr	2023	12"	1,500	$270,000
S1	Tower Interconnect Armory Rd, 69 Hwy	2022	12"	85	$15,300
S2	Interconnect Mains at 144th St and 169 Hwy	2022	12"	100	$18,000
S3	169 Hwy, Commercial to 144th St	2028	12"	2,725	$490,500
S4	169 Hwy, 144th St to Southwest Tower	2029	12"	2,590	$466,200
S5	New South Booster Pump Station	2024	NA	NA	$1,500,000
Total South Booster Pump Station and Water Main Improvements					**$2,907,600**

*Note: Estimate Cost is in 2018 $

Total estimated cost in 2018 dollars for all recommended Distribution improvements is estimated to be $4,425,500.

The highest priority is Item C1 the River Crossing between Main Street and 3rd Street in Table 1-5 followed by C2 Improvements on Maple Lane and then improvements around the South Booster Pump Station S1, S2, and C8 listed in Table 1-7.

In general, the remainder of the projects are demand driven and will need to be constructed as demands and development dictate. The estimated time frames provided in the tables are for the City's capital improvement planning. The proposed projects were developed in the hydraulic model and are aimed at improving flow, pressure and water quality in the distribution system.

In addition to the proposed projects, driven by capacity improvements, the City should also allocate funding to continue to replace aging cast iron water lines in the downtown area susceptible to breakages due to the higher pressure in the central pressure zone. Typically these projects cost around $250,000 each.

1.5 Summary of Recommended Improvements and Costs

Table 1-8 Water System Improvements Cost Estimate summarizes the total cost of the proposed improvements over the next 20 years.

Table 1-8 Water System Improvements Cost Estimate

No.	Description	Estimated Cost *
1	Raw Water Supply	$4,045,000
2	Water Treatment Plant Maintenance Improvements	$1,030,000
3	Water Treatment Plant Capacity Improvements	$9,650,000
4	Distribution Improvements	$4,425,500
Total Recommended Improvements 2018 to 2036		$**19,150,500**

*Note: Estimate Cost is in 2018 $

HDR recommends the City consider applying for low interest loans from the Missouri State Revolving Fund (SRF) to help fund the improvements.

2 Introduction

The City of Smithville (City) was incorporated in July of 1868. The City owns and operates the water supply system, which includes an intake on Smithville Lake, a raw water supply pipeline, a raw water pump station and transmission line, a surface water treatment plant, and a finish water distribution system including finish water pumping stations and finish water storage facilities.

2.1 Scope of Work

The City of Smithville contracted with HDR Engineering to perform the following tasks:

1. Demand Projections
   - Update water demand projections
   - Review last 5 years water usage
   - Determine per capita demands, maximum day demands and peak hour demands
   - Meet with city on demand projections
   - Project future demands for 10 years and 20 year growth

2. Raw Water Supply Assessment
   - Conduct field visit of the facilities
   - Review existing water supply capacity
     - Contracted lake storage and capacity for 10 and 20 year demands
   - Evaluate the raw water transmission main
     - Provide 10 and 20 year flow projections
   - Evaluate the raw water booster pump station for current and future demands
   - Develop improvement alternatives to meet future 10 and 20 year demands
   - Evaluate raw water supply reliability

3. Water Treatment Plant Assessment
   - Conduct field visit
   - Determine current average and peak treatment capacity for each process
   - Develop improvement alternatives to meet current needs
   - Develop improvement alternatives to meet future 10 and 20 year demands
   - Prioritize recommended improvements and provide estimates of probable costs

4. Distribution System Assessment Using Hydraulic Model
   - Summarize current distribution system
   - Update hydraulic model based on locations of existing water mains
   - Calibrate water model using up to 6 hydrant flow/pressure tests
   - Identify areas of potential growth
   - Evaluate model for 10 and 20 year maximum and peak hour demands
   - Identify areas of low pressure and determine available fire flow
   - Evaluate areas with frequent water main breaks
   - Evaluate existing storage
   - Evaluate existing booster pump stations for 10 and 20 year demands
   - Identify and prioritize system improvements and estimate costs

- Prepare updated water distribution system maps

5. Water Master Plan Report
    - Summarize the above tasks into a water master plan in accordance with MDNR requirements
    - Estimate probable costs for current 10 year and 20 year improvements

6. Meet with City and Finalize the Master Plan Report

7. Present Report to Board of Alderman

This Water Master Plan represents the summation of HDR's evaluation of the City of Smithville's Population Projections, Water Demand projections, as well as our analysis of the Raw Water Supply Facilities, Water Treatment Plant, and Distribution System.

2.2 Background

The original components of the existing water treatment plant were constructed in the 1970's. Various modifications were completed in 1993. The water system is managed by the Smithville Utilities Department, which provides high-quality and safe potable drinking water to the City of Smithville and the surrounding area.

The Missouri Department of Natural Resources (MDNR) and The Environmental Protection Agency (EPA) regulate the water treatment plant (WTP) operations. The plant must provide drinking water that meets the design and quality standards set by the MDNR and EPA. The WTP is permitted by the MDNR to provide 2.5 million gallons of water per day (MGD) to the City's customers. Currently the plant provides 1.1 MGD on an average day and up to 1.87 MGD on a peak or maximum day.

3 Population and Water Demand Projections

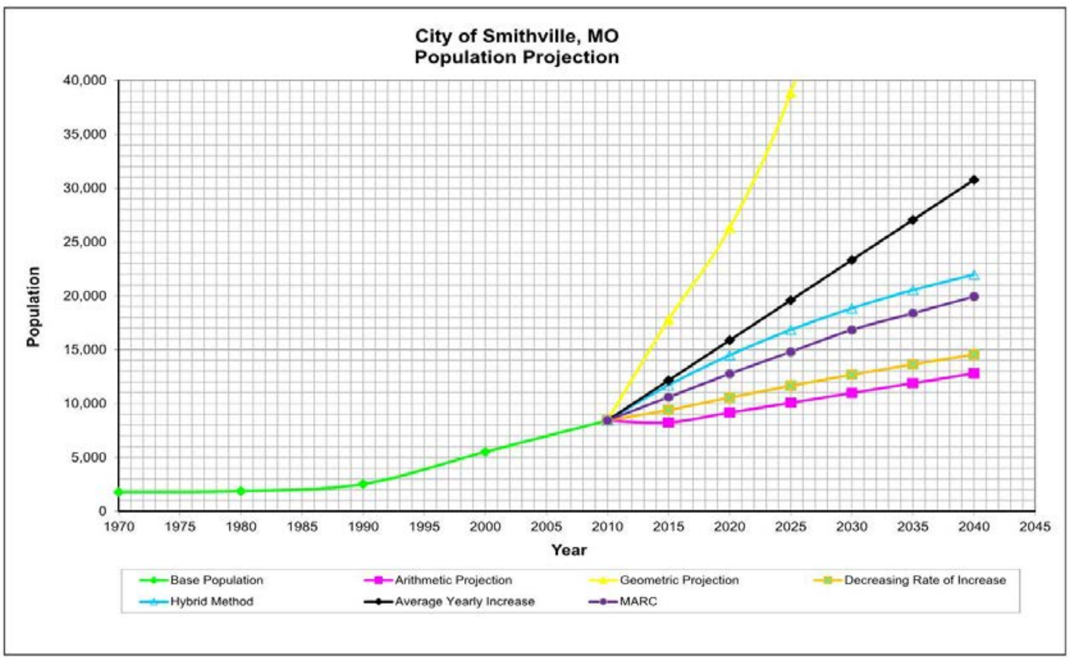
3.1 Population and Historical Water Use

Historical population growth and water usage trends were evaluated to develop future water demands.

3.1.1 Population Projections

Several population projection methods were evaluated, including an arithmetic projection, geometric projection, decreasing rate of increase, hybrid, average, and the Mid-America Regional Council (MARC) method. See Figure 3.1 Smithville Population Projection Graph below for a graphical comparison of the various population projections.

Figure 3.1 Smithville Population Projection Graph



The Mid-America Regional Council (MARC) population projections had the best correlation with the historical growth patterns of the City and was therefore selected as the basis for the City's projected population growth over the design period.

3.1.2 Historical Water Use

Historical water use for the proceeding five-year period was evaluated to determine a per capita water use. Raw water pumping rates were used as the basis for the historical water demand since it is metered and more accurate than the other available flow data sources, such as pump run times. The per capita water usage includes water loss and consumption from commercial/industrial customers.

Annual meter billing data and population estimates were used to approximate the average number of people per meter at 2.9 capita/meter. The average day water usage over the period 2011 to 2015 is 0.945 million gallons per day with an unaccounted water rate of 24%. See Table 2.1 for summary of historical water use.

Table 3-1 - Five-Year Historical Water Use

Five-Year Historical Water Use						
Raw Water Pumping Data						
Year	**2011**	**2012**	**2013**	**2014**	**2015**	**Average**
Total Pumpage (gal)	323,216,091	384,173,269	366,487,635	344,169,382	304,217,728	344,452,821
Avg Day (gpd)	885,524	1,049,654	1,004,076	945,520	838,065	944,568
Max Day (gpd)	1,597,019	1,735,607	1,611,932	1,502,201	1,371,758	1,563,703
Min Day (gpd)	529,270	706,335	107,746	509,126	601,204	490,736
MXD/ADD factor	1.80	1.65	1.61	1.59	1.64	1.66
Min/ADD factor	0.60	0.67	0.11	0.54	0.72	0.53
Meter Billing Data						
Year	**2011**	**2012**	**2013**	**2014**	**2015**	**Average**
Total Metered (gal)	260,527,046	291,301,403	264,546,339	248,798,139	243,000,168	261,634,619
Avg Day (gal)	713,773	795,905	724,784	681,639	665,754	716,371
Usage Per Person and Unaccounted Water						
Year	**2011**	**2012**	**2013**	**2014**	**2015**	**Average**
Avg meter count	3,302	3,324	3,365	3,417	3,463	3,374
gpd/meter (metered)	216.2	239.5	215.4	199.5	192.2	212.6
gpd/meter (pumped)	268.2	315.8	298.4	276.7	242.0	280.2
Population	8,859	9,292	9,726	10,159	10,593	9,726
capita/meter	2.68	2.80	2.89	2.97	3.06	2.88
gpdpc (metered)	80.6	85.7	74.5	67.1	62.8	74.1
gpdpc (pumped)	100.0	113.0	103.2	93.1	79.1	**97.7**
Unaccounted Water	**19.4%**	**24.2%**	**27.8%**	**27.7%**	**20.1%**	**23.8%**

A value of 100 gallons per day per capita (gpd/capita) was calculated and used to project a future total water demand for each pressure zone based on the projected population within the pressure zone.

3.2 Demand Projections

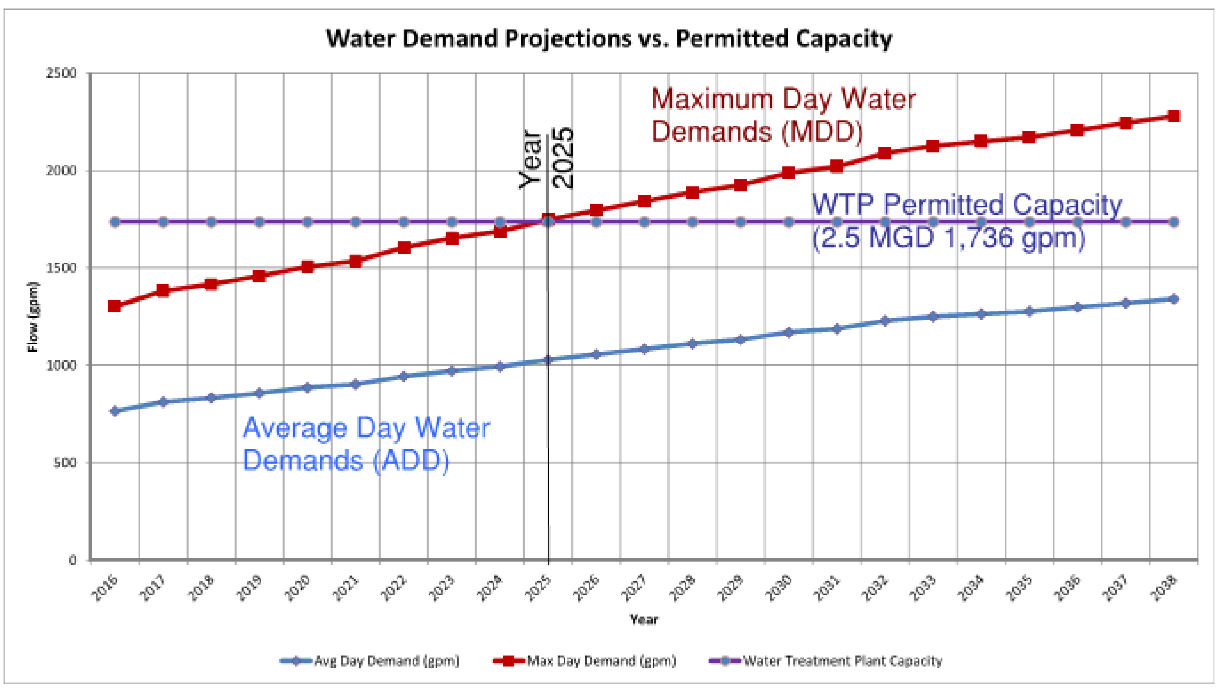
Using a value of 100 gpd per capita and 2.9 capita per connection, future system demand projections were calculated using the MARC population projections. See Table 3-2.

Table 3-2 - Smithville Water Demand Projections

Total Smithville Water System Demand Projections					
Year	**MARC Population**	**Connections**	**Average Day Demand (gpm)**	**Max Day Demand (gpm)**	**Peak Hour Demand (gpm)**
2016	11,027	3,802	766 (1.1 MGD)	1,302 (1.87 MGD)	2,278 (3.28 MGD)
2028	16,000	5,517	1,111 (1.6 MGD)	1,889 (2.72 MGD)	3,306 (4.7MGD)
2038	19,074	6,577	1,325 (1.91 MGD)	2,252 (3.24 MGD)	3,941 (5.67 MGD)

Figure 3.2 below illustrates the Average Day and Maximum Day Demand projections in relation to the existing plant permitted capacity of 2.5 MGD. The projections predict that the Maximum Day Demand will exceed the existing plant permitted capacity in the year 2025.

Figure 3.2 Water Demand Projection Graph vs. Permitted Capacity



In addition to population projections for the City as a whole, HDR worked with City staff to distribute the projected population growth between each of the three water pressure zones within the water distribution system. This data was used in the hydraulic models to distribute future flow and more accurately analyze the impact on pump stations, elevated tanks, and distribution mains within each pressure zone. See Table 3-3, Table 3-4, and Table 3-5 for a breakdown of population projections by zone.

Table 3-3 - North Pressure Zone Population and Water Demand Projections

North Pressure Zone Demand Projections					
Year	**MARC Population**	**Connections**	**Average Day Demand (gpm)**	**Max Day Demand (gpm)**	**Peak Hour Demand (gpm)**
2016	4,365	1,505	303 (0.44 MGD)	515 (0.74 MGD)	902 (1.3 MGD)
2028	5973	2060	415 (0.60 MGD)	705 (1.01 MGD)	1,234 (1.78 MGD)
2038	6952	2397	483 (0.70 MGD)	821 (1.18 MGD)	1,436 (2.07 MGD)

Table 3-4 - Central Pressure Zone Population and Water Demand Projections

Central Pressure Zone Demand Projections					
Year	**MARC Population**	**Connections**	**Average Day Demand (gpm)**	**Max Day Demand (gpm)**	**Peak Hour Demand (gpm)**
2016	4,488	1,547	312 (0.45 MGD)	530 (0.76 MGD)	927 (1.33 MGD)
2028	5,602	1,932	389 (0.56 MGD)	661 (0.95 MGD)	1,157 (1.67 MGD)
2038	6,144	2,119	427 (0.61 MGD)	726 (1.04 MGD)	1,269 (1.83 MGD)

Table 3-5 - South Pressure Zone Population and Water Demand Projections

South Pressure Zone Demand Projections					
Year	**MARC Population**	**Connections**	**Average Day Demand (gpm)**	**Max Day Demand (gpm)**	**Peak Hour Demand (gpm)**
2016	2,175	750	151 (0.22 MGD)	257 (0.37 MGD)	449 (0.65 MGD)
2028	4,225	1,526	307 (0.44 MGD)	522 (0.75 MGD)	914 (1.32 MGD)
2038	5,978	2,061	415 (0.60 MGD)	706 (1.0 MGD)	1,235 (1.78 MGD)

Notes:
1. Assume 2.9 capita per connection (meter)
2. Assume 100 gpd per capita, which includes commercial/industrial usage and 24% water loss.
3. Max Day/Avg Day factor of 1.7
4. Max Day/Peak Hour factor of 1.75

3.2.1 Large Customers

The top ten largest customers were identified and located to ensure those locations were properly allocated within the hydraulic model. When the large customer demands were placed in the hydraulic model, the other demands within each pressure zone were adjusted to maintain the 100 gpd/capita average for the zone. Large customer demands were not expected to grow through the analysis period. Average day, maximum day and peak hour demands for each large customer are shown in Table 3-6.

Table 3-6 - Top Ten Large Customers

Top Ten Large Customers										
	PWSD # 8	**Clay County Parks Hwy DD**	**Beverly Enterprises**	**St. Luke's Hospital**	**Clay County Parks Hwy F**	**Housing Authority**	**Smithville Park**	**Lucid Lodging**	**McDonalds**	**Sonic**
Annual Average (gpm)	60.62	15.32	9.22	7.49	5.53	4.21	6.50	2.59	1.94	2.08
Max Day gpm (Avg Dayx1.7)	103.05	26.05	15.68	12.74	9.41	7.16	11.04	4.40	3.30	3.53
Peak Hour gpm (Max Day x1.75)	180.34	45.58	27.44	22.29	16.46	12.54	19.33	7.70	5.77	6.18

4 Supply and Treatment Facilities

The City of Smithville's drinking water supply and treatment facilities are comprised of a surface water source at Smithville Lake; a raw water pipeline; a raw water pump station and transmission main; and a 2.5 MGD conventional surface water treatment plant (WTP). This section describes these components, evaluates the existing conditions, and recommends improvements necessary in order to meet regulatory requirements and provide for projected water demand through the 20-year planning period (2018 to 2038).

4.1 Raw Water Supply

The City receives its surface water supply from Smithville Lake. Raw water is pumped directly from the Lake by the Raw Water Pump Station to the WTP for treatment. The following sub-sections describe the raw water supply system condition and capacity.

4.1.1 Raw Water Quality

Water operators run daily raw water quality tests. The parameters analyzed are presented in Table 4.1. In general, the source raw water is of good quality and provides sufficient capacity. However, similar to other surface water reservoir conditions in the region, certain raw water quality parameters can vary significantly and generally unexpectedly during any given year, requiring WTP personnel to adjust chemical doses or treatment regimens.

Low water temperatures typically cause chemical reactions to occur slower compared to warmer temperatures. High hardness, manganese, and turbidity levels lead to precipitation, resulting in more solid waste. Raw water with low alkalinity, like Smithville Lake, has little buffering capacity requiring additional chemical feed to improve drinking water quality. With surface water sources, taste and odor issues arise when the lake water temperatures change in the spring and fall. Raw water quality is further impacted when there is high turbulence due to reservoir maintenance by USACE on or around the earthen dam.

Alternative source water options are extremely limited for the City. A connection to Kansas City Water Services is likely the only alternative source water option. This will require construction of a large diameter pipeline from the center of town to south of Interstate 435 and an additional booster pump station, or stations. The alternative water source will also require additional improvements within the Kansas City distribution system and the City would also need to pay for additional studies by Kansas City Water Services, to determine the required improvements. The high capital costs and connection fees associated with the new pipe and pump stations needed to convey the water from Kansas City to Smithville preclude this from being a viable option, given the fact that the City currently has an adequate water supply source that can meet future demands well into the future, as well as a working WTP with room for expansion.

In recent weeks Zebra Mussels have been found clogging the raw water pump intakes. Currently sodium permanganate is injected to control the mussels. Long term improvements needed include moving the injection point for the sodium permanganate to a location upstream of the raw water pump station and installing a raw water pipeline flushing hydrant. The USACE has also found that other chemicals have been effective at killing this invasive mussel species. Other chemical alternatives approved for drinking water may need to be investigated further in order to implement the most cost effective solution. Long term control really begins at the Lake intake structure, which will require further coordination and investigation with the USACE.

Table 4-1 Existing Raw Water Quality (2012-2016)

Year	pH		Temperature		Alkalinity		Hardness		Turbidity		Manganese	
	Range	Average	Range	Average	Range	Average	Range	Average	Range	Average	Range	Average
2012	7.7 - 8.7	8.1	1.8 – 28.2	16.4	87 - 102	92	88 - 109	96	0.51 – 15.1	4.69	0.02 – 0.31	0.114
2013	7.6 - 8.7	8.1	3.1 – 27.2	14.8	89 - 103	93	91 - 116	97	1.98 - 17	4.5	0.04 – 0.31	0.117
2014	7.5 – 8.6	8.0	2.0 – 27.5	15.3	85 - 104	92	89 - 109	97	1.35 – 18.2	4.45	0.04 – 0.31	0.111
2015	5.3 – 8.7	7.9	2.6 - 30	16.7	83 - 105	92	85 - 108	96	1.91 – 20.5	6.01	0.03 – 0.52	0.124
2016	7.3 – 8.5	7.9	3.0 – 28.2	17.2	72 - 98	85	76 - 104	89	2.06 – 17.3	7.01	0.04 – 0.26	0.118

4.1.2 Raw Water Source

Smithville Lake is a multi-purpose reservoir constructed by the United States Army Corps of Engineers (USACE) in 1977. The lake is utilized for flood control, drinking water, and recreation for Clay County and the Cities of Smithville, Plattsburg and Kansas City, Missouri. The USACE estimates approximately 102,200 acre-feet (AF) of multi-purpose storage and an additional 92,000 AF of flood control storage. The multi-purpose storage includes 95,200 AF of water supply storage, of which 13,500 AF has been contracted for use by the Cities of Plattsburg and Smithville.

Based on U.S. Geological Survey gage records, it is estimated between October 1988 and December 2015 the reservoir's multi-purpose pool average level lowered approximately 5.3 feet. Figure 4.1 Smithville Lake Daily Water Surface Elevation (1988-2015) shows the historical daily water surface elevation between 1988 and 2015.

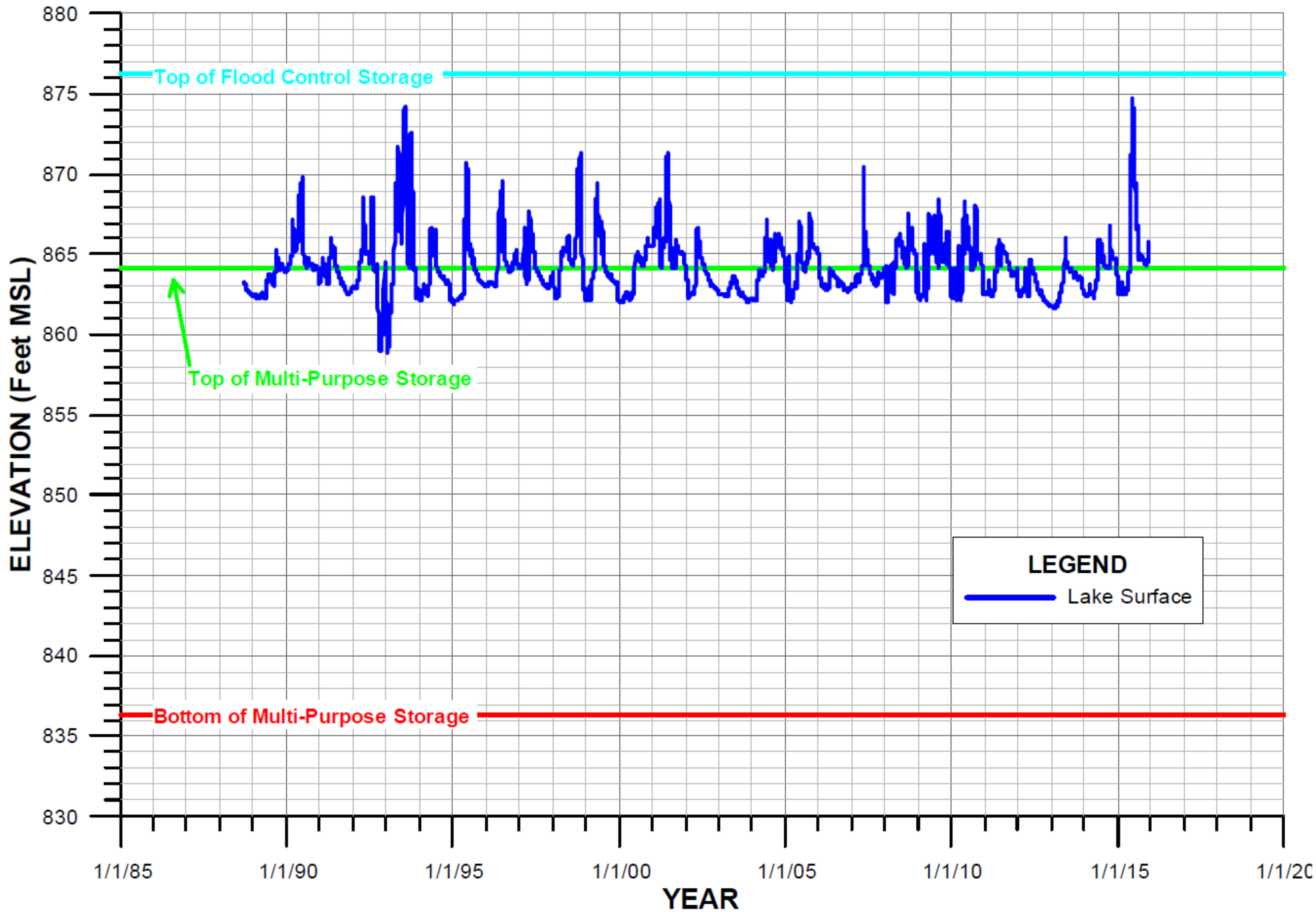
The City executed a contract with the U.S. Government in 1972 for use of a portion of the drinking water supply stored in Smithville Lake (Contract No. DACW41-73-C-0007). The original contract provided the City with 2,000 AF per year (652 MG per year) of usable storage and 6,000 AF per year (1,956 MG per year) of future use storage with a deferred payment plan for the future use storage.

Under the Water Resources Reform and Development Act of 2014 (WRRDA 2014), the City was given the opportunity to be relieved from their contractual obligations to pay for future use storage either by converting the 6,000 acre-feet to current use storage or revert the future use storage back to the U.S. Government. The original reservoir supply contract required the City to pay for the future use storage with interest at the end of the contract period (i.e., 2031). The estimated cost for the future use was estimated to be $6.6 million.

The City retained HDR to evaluate the need for the 6,000 AF of future use storage. As part of this effort, HDR developed future population projections and estimated future water demands. Of the 2,000 AF of current storage, the City requires approximately 1,058 AF per year to meet existing demands. It was determined that the 6,000 AF was more than needed and adding another 1,000 or 2,000 AF would not be required until the year 2031 to meet future demands. Based on future water demands, remaining supply storage capacity and the cost of the water supply storage, the City elected to revert the 6,000 AF of future use storage back to the U.S. Government.

The City maintains contractual rights to 2,000 acre-feet of water from the Lake with the potential to contract for additional drinking water supply in the future. Of the 95,200 AF of storage allocated for water supply, 75,700 AF was not under contract as of 2015. If the City elects to contract for additional supply in the future it will likely require an environmental assessment (EA) by the USACE to analyze potential impacts to the existing authorized purposes of the Lake. Further, the contract for new storage will ultimately require the Assistant Secretary of the Army for Civil Works for approval; however, depending on the amount of storage being requested, the delegation of authority may be authorized at a lower level.

Figure 4.1 Smithville Lake Daily Water Surface Elevation (1988-2015)



4.1.3 Raw Water Supply Pipeline

Raw water from Smithville Lake is conveyed to the Raw Water Pump Station via a 12" raw water pipeline, which is located approximately 1,500 feet west of the dam (see Figure 4.2). The raw water pipeline originates at the intake structure on the east side of the Smithville Lake Dam as a 69-inch diameter pipe. The 69-inch pipe runs a length of 700-feet, reduces to 18-inches for 50-feet, and then, ultimately, reduces to 12-inches for 1,500-feet where it connects directly to the raw water pumps.

An 18" isolation valve is buried within the toe of the Lake's earthen dam at the beginning of the 18" pipeline. The USACE has indicated the valve shall be inspectable inside and outside. Therefor it is recommended to relocate and construct the valve within a concrete vault. The relocation will enable USACE to inspect the condition of the valve and allow the City to maintain the valve as well as protect the dam in the event of a valve failure. This work must be coordinated and approved by USACE prior to initiating any construction activities.

Currently, there is no flow meter or flow measuring device associated with the raw water intake system and there are no pressure gauges. For the purpose of this study, the capacity of the raw water pipeline is calculated using Smithville Lake water elevations, the raw water pipeline sizes and lengths, the raw water pump station configuration and the WTP primary rapid mix basin elevation. Using the minimum water surface elevation of the Lake shown in Figure 4.1 of 859-feet, the Raw Water Pump Station ground elevation of 810-feet and the raw water pipeline varying diameters and lengths, the capacity of the pipeline is estimated to be approximately 2467-gpm (3.55 MGD). This is based on maximizing the raw water pipe line velocity at 7 feet per second (fps) and insuring the hydraulic grade line at the pump station is at least 25 feet above grade (Net Positive Suction Head available is ≥ 25 feet). It is recommended that pressure and flow tests be conducted to confirm these calculations during the next phase of design. The calculated pipeline capacity is greater than the projected maximum day demand (MDD) of 2,252 gpm (3.24 MGD) for planning year 2038.

Estimating the design and construction cost of a new concrete vault constructed around the existing raw water valve is difficult due to the location of the valve in the toe of the dam and the possible liability of potentially weakening the earthen dam to construct a vault. The USACE has had a difficult time reaching consensus amongst their own staff regarding the ability to construct a vault in the toe of the dam. If the valve vault work is required to be performed, it would be preferred to extend either a 16" or 18" pipeline to 500 feet beyond the toe of the dam and install a new valve and valve vault outside of the toe of the dam. For Smithville design capacity purposes, a 16" pipeline would be adequate. This work should be scheduled when the USACE has the raw water pipe shut down. The City could then connect to this pipe at any time in the future without working in the toe of the dam.

The estimated cost to construct two valve vaults and 500 feet of 16-inch pipe is $366,600. The cost estimate can be found in Appendix A, Cost Estimate Tables.

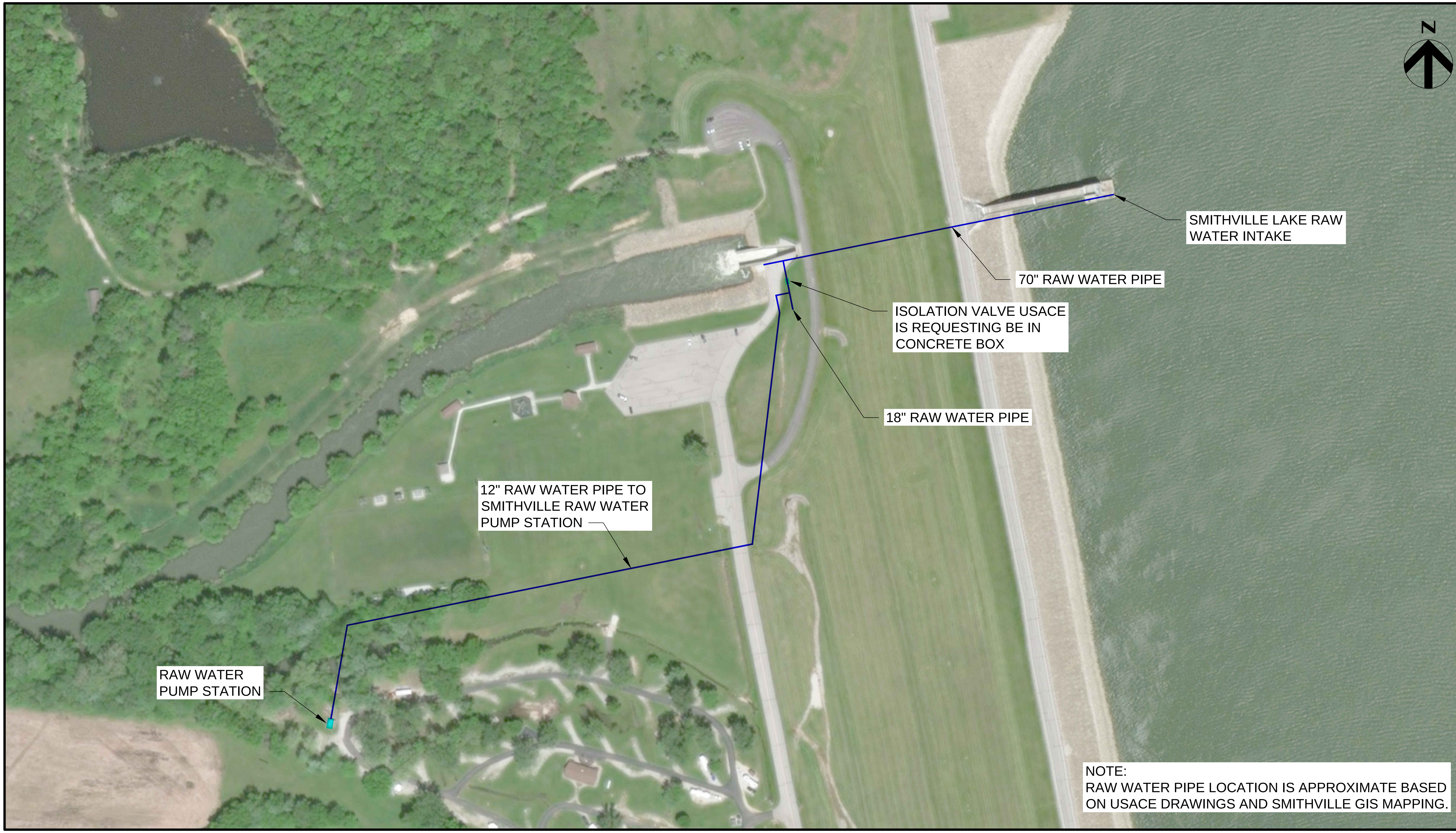


FIG 3-2.dwg
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RAW WATER PUMP STATION AND RAW WATER LINE

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Figure 4-2

4.1.4 Raw Water Pump Station

The Raw Water Pump Station consists of two raw water pumps: a vertical turbine pump (duty) and a submersible pump (standby). The 12-inch raw water main from the lake connects directly to each pumps intake. Table 4.2 presents the design conditions of the raw water pumps. The Raw Water Pump Station transmission main discharges directly to the Primary Rapid Mix Basin located at the WTP. The transmission main is a 12-inch diameter cast iron pipe, approximately 9,100 feet in length.

The Raw Water Pump Station, shown in Figure 4.3 Raw Water Pump Station, is a small (15-foot by 12-foot) concrete block building. In addition to the two raw water pumps, the building contains a 55-gallon drum of sodium permanganate and associated metering pump. The sodium permanganate is added to the raw water to oxidize and precipitate iron and manganese, assist with taste and odor issues, and to provide some zebra mussel control.

An electrical transformer is mounted on a concrete pad adjacent to the building. The building entrance and transformer are located inside a 6-foot tall fence with a locking gate. There is no wet well associated with the Pump Station. There is also no provision for back-up power. Due to different pump operating characteristics only one raw water pump can be operated at a time. The submersible pump provides less flow than the vertical turbine and has less operating head and so both pumps cannot be operated simultaneously. The standby pump is operated so infrequently that for the purpose of this report it is assumed to be unreliable and/or inoperable.

Figure 4.3 Raw Water Pump Station

Table 4-2 Raw Water Pump Design Capacity

Pump	Rated Pump Capacity (gpm)	Rated Pump Capacity (MGD)	Total Dynamic Head (feet)	Pump Horsepower (HP)
Vertical Turbine	1,800	2.59	64	50
Submersible	1,200	1.73	Unknown	40

The Pump Station is located within the FEMA 100-year floodplain. On two separate occasions in 2017, the area surrounding the Pump Station flooded. During each flooding event, the water rose to within inches of the vertical turbine pump motor, ruined the pump station cooling system, and partially submerged the electrical gear. Figure 4.4 Water Mark on Raw Water Pump Station Wall shows the high water mark from a flooding event in 2017.

Figure 4.4 Water Mark on Raw Water Pump Station Wall

According to City staff, the vertical turbine pump has a maximum available capacity of 1,450 gpm (2.09 MGD) this is the firm and maximum capacity of the pump station. This capacity is 286 gpm less than the WTP current design capacity of 2.5 MGD. Although the pumping capacity is limited, the vertical turbine pump does provide enough capacity to meet the 2019 MDD of 1,450 gpm (2.09 MGD). The pump was designed to provide 1,800 gpm or 2.59 MGD.

There are various possible causes for the lower reported pump capacity, as listed below:

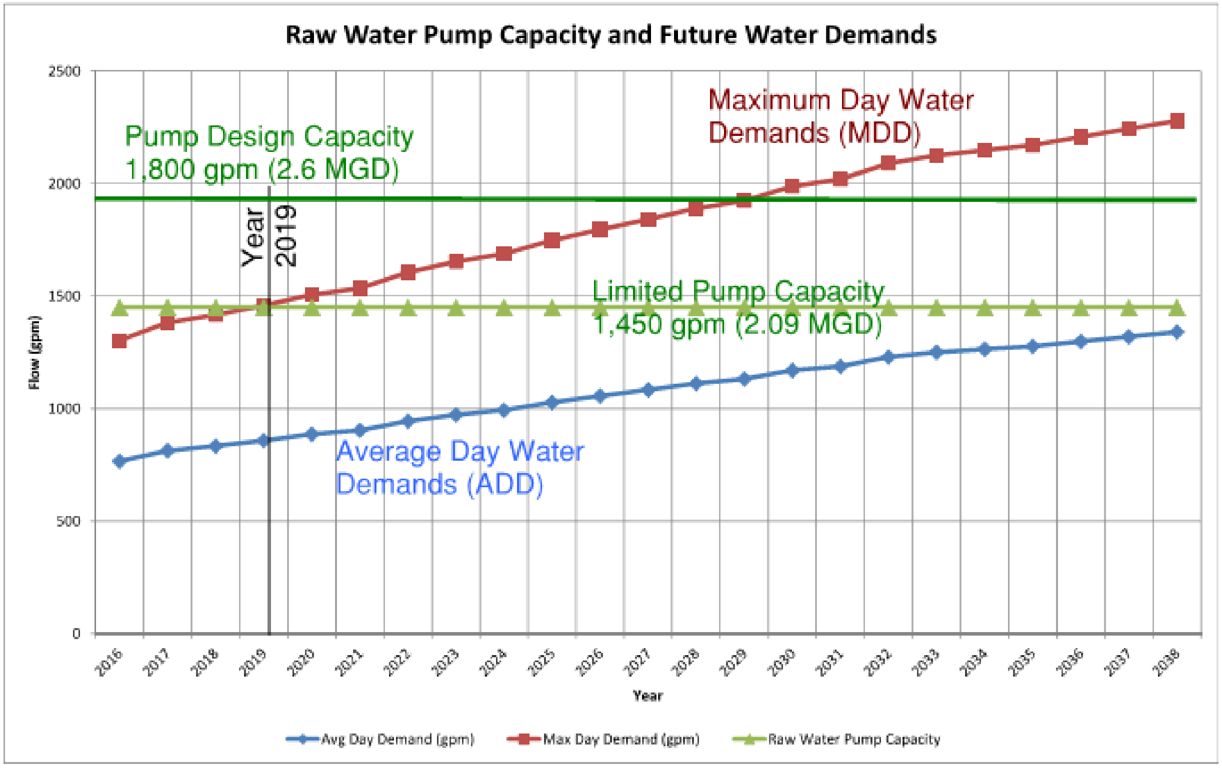
- The pump run time meter or plant flow meter may not be reading accurately;
- The raw water pipeline may be smaller than 12-inches or restricted;
- The raw water supply pipeline between the dam and pump station, or between the pump station and the WTP, may be restricted due to zebra mussels, partially closed valves, sedimentation, corrosion;
- The lifting capacity or total dynamic head rating of the pump may not be adequate to overcome the static and hydraulic conditions; or
- The pump impellers may be worn.

Based on a desktop review of topographic data, existing plant drawings, the static lift from the Raw Water Pump Station to the water level at the WTP is estimated to be minus 6 feet using the Lake minimum water elevation of 859 and the primary rapid mix basin elevation of 853. Assuming a flow rate of 1,800 gpm (2.59 MGD) and a Hazen Williams friction C-factor of 130, the total friction loss in the pipe between the lake and the WTP is estimated to be 74 feet. The pump should be able to deliver 1,800-gpm at 74 feet of head. However plant operators report the pump can only deliver 1,450 gpm (2.09 MGD).

Recently zebra mussels have been discovered on the pump intake. The mussels reduce the pump capacity and need to be controlled with chemical. The chemical needs to be injected upstream of the pump station. The estimated cost is $20,000.

Due to the inability of the raw water pump station to supply 2.5 MGD (1,736 gpm) to the WTP, the flooding potential of the electrical components at the Pump Station, lack of a back-up pump or power source, and the small size of the existing building, this facility should be replaced. The station does not meet MDNR standards because the pumps are not the same size and current electrical losses. At the current pump capacity and the City's estimated future water demands presented in Table 2.2, the pump can only supply the maximum daily demand through 2019 unless improvements are implemented. Therefore raw water pump station improvements are urgent. Figure 4.5 Raw Water Pump Capacity vs Future Water Demand shows the Raw Water Pump Station Capacity and Raw Water Main capacity as compared to average and maximum day water demands.

Figure 4.5 Raw Water Pump Capacity vs Future Water Demand



The estimated cost to replace the Raw Water Pump Station is $1,659,000. The cost estimate is included in Appendix A, Cost Estimate Tables. The cost estimate accounts for either constructing a new pump station at a new location at higher grade (above the 100-year flood elevation and known historical flood elevations) or in the same vicinity, but elevating the mechanical and electrical components. It is recommended that the station be provided with two pumps initially, one duty and one standby, each rated at 1,736 gpm or 2.5 MGD. The station should be sized to allow for the installation of a third pump in the future, in order to provide a future firm capacity of 5 MGD. The pumps will be provided with variable frequency drives to control flow to the plant, back-up power, and room for chemical totes and feed pumps to control zebra mussels or provide pretreatment of the raw water.

An optional immediate improvement would be to replace the existing pump with a higher head pump capable of 1800 gpm to extend the time frame for replacement of the raw water pump station building by three of four years.

4.1.5 Raw Water Transmission Main

The raw water pump station discharges raw water through a 12-inch transmission main to the water treatment plants primary rapid mix basin. The transmission main is approximately 9,100 feet in length. The capacity of the pipe is dependent on the pressure rating and the velocity through the pipe. For design, it is common practice to design transmission mains for maximum velocities between 5 and 7 feet per second (fps). Consistent with Section 4.1.3 Raw Water Pipe, the capacity of this transmission main will be based on a maximum velocity of 7 fps with a nominal diameter of 12 inches. The capacity is therefore 2,467 gpm (3.55 MGD).

The existing pipeline capacity is greater than the projected maximum day demand (MDD) of 2,252 gpm (3.24 MGD) for the planning year 2038. A parallel pipe should be constructed between the raw water pump station and the treatment plant when projected demands reach 3.55 MGD, which is beyond the timeline of this report.

4.1.6 Summary of Proposed Raw Water Supply Improvements

The recommended improvements for the raw water pump station and transmission main are shown in Table 4-3 along with proposed years to initiate the work and complete the work.

Table 4-3 Summary of Raw Water Supply Improvements

Item No.	Description	Initiation Year	Completion Year	Estimated Cost*
1	Zebra Mussel Control	2018	2018	$20,000
2	Raw Water Pump Station Replacement	2018	2019	$1,659,000
3	Valve Boxes at Dam	Based on USACE Schedule		$366,000
4	Negotiate Additional Water Storage in Smithville Lake	2031	2033	$2,000,000
Total Raw Water Supply Improvements				**$4,045,000**

*Note: Estimate Cost is in 2018 $

4.2 Water Treatment Plant

The City's existing WTP consists of rapid or "flush" mixing, primary and secondary contact or sedimentation basins, followed by gravity filtration. After the water is filtered, it is disinfected and pumped to the chlorine contact basins. High service pumps then deliver the finished or treated drinking water to the distribution and storage system. An aerial view of the Water Treatment Plant is shown in Figure 4.6 Existing Smithville WTP Layout.

Surface water treatment plants must comply with the USEPA's Surface Water Treatment Rule requiring facilities to remove or inactivate microorganisms such as giardia, cryptosporidium, and other viruses. MDNR requires surface water plants to have longer detention times for treatment and disinfection. Typically, surface water plants have wider swings in turbidity and lower alkalinity than groundwater from wells. Well water typically has high hardness, high alkalinity, and more iron and manganese. The tradeoffs being that surface water plants generally require larger basins while ground water plants generally require more chemicals.

The existing surface WTP has a maximum design flow rate of 2.5 MGD (1,736 gpm) as permitted by the Missouri Department of Natural Resources (MDNR). Over the last five years, the WTP treated an average of 0.945 MGD. While the design capacity of the plant is 2.5 MGD, staff is only able to treat approximately 2.09 MGD (1,450 gpm) due to pumping limitations of the Raw Water Pump Station (see Section 3.1.3 Raw Water Pump Station).

Future water demands were presented in Table 3-2 - Smithville Water Demand Projections. According to these projections, the maximum day water demands will begin to exceed the water treatment plants design capacity by 2024.

The proposed improvements outlined in the following sub-sections are identified as either Capacity Improvements (required meet future demand projections) or Maintenance Improvements (needed to improve safety or reliability or to satisfy MDNR requirements).

Figure 4.6 Existing Smithville WTP Layout

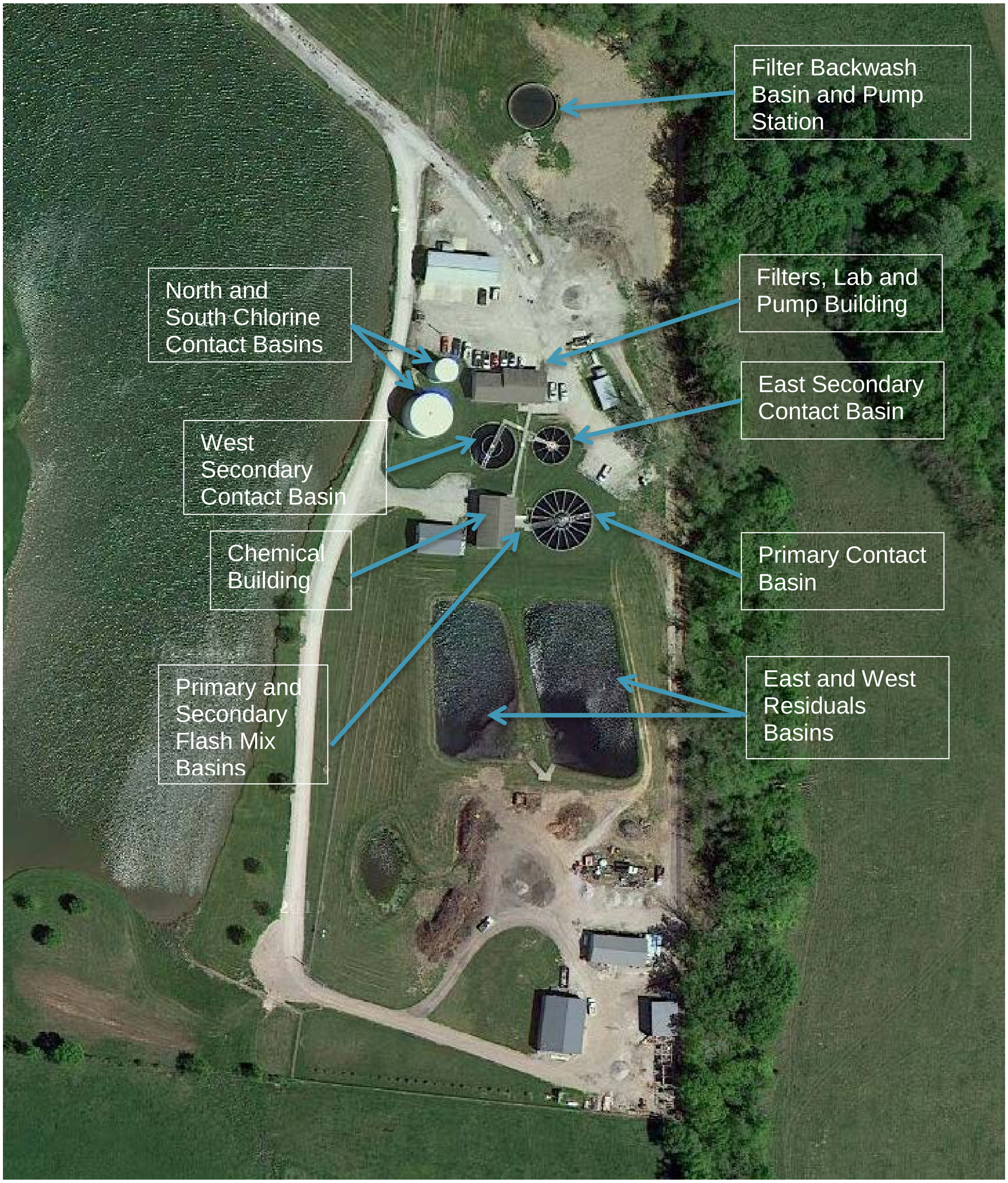


Figure 4.7 Water Treatment Plant Process Flow Diagram illustrates the existing water treatment plant process flow diagram.

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EXISTING WATER TREATMENT PLAN LAYOUT

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Figure 4.6

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EXISTING PROCESS FLOW DIAGRAM

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4.2.1 Primary Rapid Mixing Basin

Raw water is discharged directly into the Primary Rapid Mixing Basin. Chlorine dioxide is injected into the raw water transmission main prior to entering the Mixing Basin. Powder activated carbon, poly-aluminum chloride, and lime are discharged into the top of the Basin. A 25 HP mixer blends the chemicals with the water at 1,200 RPM prior to it entering the Primary Sedimentation Basin.

The Primary Rapid Mix Basin is 3 feet-4 inches wide by 5 feet long by 6 feet deep with an approximate volume of 750 gallons. Figure 4.8 depicts the Primary Rapid Mixing Basin. Table 4-4 presents the estimated Primary Rapid Mix Basin detention times based on the design capacity of the WTP and the existing operating conditions.

Figure 4.8 Primary and Secondary Rapid Mix Basins

Table 4-4 Primary Rapid Mix Basin Detention Time

Operating Condition	Flow Rate (gpm)	Flow Rate (MGD)	Detention Time (sec)
WTP Rated Design	1,737	2.5	25.8
Existing Conditions[1]	1,450	2.01	31
MDNR Design Requirement[2]	-	-	≤ 30

[1] Based on limited pumping capacity of Raw Water Pump Station.

[2]MDNR Design Requirement at maximum design flow rate

The supports for the mixers are rusted and the weirs are also rusting. In general the equipment is beginning to reach its useful life and need replaced. The current Primary

Rapid Mix Basin layout and chemical feed injection locations do not provide adequate detention time for removal of taste and odor causing compounds in the raw water. Powder Activated Carbon does not currently have adequate contact time. It would also benefit from being introduced upstream of the chlorine dioxide to prevent premature reacting. New rapid mixing and chemical addition facilities are needed to improve taste and odor issues. Scheduling these improvements with other Capacity Improvements would be the most economically advantageous implementation plan.

4.2.2 Primary Settling Basin

Following the Primary Rapid Mixing Basin, a 14-inch diameter cast iron pipe conveys water through a 14-inch manual sluice gate to the center of the Primary Settling Basin. Water is directed downward in the basin by a cone-shaped baffle. Water rises from the bottom of the Basin and enters one of the 12 launders located at the surface of the Basin. Each launder has orifices located at or just below the water surface that allow the water to enter the trough. The launders deliver water to a central launder that collects into an 18-inch diameter pipe then an 18-inch manual sluice gate to the Secondary Rapid Mixing Basin. Figure 4.9 depicts the primary settling basin.

Figure 4.9 Primary Settling Basin

MDNR requires that the outlet weirs not be located lower than 3 feet below the water surface and the entrance velocity through submerged orifices shall be sufficient to provide enough head loss that even flow is provided through each orifice, but shall not exceed 0.5 feet per second (fps). MDNR requires a minimum of four hours of settling time and a rise rate of less than 0.75 gpm/ft^2. The existing Primary Settling Basin design meets the general MDNR requirements. The Primary Settling Basin design summary is presented in Table 4-5.

Table 4-5 Primary Settling Design Summary

Primary Settling Basin	
Inside Diameter	65 feet
Average Depth	20.8 feet
Volume	516,000 gallons
Detention Time [1]	5 hours
Rise Rate [2]	0.12 gpm/ft^2
[1] Detention time is greater than MDNR minimum of 4 hours at WTP design capacity 2.5 MGD (1,737 gpm). [2] Rise Rate is less than MDNR maximum of 0.75 gpm/ft^2 at WTP design capacity 2.5 MGD (1,737 gpm).	

In general the Primary Settling Basin can provide 2.5 MGD (1,736 gpm) through the year 2025 when other plant capacity improvements are required. In order to increase the capacity of the plant to meet the future demands beyond 2.5 MGD, additional volume for the primary settling basin will be necessary to meet the detention time required by the MDNR. This can be accomplished by constructing two New Primary Settling Basins south of the Sludge Lagoons on existing City property. Each basin would be sized for 2.5 MGD and be the size of the existing Primary Settling Basin. The existing Primary Settling Basin would be converted to a Secondary Settling Basin and one of the existing secondary settling basins would be replaced with a basin of equal size.

The proposed WTP layout is shown in Figure 4.10. The proposed layout separates the plant into two separate treatment trains, with each train designed to provide 2.5 MGD flow for a total future plant capacity of 5 MGD. With this flexibility, the Operators have the ability to perform maintenance during the off peak months on one train and still maintain average daily flow. The proposed design would allow the treatment plant to treat 5.0 MGD and exceed the 20 year water demands.

These improvements would be considered Capacity Improvements that do not need to be started until the year 2022 and completed in 2024.

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PROPOSED WATER TREATMENT PLAN LAYOUT

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SCALE 1' = 40'

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4.2.3 Secondary Rapid Mixing Basin

The Secondary Rapid Mixing Basin is downstream of the Primary Settling Basin and is located immediately adjacent to, and just north of, the Primary Rapid Mixing Basin. At the Secondary Rapid Mixing Basin, additional chemicals are added. Chlorine dioxide is added for disinfection of viruses and bacteria and oxidation of metals. Poly aluminum chloride is added as a coagulant to further assist removal of particles. The Basin includes a mixer and a weir over which water must flow to enter Secondary Settling Basins. Figure 4.11 depicts the Secondary Rapid Mixing Basin.

Figure 4.11 Secondary Rapid Mixing Basin

Table 4.6 presents the estimated Secondary Rapid Mix Basin detention times based on the design capacity of the WTP and the limited pumping capacity of the Raw Water Pump Station.

Table 4-6 Secondary Rapid Mix Basin Detention Time

Operating Condition	Flow Rate (gpm)	Flow Rate (MGD)	Detention Time (sec)
WTP Rated Design	1,737	2.5	19.3
Existing Conditions[1]	1,450	2.01	23.1
MDNR Design Requirement[2]	-	-	≤ 30

[1] Based on limited pumping capacity of Raw Water Pump Station.

[2] MDNR Design Requirement at maximum design flow rate

The secondary rapid mix basin has a volume of approximately 560 gallons and a detention or mixing time of 23.1 seconds at the current maximum achievable plant flow rate of 1,450 gpm. At 1,750 gpm, or 2.5 MGD, the detention time would be 19.3 seconds. MDNR states that mixing shall not be more than 30 seconds at the maximum design flow rate. The secondary rapid mixing basin meets the MDNR requirements at the design flow as well as the existing maximum flow rate.

Like the Primary Rapid Mixing Basin, the Secondary Rapid Mixing Basin needs a new weir, new slide gates, baffle, and work on the supports for the mixer. The structural supports show rust and they are difficult to work around because of the pipes and proximity to the Primary Rapid Mixing Basin.

4.2.4 Secondary Settling Basins

From the Secondary Rapid Mixing Basin, flow is conveyed to two Secondary Settling Basins. Prior to the 1993 WTP Improvement Project, the east basin was the Primary Settling Basin and the west basin was the Secondary Settling Basin. As part of the 1993 WTP Improvement Project, a new Primary Settling Basin was constructed and the east Primary Basin was converted to a Secondary Basin.

The east secondary basin receives water from the Secondary Rapid Mixing Basin via 140 feet of 12-inch diameter cast iron pipe. The west secondary basin receives water from the secondary rapid mix basin through a 14-inch diameter cast iron pipe that is approximately 80 feet in length. Water enters the center of the each Basin through a baffle cone, flows downward, and then exits the cone flowing back up to either rectangular surface troughs (i.e., East Basin, presented in Figure 4.12) or a submerged collection pipe (i.e., West Basin, presented in Figure 4.13).

Figure 4.12 East Basin with Collection Troughs

Figure 4.13 West Basin with a Submerged Collection Pipe

The Secondary Settling Basin design summary is presented in Table 4-7.

Table 4-7 Secondary Settling Basin Design Summary

	East	West
Inside Diameter	40 feet	50 feet
Average Depth	14 feet	15 feet
Volume	131,600 gal	220,300 gal
[1] Flow Rate	642-gpm	1,094-gpm
[1] Detention Time	3.4 hours[3]	3.4 hours[3]
[2] Detention Time at 2.01 MGD (1,450 gpm)	4.1 hours	4.0 hours

[1] Flow at WTP Design Capacity 2.5 MGD (1,736-gpm) divided between the basins based on volume.

[2] Based on limited pumping capacity of Raw Water Pump Station with flow divided between the basins based on Volume

[3] Detention time is less than the MDNR 4 hour design requirement for Primary Settling Basins but meets best practices design requirements of 2 to 4 hours for Secondary Settling Basins. Water Treatment Plant Design by AWWA/ASCE 3rd Edition 1998, pg 113

Neither the East nor the West Secondary Basins have an automated method to remove settled solids from the bottom of the Basins. Operators take the basins offline once per year, drain them, and manually pump out any residual solids held in the basins.

The addition of sludge removal equipment in the secondary settling basins would be beneficial by automating removal of the settled solids on a regular basis. To provide this improvement, the secondary basins would need to be completely reconstructed. This improvement is not needed immediately due to the fact that the basins produce a relatively small amount of solids. This is therefore considered a Maintenance Improvement. As part of the proposed Capacity Improvements these basins will be demolished. A new Secondary Sedimentation Basin will be constructed in place of the east secondary basin and the existing primary basin.

4.2.5 Filters

The Filter Building houses five gravity sand and anthracite filters. Water from the secondary settling basins enters the filters through a common header on the south wall of the Filter Building. Each filter bay is 7.5 feet in width and 16 feet in length with an approximate surface area of 120-ft^2 each (see Figure 4.14 Gravity Filter). The filter media is approximately 48-inches in depth and the water depth over the media is approximately 5 feet when the filter is in operation.

A single 20-inch wide trough is located in the center of each filter and runs the full length of the filter. Underdrains collect the filtered water, which flows by gravity through a 10-inch diameter pipe to the transfer pump chamber.

Manual and air actuated butterfly valves control the backwash, filter to waste, and filter operations. Filtered water, stored in the steel above grade chlorine contact valves on the west side of the filter building, is used for backwashing the filters. The filter backwash rate is approximately 1,800 gpm for 15 minutes. Plant operators use a control panel, located in the filter gallery, which contains flow control valves and a backwash water flow meter readout, to control the rate of flow through the filters. See Figure 4.15.

The backwash water enters the underdrains and exits the filter through the 20-inch trough and 14-inch pipe located at the north end of the lower level pipe gallery. Filter to waste and backwash water flows by gravity from the building to the filter backwash basin located north and east of the filter building.

The filters are not equipped with air scour or surface scour equipment. City staff uses a fire hose to wash down the walls, troughs, and scour the surface of each filter periodically.

Figure 4.14 Gravity Filter

Figure 4.15 Filter Control Panel

Filter Capacity

The design or firm capacity of the filters is based on one of the five filters being out of service. The treatment plant design capacity is 2.5 MGD (1,736 gpm). Dividing 1,736 gpm by 480 square feet (4 x 120 SF per Filter) results in a filtration rate of 3.6 gallons per minute per square foot (gpm/sf).

The existing filters generally meet the minimum MDNR design requirements as listed below. The filter flow rate of 3.6 gpm/sf is higher than the MDNR maximum of 2.0 gpm/sf but is within the range of 2-4 gpm/sf recommended in the 10 state standards and is at a rate that MDNR has approved in the plants operating permit. Like most other plants with higher rates of flow, the plant meets or exceeds the drinking water standards and is therefore allowed to continue operating at this rate.

The following gravity filter design criteria are based on the MDNR Design Guidelines.

- For redundancy, the plant must be able to meet maximum flow with 1 filter off-line

- Maximum Rate of Filtration for the Gravity Filters is 2 gpm/ft^2 of Surface Area
  - The design rate of filtration for the existing Gravity Filters is 3.62 gpm/ft^2 of Surface Area
    - Historically, this rate has not been an issue for treatment: OK
- Maximum velocity of water entering the filter is 2 ft/sec: OK
- Maximum distance a particle must travel to reach a trough is 3 ft.: OK
- Minimum Depth is 8.5 ft: OK
- Filter Surface Area
  - 7.5 ft x 16 ft = 120 ft^2 each.
  - 120 ft^2 x 4 cells x 3.62 gpm/ft^2 = 1,737 gpm or 2.5 MGD: OK
- Backwash rate minimum = 15 gpm/ft^2 x 120 ft^2 = 1,800 gpm; maximum 20 gpm/ft^2 x 120 ft^2 = 2,400 gpm: OK
- Water required for Backwash, Minimum 15 gpm / ft^2 for 15 minutes; maximum 20 gpm / ft^2 for 15 minutes
  - 1,800 gpm x 15 min = 27,000 gallons minimum
  - 2,400 gpm x 15 min = 36,000 gallons maximum
  - Supplemental backwash is provided from plant storage and distribution system pressure: OK

Filter Improvements

The condition assessment indicated several leaking pipes and valves in the pipe gallery below the filters. The leaking valves and pipe need new gaskets, repairs and in some cases they need to be replaced. The filter controls above the pipe gallery are outdated and controlled manually but appear to have been well maintained. New plants are moving to more automation to provide for remote trouble shooting and shut down to decrease labor costs.

The piping and valve repairs are considered to be maintenance improvements and should be performed as soon as possible to prevent additional damage to the existing equipment or structures.

In order to increase the capacity of the plant and meet future water demands. It is recommended that 4 additional filters be constructed in a new building to meet future 2038 water demand projections. The plant would then have a total of 9 filters, with a firm filtration capacity of 5 MGD.

4.2.6 Transfer Pumps

Filtered water then enters a 5-foot by 20-foot vault on the northwest corner of the filter building. This vault or transfer chamber is 13 feet deep. Three vertical turbine transfer pumps are mounted above the transfer chamber outside the north wall of the filter building. Each of the vertical turbine pumps has the following characteristics:

Figure 4.16 Vertical Turbine Pump Characteristics

Number of Pumps	3
Flow	800 gpm
Total Dynamic Head	35 feet
Motor	10 HP
Firm Capacity	1,600 gpm or 2.3 MGD

The transfer pumps discharge into the steel above grade chlorine contact tanks located west of the filter building through 8-inch and 12-inch diameter piping. The individual pump discharge pipes are 8-inch. A common 12-inch header exits the pipe gallery.

A 12% solution of sodium hypochlorite is diffused into the filtered water inside the transfer chamber. This provides a final dose of chlorine disinfectant to kill viruses and other pathogens that may not have been removed in the prior treatment processes and establishes the chlorine residual for the distribution system.

According to MDNR standards the transfer pumps have a firm capacity of 1,600 gpm (2.3 MGD) with one pump out of service. With three transfer pumps operating the pumps could transfer approximately 2,400 gpm (3.4 MGD). Actual firm transfer pump capacity will not meet either the current plant capacity of 2.5 MGD or the future maximum day water demand of 3.24 MGD in the year 2038. Larger transfer pumps are therefore needed. The cost for improving the Transfer Pumps is included in the overall Capacity Improvements.

4.2.7 Chlorine Contact /Finished Water Storage Tanks

Filtered water from the transfer chamber is pumped to the Chlorine Contact (or finished water storage) tanks located west of the filter building. Both tanks are constructed of steel with welded joints and painted on the inside and outside. A 12-inch transfer pipe delivers water to the south tank. The inlet pipe in the south tank terminates 12-inches above the floor, while the outlet pipe stands 20 feet tall in the tank. Water leaving the south tank then enters the north tank which has a 20-foot tall inlet pipe. The outlet pipe of the north tank is 6-inches above the floor. Water flows by gravity from the north tank through a 12-inch diameter pipe to the high service pumps, located in the basement of the filter building.

The two tanks provide contact time for the disinfectant (sodium hypochlorite) to kill viruses and pathogens. The capacities and dimensions of the two tanks are provided in Table 4-8 Capacities and Dimensions of the Chlorine Contacts Basins below:

Table 4-8 Capacities and Dimensions of the Chlorine Contacts Basins

Chlorine Contact Basin	Diameter (ft)	Height (ft)	Volume (full) (gal)	Minimum Operational Height (ft)	Chlorine Contact Volume (min) (gal)
South	46	32	397,790	20	248,494
North	25	32	117,495	16	58,718

Chlorine contact time is based on the minimum operational volume of the Chlorine Contact Basins and includes pipe volume upstream of the first customer. The outlet pipe of the south tank is 20 feet above the floor and controls the minimum volume at 248,494 gallons. According to information provided by staff, the high service pump controls are set to shut the pumps off if the north tank water level falls below 16 feet. The minimum volume for the north tank is therefore 58,718 gallons. Accordingly, the minimum chlorine contact volume of the south and north tanks combined, is 307,212 gallons.

Per EPA and MDNR guidelines, the Smithville Water Treatment Plant must achieve 3.0 log removal of Giardia Lamblia cysts and 4.0 log removal of viruses. Conventional treatment and filtration process are credited with 2.5 log removal of Giardia Lamblia cysts and 2.0 log removal of viruses per EPA and MDNR guidelines. The water treatment plant provides conventional treatment and filtration and is granted the reduction in removal rates. Therefore, the disinfection process must be sufficient enough to provide 0.5 log removal of Giardia Lamblia and 2.0 log removal of viruses.

A tracer study was not performed to determine actual contact time of the Chlorine Contact Basins. Calculations were performed using the MDNR and EPA “rule of thumb” baffling condition - and tables located in the “Missouri Guidance Manual for Surface Water Systems Treatment Requirements”, 1992.

The Chlorine Contact Basins do not have intra-basin baffles, but do have multiple inlets and outlets. This description is consistent with the “Poor” baffling condition from the MDNR baffling conditions table. This condition provides a T_{10}/T ratio of 0.3.

To calculate the CT value, the minimum storage is used as determined above, a finished water pH of 8.5 and a minimum finished water temperature of 5° C. The minimum storage is 307,212 gallons for both storage tanks. The flow rate used to determine the CT value will be the rated treatment capacity through the plant, which is 2.5 MGD or 1,736 gallons per minute. Plant personnel indicate the minimum free chlorine concentration leaving the plant is 1.8 mg/l. The CT value is calculated as follows:

- 1.8 mg/l x ((0.3 T_{10}/T x 307,212 gallons) / 1,736 gallons per minute)
- This provides a CT value of 95.56 mg/l-min.
- Referencing the MDNR CT tables (table 10 and 15) provided in Appendix B, the following are the minimum required CT needed to meet the remaining log removals.
  - CT required for 0.5 log inactivation of Giardia is 48 mg/L-min

- CT required for 2.0 log inactivation of viruses is 4 mg/L-min
- This is less than the CT provided

Therefore, based on the MDNR Design Guide for Chlorine Contact Time, the existing Chlorine Contact Basins exceed the minimum MDNR and EPA requirements for the current plant capacity of 2.5 MGD as well as for the 2038 max demand of 3.24 MGD.

Improvements needed for the ground storage tanks would be to increase the pipe diameter leading to and from the tanks. Based on the calculations additional volume would be required when water demands approach 5.0 MGD, however, a new tank is not included in the Capacity Improvements or in the 20-year planning period.

4.2.8 High Service Pumps

Finished water from the ground storage tanks is pumped to the City of Smithville customers using three high service pumps, located on the lower level of the filter/lab building. The characteristics of the pumps are listed in Table 4-9 High Service Pump Characteristics below. The pumps are shown in Figure 4.17 High Service Pumps.

Table 4-9 High Service Pump Characteristics

Number of Pumps	**3**
Flow	900 gpm
Total Dynamic Head	240 feet
Motor	75 HP
Firm Capacity	1,800 gpm or 2.59 MGD

The firm capacity of the high service pumps is based on two pumps operating simultaneously with one pump out of service. Two high service pumps can provide 1,800 gallons per minute or 2.59 MGD, which exceeds the rated design capacity of the water treatment plant of 2.5 MGD.

The three high service pumps will not be able to meet future demands beyond 2.59 MGD. New higher capacity pumps will be included in the proposed Capacity Improvements to provide for a future plant capacity of 5 MGD (3,472 gpm). The new high service pumps will be located in the proposed Filter Building expansion.

Figure 4.17 High Service Pumps

4.2.9 Chemical Feed Systems

All chemical storage and feed equipment is located in the Chemical Feed Building, which is located immediately west of the Primary Settling Basin. A loading bay on the west side allows for delivery of the chemicals. The existing chemical feed building is in need of various improvements in order to comply with current MDNR requirements. Each bulk liquid storage tank located within the building requires secondary containment to prevent spills and cross contamination.

MDNR also requires that chemical feed operations have redundancy in the event one piece of equipment breaks to ensure that the plant can still treat and provide fully treated water to the general public. Chemical metering pumps need to have back up pumps readily available to switch out upon failure.

The chemical feed building needs various other general improvements, such as removal of abandoned chemical feed lines and conduit, removal of abandoned electrical or control panels. Lighting improvements are needed to facilitate plant operations and improve safety conditions.

The following chemical systems are currently in operation.

- Chlorine Dioxide
- Powder Activated Carbon
- Poly-aluminum Chloride
- Lime
- Sodium Hypochlorite

Chlorine Dioxide

Chlorine Dioxide is generated onsite. This process involves the reaction of sodium hypochlorite and chlorine gas. The result is chlorine dioxide with an almost neutral pH. After generation, the chlorine dioxide is stored in a 750-gallon bulk tank. Chlorine dioxide is fed to the treatment process, prior to each settling basin, for disinfection of viruses and bacteria as well as oxidation of metals.

Samples are collected from the primary and secondary basins to monitor the chlorine residual and dosing is adjusted to meet the desired residual, which is 0.5 mg/l. The chlorine gas and chemical reaction area is separated in a controlled, ventilated area, in compliance with MDNR rules. Chlorine dioxide is stored in an adjacent chemical room with other bulk chemicals, however, the bulk tank does not have secondary containment.

Powder Activated Carbon

Powder activated carbon (PAC) is fed prior to the primary settling basin. PAC is used to react with organics in the raw water in order to reduce the taste and odor in the finished water. The PAC is mixed with water in a volumetric feeder to create a carbon slurry. This slurry is fed to the primary mixing basin at an approximate dose of 18 mg/l. The primary settling basin provides 5 hours of contact time, meeting the MDNR's requirement of 20 minutes with slow mixing.

PAC is stored as a solid powder, in 40-pound bags, in a separate enclosed room. Each bag is manually carried up three stairs where it is then broken open and dumped into a mixing chamber to create a solution. A scissor lift or lifting table should be provided to assist the operators in loading the bags into the chemical feeders to eliminate the requirement to carry bags up stairs.

Poly-Aluminum Chloride

Poly-aluminum chloride (Poly) is fed prior to the primary and secondary settling basins. Poly is used as a coagulant because of its high charge, which makes it very effective at destabilizing and removing suspended materials that are within the water.

The Poly is delivered to the plant as a liquid and stored in bulk tanks within the chemical building, however, the bulk tank does not have secondary containment. Chemical feed pumps deliver the Poly at an approximate dose of 19 mg/l to the primary basin and 3 mg/l to the secondary basins for a combined dose of approximately 22 mg/l. After it is mixed with the water, its charge attracts the particles in the water until the particles are large enough to settle out in the settling basins.

Lime

A lime solution is fed in the primary rapid mixing basin. Lime is being used in small doses, 2 to 5 mg/l, to slightly increase the pH of the water to approximately 8.1. At these doses and an average flow of 1.1 MGD, they are feeding an average of 18 to 45 lbs/day of lime. Lime is not used to soften the water. With varying temperatures and raw water turbidities, chemical reactions and filtration can be affected by the feed water pH. By adjusting the dose of lime feed, the operator can optimize pH and increase treatment efficiency.

Lime is stored as a solid powder inside the chemical building in 40-pound bags. The bags are carried up seven stairs where it is broken open and dumped into a mixing chamber or slaker to create a solution that is transported by gravity to the feed location. Figure 4.18 shows a picture of the 40 pound bags of lime and the stairs to the lime slacker.

The existing lime slaker has outlived its useful life. The operators use the old alum slaker for parts when repairs are needed. Parts for this slaker are no longer available. A new slaker or lime feed system is necessary. MDNR generally requires redundant lime slaking units. The immediate need is for the installation of one new unit. This improvement assumes replacement in kind. The estimated cost of a new lime slaker is $100,000. This is categorized as a maintenance improvement. MDNR requirements for a redundant chemical feed unit will be accommodated in the Capacity Improvements.

Figure 4.18 Lime and Lime Slaker Equipment

Sodium Hypochlorite

Sodium Hypochlorite (Hypo) is dosed in the filter building transfer chamber upstream of the chlorine contact basins. Hypo is used for disinfection of viruses and bacteria throughout the distribution system. Hypo is delivered to the plant as a liquid at a 12.5% concentration and stored in 750-gallon bulk tanks within the chemical building. The storage tanks are located in the vicinity of other chemicals and the building is well ventilated. Chemical feed pumps deliver the Hypo at an approximate dose of 6 mg/l.

The plant monitors the chlorine residual of the water leaving the plant. The hypo dosage is set to maintain a desired range of 1.8 – 2.0 mg/l of free chlorine.

Samples are collected at the ends of the distribution system to ensure that there is still sufficient chlorine residual present. If the samples indicate a residual does not meet the MDNR minimum total chlorine residual requirement of 0.2 mg/L, the hypo dosage is adjusted accordingly.

The existing hypo bulk storage tanks do not have adequate secondary containment. Acceptable containment is required and separation between hypo and other chemicals is needed for employee safety.

4.2.10 Plant Waste Streams

The plant has three major waste streams; primary settling basin sludge waste, secondary settling basins waste, and filter backwash waste. All waste streams are pumped to the sludge holding lagoons located south of the plant. Decant water from the holding lagoons is de-chlorinated and then discharged by gravity to the nearby creek. Once the lagoons are full, a contractor is hired to dredge and properly dispose of the residual solids.

Primary Settling Basin Sludge Waste

Sludge from the Primary Settling Basin is collected in a trough at the bottom center of the circular basin, transferred by gravity to the primary sludge pump vault. Primary sludge pumps discharge through a 4-inch PVC pipe approximately 250 feet to the holding lagoon splitter box, which then discharges into one of the two holding basins. Primary sludge is dark in color due to the powdered activated carbon added in the primary rapid mix basin. A timer is used to energize the sludge valve to open and allow the sludge to be automatically removed from the primary basin. The timer is currently set to open for 8 minutes every three hours.

The primary sludge pump vault is located next to the Primary Rapid Mix Basin. The Gorman-Rupp T-Series sludge pump is operated by a 7.5 horsepower motor and can pump 440 gpm.

Secondary Settling Basin Sludge Waste

The secondary settling basins do not have sludge pumps or piping to transfer settled sludge waste to the holding lagoons. The secondary sludge collected inside these basins is removed on an annual basis by taking the basins off-line, lowering a portable submersible pump into the basin, and pumping the contents through portable piping to the holding lagoons. The basins are pressure washed during this process.

Filter Backwash Waste

Filter backwash waste is drained to the Filter Backwash Basin located to the north of the filter building. The backwash basin contents are then pumped to the Residuals Holding Basin Splitter Box.

Backwash Equalization Basin / Pump Station

During filter backwash cycle, which are manually initiated by the operators, all backwash water is discharged by gravity to the Filter Backwash Basin. To meet the MDNR requirements, each filter is backwashed with a minimum of 27,000 gallons of water. Plant Operators use a hose to scour the surface of each filter during the process. Each of the five filters is generally backwashed once a week or more often if required.

The filter backwash basin has an associated pump station to transfer the equalized filter backwash waste to the holding lagoons. The pump station contains two Flygt pumps with 20 horsepower motors each rated at 815 gpm and 61 feet TDH.

4.2.11 Sludge Holding Lagoons

The sludge holding lagoons are located to the south of the primary settling basin at an elevation above the treatment processes. Staff alternates flow to either basin using slide gates inside the concrete splitter box located between the basins. Each lagoon is approximately 75 feet by 175 feet with a depth of 5 feet plus two feet of free board. The depth of the residual is monitored to determine when the sludge needs to be removed. Once it is determined that the lagoons are at 75% of their capacity, the plant schedules the sludge to be removed and properly disposed.

Sludge was last removed from the lagoons in 2014. The prior removal project was completed in 1993 when the lagoons were modified during that plant expansion. Increasing water demands will require the basin sludge to be emptied more regularly. Sludge depth is monitored by staff when they operate the valve to decant the water in the lagoon. The cost to dewater and remove the residual depends on the amount of solids that are removed from the lagoons. The cost for removal of 642 dry tons in 2014 was $430,000. This work may be required again in 6 years. Costs are dependent on hauling cost to a local farm and competition within the industry. The city should plan to spend approximately $594,000 for this work in the year 2025 or 2026 depending on the residual levels.

Decant and Treatment

The decant water from the holding lagoons is discharged to the nearby creek to the east of the plant under MDNR NPDES number MOG640123. After residuals in the holding lagoons settle, a telescoping valve is opened to discharge the decant water. The NPDES permit includes chlorine and total suspended solids (TSS) limits which must be monitored and test results submitted to MDNR quarterly.

From the telescoping valves, the decant travels through an 8-inch PVC gravity discharge pipe. Sodium Thiosulfate tablets are used to de-chlorinate the decant flow prior to discharge at the creek. A sample station is located at the outfall downstream of the de-chlorination tablets so samples can be collected to ensure proper de-chlorination is being performed. Prior to the discharge of the decant water, a sample will be collected to determine the TSS of the decant water. The plant must ensure that the TSS results and chlorine levels meet or exceed the NPDES permit limits before decant water can discharge to the creek.

4.3 WTP Summary and Proposed Improvements

The Water Treatment Plant is permitted by MDNR to produce 2.5 MGD (1,736-gpm) of drinking water. Calculations indicate the plant can produce 2.5 MGD but plant production is currently limited by the Raw Water Pumping System (2.09 MGD or 1,450 gpm).

Figure 3.2 Water Demand Projection Graph vs. Permitted Capacity illustrates Maximum Day Demand projections in relation to the existing plant production and design capacity of 2.5 MGD or 1,739 gpm. The projections predict that demand will exceed capacity in the year 2025. Capacity Improvements will be required to meet the future demands. Figure 3.10 shows the proposed Capacity Improvements for the Water Treatment Plant

4.3.1 Summary of Proposed Water Treatment Plant Improvements

Table 4-10 lists the proposed Maintenance Improvements for the Water Treatment Plant, estimated capital costs, and time frame they should be implemented.

Table 4-10 WTP Maintenance Improvements

No.	Description	Year Initiated	Year Complete	Estimated Cost*
1	Replace Lime Feeder	2018	2018	$100,000
2	General Plant Improvements	2019	2019	$101,000
3	Chemical Feed Building Improvements	2020	2021	$235,000
4	Residual Removal	2025	2026	$594,000
Total Water Treatment Plant Maintenance Improvements				**$1,030,000**

*Note: Estimate Cost is in 2018 $

Table 4-11lists the proposed Capacity Improvements illustrated in Figure 4.10 Proposed Water Treatment Plant Layout, which are needed to meet future water demands.

Table 4-11 WTP Capacity Improvements

No.	Description	Year Initiated	Year Complete	Estimated Cost*
1	New Primary and Secondary Sedimentation Basins	2022	2025	$4,619,000
2	New Filter and High Service Building	2022	2025	$4,244,000
Total Water Treatment Plant Capacity Improvements				**$8,863,000**

*Note: Estimate Cost is in 2018 $

5 Distribution System

5.1 Existing Facilities

The existing distribution system has three pressure zones; the central pressure zone, the north pressure zone and the south pressure zone. The water treatment plant is located on the east side of the downtown area and has three high service pumps that pump directly into the central zone. The central zone has two elevated storage tanks, called the north elevated tower and the south elevated tower, which provide storage for the zone. Near each of these elevated tanks is the north booster station and south booster station that supply water to the north pressure zone and south pressure zone, respectively. The north pressure zone and south pressure zone each have a single elevated tank referred to as the northwest water tower and southwest water tower, respectively. Pipe diameters in the distribution system range from 2-inch to 12-inch.

A Water Distribution System Map in Appendix C of this document show the locations of the three pressure zones, storage tanks, booster pump stations, water treatment plant and distribution system piping.

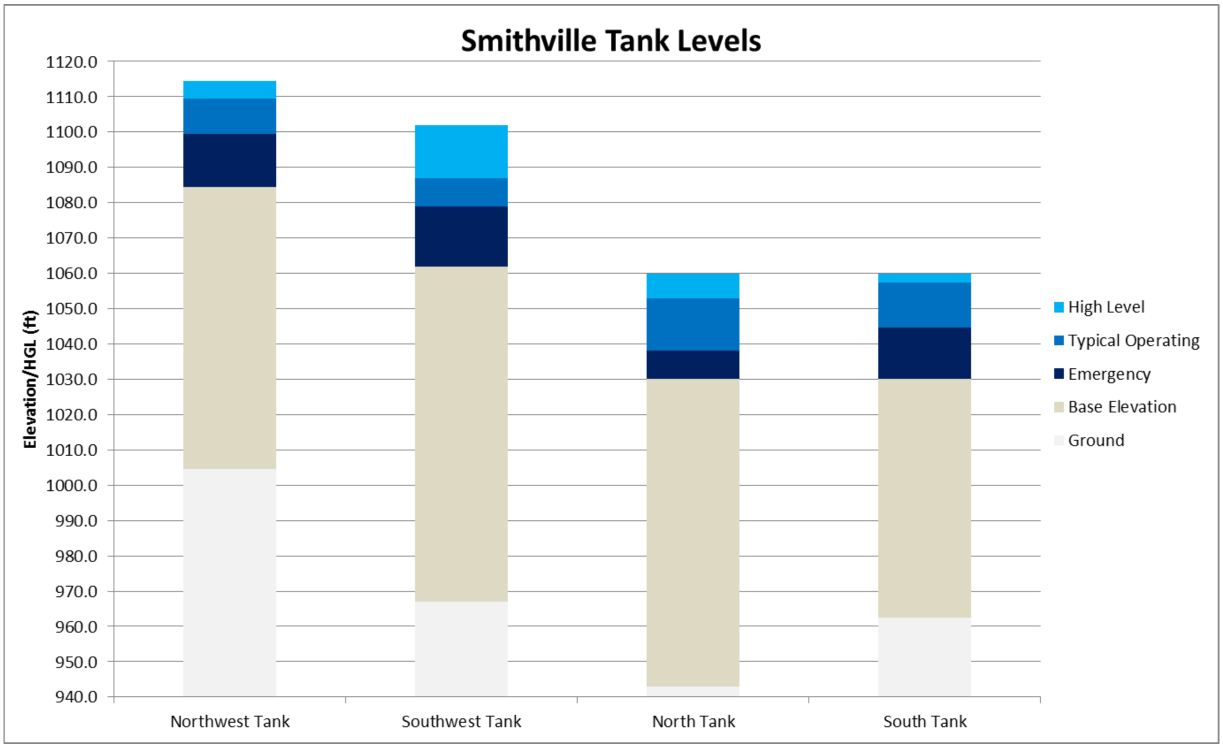
5.1.1 Distribution Storage Tanks

Table 5-1 Elevated Storage Tank Elevation Data lists the operating heights and elevations of the elevated storage tanks. Figure 5.1 Elevated Storage Tank Hydraulic Profiles graphically presents the information listed in Table 5-1.

Table 5-1 Elevated Storage Tank Elevation Data

			Heights from Base				Hydraulic Grade Line Ranges (Elevations)			
Tank Name	Nominal Size (gallon)	Base Elevation	Min	Operating Low	Operating High	Overflow	Min	Operating Low	Operating High	Overflow
Northwest Tank	500,000	1004.5	80.0	95.0	105.0	110.0	1084.5	1099.5	1109.5	1114.5
Southwest Tank	500,000	967.0	95.0	112.0	120.0	135.0	1062.0	1079.0	1087.0	1102.0
North Tank	500,000	943.0	87.0	95.0	110.0	117.0	1030.0	1038.0	1053.0	1060.0
South Tank	750,000	962.5	67.5	82.0	95.0	97.5	1030.0	1044.5	1057.5	1060.0

Figure 5.1 Elevated Storage Tank Hydraulic Profiles



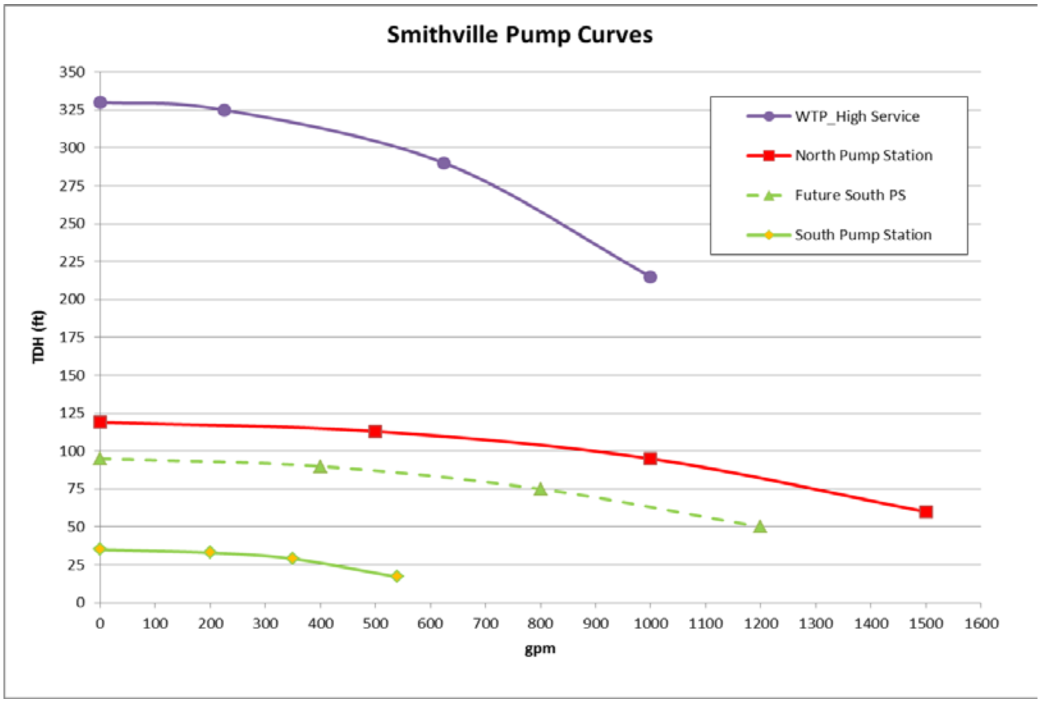
5.1.2 Pumping Stations

Table 5-2 Distribution System Pump Information lists the distribution system pumps, capacities and characteristics. Figure 5.2 shows the pump curves for the three pumping stations and the Water Treatment Plant.

Table 5-2 Distribution System Pump Information

Pump Name	Manufacturer	Pumping Head (ft)	Pump Flow (gpm)
Water Treatment Plant High Service Pumping			
WTP_P1	Aurora	240	900
WTP_P2	Aurora	240	900
WTP_P3	Aurora	240	900
		Firm Capacity	1800
North Pump Station			
NPS_P1	Aurora	95	1000
NPS_P2	Aurora	95	1000
		Firm Capacity	1000
South Pump Station			
SPS_P1	Aurora	29	350
SPS_P2	Aurora	29	350
		Firm Capacity	350
Note: Pump head and flow is at highest pump efficiency.			

Figure 5.2 - Distribution System Pump Curves



Hydraulic Model Analysis

A hydraulic water model was created using Bentley WaterCAD to evaluate the pumps, elevated water storage facilities, and distribution pipes ability to meet current and future water demands. A Maximum Day Demand (Max Day) and Peak Hour steady-state scenario was set up for current (2018), 2026 and 2036 planning years. An Average Day and Maximum Hour extended period simulation was also set up for 2016, 2026 and 2036 planning years. A discussion of the hydraulic model results are described by pressure zone as follows:

5.1.3 Central Pressure System

All water supplied to the central zone system comes from the water treatment plant high service pumps. The pumps have adequate head and firm capacity for current (2016) and 2026 maximum day demands. Firm pumping capacity cannot meet the projected 2036 demands so a future review of pumping capacity will need to be conducted at that time to determine if pump upsizing is required or if the City will pump above firm capacity by having all three pumps operational during peak demand periods.

The existing north and south towers provide adequate equalization storage for all planning periods. However, the distribution system currently has restricted capacity to supply water to the north tower. The south tower is closer to the water treatment plant and has 12-inch water mains along the majority of its supply feed while the north tower is further away and has 8-inch mains along the majority of its supply pipeline. The location of the two towers and the pipe diameters of the supply pipes have caused an imbalance in the system, which results in higher system pressures and throttling of valves at the south tower to prevent overtopping in order to supply water to the North

Tower. In order to reduce required system pressures and allow for the north and south towers to track closer together, it is recommended that the City install a 12-inch continuous supply line from the water treatment plant to the north tower and north pump station. These improvements are identified as C1, C2, C3 and C4 and C6 on the Water Distribution System Map in Appendix C, Water Distribution System Map.

The existing distribution center in the core urban area north and south of the Little Platte River has a mix of newer 8-inch mains and older undersized mains (2-inch to 6-inch). As the City replaces the older mains in the future due to break history or development needs, the locations and lengths of replacement mains should be analyzed to look for opportunities to eliminate bottlenecks, provide system looping, and increase system redundancy by adding or increasing water main diameters.

5.1.4 North Pressure System

The model analysis indicated the existing north pump station has adequate pumping head and capacity for all planning periods. The northwest tower also has adequate equalization storage for all planning periods. The two recommended north zone distribution system improvements are to upsize the existing 6-inch north pump station supply and discharge main to 8-inch and to complete the missing section of 8" main on 188th Street between Primrose Street and Wildflower Street. All new developments should have a minimum main size of 8 inches and provide system looping as much as possible.

5.1.5 South Pressure System

The model analysis indicated the existing south pump station does not have adequate pumping head or capacity for current or future needs. The existing pump station is a pre-manufactured underground facility. Further analysis and design will be required to determine if it is possible to upsize the pumps within the existing pump station or if a new pump station is required.

The southwest tower has adequate equalization storage for all planning periods. The recommended distribution system improvement includes completing a continuous 8-inch main along the west side of Hwy 169 that is connected to the existing main on the east side of the highway. These improvements are identified as C9, S1, S2, S3 and S4 on the fold out map. All new developments should have a minimum main size of 8 inches and provide system looping as much as possible.

5.2 Distribution System Storage

The definition of water "Storage" for a distribution system includes equalization storage and emergency storage. Equalization storage is the amount of storage required to meet diurnal variations while maintaining minimum normal operating pressures. Emergency storage is storage beyond equalization storage that can be used for emergencies such as fire, power failures and supply outages while maintaining system pressures above 20 psi. Since all of the elevated towers can maintain a 20 psi minimum throughout their storage range, any storage not taken by equalization can be considered available for emergency storage. Available storage for each pressure zone is listed in Table 5-3 Distribution System Storage Summary (MGD) as follows:

Table 5-3 Distribution System Storage Summary (MGD)

Tank Name	Nominal Size	Equalization	Emergency
North Tower	500,000		
South Tower	500,000		
Central Zone Total	**1,000,000**	**150,000**	**850,000**
Northwest Tower	500,000		
North Zone Total	**500,000**	**150,000**	**350,000**
Southwest Tower	750,000		
South Zone Total	**750,000**	**150,000**	**600,000**

Based on available emergency storage, the north, south and central pressure zones have enough storage to supply a 3-hour fire with 2000, 3000 and 3500 gpm flows respectively, assuming no pumps are supplying water into the zones. Operating pumps supplying the pressures would increase the fire flow available in those zones. In addition to storage and pumping, the distribution mains can also limit the fire flow available at any particular location within the distribution system. Fire flow analysis of the distribution system is discussed further in the next section.

5.3 Fire Flow Analysis

A fire flow analysis hydraulic model was conducted for the current conditions (2018) and 2036 conditions assuming recommended improvements are completed. The analysis used the maximum day steady state scenario. Available fire flow in the north and south pressure zones assumes a single pump supplying water into the zone remains operating. The central zone fire flow assumes two pumps are supplying water from the water treatment plant.

Fire flow availability varies across the distribution system but is generally in a range between 500 to 3500 gpm depending on main size, system looping and proximity to water tower or pump station. Most residential areas have values above 1000 gpm where mains are looped and 8-inch or larger diameter. Most commercial areas values ranged between 1000 to 3500 gpm.

In locations where fire flow for existing or proposed developments are determined to be inadequate, then an analysis to determine proper size of any required main extensions or upsizing should be conducted on a case by case basis.

5.4 Summary of Proposed Distribution System Improvements

The following improvements are shown on the Water Distribution System Map in Appendix C.

Estimated project costs were developed using $15 per inch diameter multiplied by the length of pipe. The estimated costs were intended to include construction and general Engineering, assuming the City of Smithville would provide resident project representatives. Easements and special construction are not included in the estimated cost. Engineering and contingency is estimated to be 27% of the total price, but each project will need to be evaluated in more depth at the time services are recommended.

5.4.1 WTP to North Tower Transmission Main

A 12-inch water main is needed from the WTP to the North Tower to allow filling of the North tower without overtopping the South Tower. The project can be completed in phases with the river crossing as the first priority.

Table 5-4 WTP to North Tower Water Main Improvements

Map ID	Description	Year	Pipe Dia.	Length	Estimated Cost*
C1	River Cross – Main St to 3rd St	2020	12”	2,560	$461,000
C2	Maple Ln, Highway F to Maple Ln	2021	12”	1,180	$212,400
C3	Helvely Park Dr., WTP to Liberty Rd	2026	12”	3,280	$590,400
C4	Hwy F, East pope Ln to North Tower	2030	12”	3,650	$657,000
N1	188th St, Primrose to Wildflower St	2031	8”	700	$84,000
Total Water Treatment Plant to North Tower Water Main Improvements					**$1,263,800**

*Note: Estimate Cost is in 2018 $

5.4.2 Downtown Water Main Improvements

Improvements to water mains are proposed along Bridge Street and Main Street. Main Street water mains should be upsized to 12-inch in locations that overlap.

Table 5-5 Downtown Water Main Improvements

Map ID	Description	Year	Pipe Dia	Length	Estimated Cost*
C5	Main St, Bridge St to River Crossing	2027	8”	1,180	$141,600
C6	Main St, River Crossing to Liberty Rd	2025	12”	625	$112,500
Total Downtown Water Main Improvements					**$254,100**

*Note: Estimate Cost is in 2018 $

5.4.3 South Booster Pump Station and Transmission Main Improvements

A new booster pump station with a 12-inch main along US Hwy 169 is proposed in order to adequately supply the Southwest Tower and development in the south pressure zone. The 12-inch water main improvements are located in both the central and south pressure zones to provide adequate supply and discharge capacity to the new booster pump station.

Table 5-6 South Booster Station and Water Main Improvements

Map ID	Description	Year	Pipe Dia.	Length	Estimated Cost*
C8	Hwy 92, Commercial Ave to 169 Hwy	2022	8"	1,230	$147,600
C9	169 Hwy, Highway 92 to Park Dr	2023	12"	1,500	$270,000
S1	Tower Interconnect Armory Rd, 69 Hwy	2022	12"	85	$15,300
S2	Interconnect Mains at 144th St and 169 Hwy	2022	12"	100	$18,000
S3	169 Hwy, Commercial to 144th St	2028	12"	2,725	$490,500
S4	169 Hwy, 144th St to Southwest Tower	2029	12"	2,590	$466,200
S5	New South Booster Pump Station	2024	NA	NA	$1,500,000
Total South Booster Pump Station and Water Main Improvements					**$2,907,600**

*Note: Estimate Cost is in 2018 $

The years listed in Tables 5.4, 5.5 and 5.6 indicate a possible ranking of the priority or importance of the projects. Delivering water from the Water Treatment Plant to the North Water Tower and improvements around the South Booster Pump Station are considered the high priorities.

The total estimated cost of the Distribution System Improvements identified in the hydraulic model and through discussions with water department staff during the 20 year planning period is $4,425,500.

6 Summary of Proposed Improvements and Cost Estimates

The tables below represent the estimated construction costs for the recommended improvements discussed in Sections 2, 3 and 4 and a timeline, which represents the prioritization of the proposed improvements.

6.1 Raw Water Supply

Table 6.1 lists the proposed improvements and costs for the Raw Water Supply.

Table 6-1 Raw Water Supply Improvements

Item No.	Description	Initiation Year	Completion Year	Estimated Cost*
1	Zebra Mussel Control	2018	2018	$20,000
2	Raw Water Pump Station Replacement	2018	2019	$1,659,000
3	Valve Box at Dam	Based on USACE Schedule		$366,000
4	Negotiate Additional Water Storage in Smithville Lake	2031	2033	$2,000,000
Total Raw Water Supply Improvements				**$4,045,000**

*Note: Estimate Cost is in 2018 $

The total estimated cost, in 2018 dollars, for the improvements to the Raw Water Supply is estimated to at $4,045,000. Replacement of the Raw Water Pump Station is the highest priority.

6.2 Water Treatment Plant

Table 6-2 & Table 6-3 list the proposed improvements for the water treatment plant

Table 6-2 Water Treatment Plant Maintenance Improvements

No.	Description	Year Initiated	Year Complete	Estimated Cost*
1	Replace Lime Feeder	2018	2018	$100,000
2	General Plant Improvements	2018	2019	$101,000
3	Chemical Feed Building Improvements	2020	2021	$235,000
4	Residual Removal	2025	2026	$594,000
Total Water Treatment Plant Maintenance Improvements				**$1,030,000**

*Note: Estimate Cost is in 2018 $

Table 6-3 Water Treatment Plant Capacity Improvements

No.	Description	Year Initiated	Year Complete	Estimated Cost*
1	New Primary and Secondary Sedimentation Basins	2022	2025	$5,450,000
2	New Filter and High Service Building	2022	2025	$4,200,800
Total Water Treatment Plant Capacity Improvements				**$9,650,000**

*Note: Estimate Cost is in 2018 $

Total estimated cost (in 2018 dollars) for WTP Maintenance Improvements is $1,030,000. The total estimated cost for WTP Capacity Improvements is $9,650,000 for a future plant capacity of 5 MGD (3,472 gpm).

The estimated cost to construct a new 3.5 MGD treatment plant on the existing City of Smithville property south of the sludge lagoons is $14,000,000 to $17,500,000.

Replacement of the Lime Feed system is the highest priority followed by capacity improvements at the Water Plant.

6.3 Distribution

Table 6-4 & Table 6-5 & Table 6-6 list the proposed costs for the Distribution System Improvements

Table 6-4 WTP to North Tower Water Main Improvements

Map ID	Description	Year	Pipe Dia.	Length	Estimated Cost*
C1	River Crossing – Main St to 3rd St	2020	12”	2,560	$461,000
C2	Maple Ln, Highway F to Maple Ln	2021	12”	1,180	$212,400
C3	Helvely Park Dr, WTP to Liberty Rd	2026	12”	3,280	$590,400
C4	Hwy F, East pope Ln to North Tower	2030	12”	3,650	$657,000
N1	188th St, Primrose to Wildflower St	2031	8”	700	$84,000
Total Water Treatment Plant to North Tower					**$1,263,800**

*Note: Estimate Cost is in 2018 $

Table 6-5 Downtown Water Main Improvements

Map ID	Description	Year	Pipe Dia.	Length	Estimated Cost*
C5	Main St, Bridge St to River Crossing	2027	8”	1,180	$141,600
C6	Main St, River Crossing to Liberty Rd	2025	12”	625	$112,500
Total Downtown Water Main Improvements					**$254,100**

*Note: Estimate Cost is in 2018 $

Table 6-6 South Booster Station and Water Main Improvements

Map ID	Description	Year	Pipe Dia.	Length	Estimated Cost*
C8	Hwy 92, Commercial Ave to 169 Hwy	2022	8”	1,230	$147,600
C9	169 Hwy, Highway 92 to Park Dr	2023	12”	1,500	$270,000
S1	Tower Interconnect Armory Rd, 69 Hwy	2022	12”	85	$15,300
S2	Interconnect Mains at 144th St and 169 Hwy	2022	12”	100	$18,000
S3	169 Hwy, Commercial to 144th St	2028	12”	2,725	$490,500
S4	169 Hwy, 144th St to Southwest Tower	2029	12”	2,590	$466,200
S5	New South Booster Pump Station	2024	NA	NA	$1,500,000
Total South Booster Pump Station and Water Main Improvements					**$2,907,600**

*Note: Estimate Cost is in 2018 $

Total estimated cost in 2018 dollars for all recommended Distribution improvements is estimated to be $4,425,500.

The highest priority is Item C1 the River Crossing between Main Street and 3rd Street followed by C2 Improvements on Maple Lane and then improvements around the South Booster Pump Station S1, S2, and C8.

Table 6.7 summarizes the recommended improvements for the Water Supply System

Table 6-7 Recommended Improvements and Costs

No.	Description	Estimated Cost*
1	Raw Water Supply	$4,045,000
2	Water Treatment Plant Maintenance Improvements	$1,030,000
3	Water Treatment Plant Capacity Improvements	$9,650,000
4	Distribution Improvements	$4,425,500
	Total Recommended Improvements 2018 to 2036	$**18,363,500**

*Note: Estimate Cost is in 2018 $

6.4 Financing

HDR is willing to assist the City with applying for State Revolving Loans and any grants if they become available. In general the City because of its economic status would not generally satisfy the minimum requirements for obtaining a grant as these are generally given to communities with less income per capita than what is typical in the Kansas City Metropolitan Area.

6.5 Recommendation

It is recommended the City begin with improvements to the Raw Water Pump Station. The facility was flooded in 2017, requiring electrical equipment to be replaced on two occasions. The raw water pump station needs to be improved in 2019 to meet future maximum day water demands.

The water treatment plant needs a new Lime Feed Slaker that should be replaced in 2018 since replacement parts are no longer available and the unit is beyond its useful life and the chemical is important to the treatment process.

Estimated maximum day water demands in the Year 2025 are expected to exceed the capacity of the Water Treatment Plant. The proposed improvements for a 5.0 MGD treatment plant will satisfy water demands beyond the 20 year planning period and improve the ability of the Operators to control taste and odor issues originating in the raw water.

For the Distribution System a new 12-inch water supply main between the Water Treatment Plant and North Elevated Water Storage Tank is needed to maintain supply and meet future demands for the North Pressure Zone. The City should complete project number C1, River Crossing by 2020. These improvements would be followed by the improvements around the South Booster Pump Station and eventual replacement of the station to provide sufficient supply and pressure to the South Pressure Zone.

HDR Engineering is ready to assist the City with these projects and looks forward to discussing them further with the Board of Alderman and City Staff.

Appendix A, Cost Estimate Tables

Raw Water Pump Station Improvements	Cost
Zebra Mussels Control	$ 20,000
New Raw Water Pump Sation	$ 1,659,000
Raw Water Valve and Pipe, Dam to Pump Station	$ 366,000
Additional Water Supply	$ 2,000,000
Total (2018$)	**$ 4,045,000**

Zebra Mussels Control

ITEM	QUANTITY	UNIT	UNIT PRICE	EXTENSION
Chemical Feed Pump[1]	1	LS	$ 5,000	$ 5,000
2" Pipe and 3/4" Tubing[2]	200	LF	$ 25	$ 5,000
Meter Vault	1	EA	$ 1,200	$ 1,200
Tapping Tee	1	EA	$ 250	$ 250
Flushing Hydrant	1	LS	$ 2,500	$ 2,500
			Subtotal	$ 13,950
			Contingency (20%)	$ 2,790
			Engineering[2] (15%)	$ 2,511
			Total Project Cost (2018$)	$ 19,251
			Rounded Total (2018$)	**$ 20,000**

[1] Existing Pump May Work

[2] Unit price includes installation cost.

[3] Assumes no RPR, Engineering May Not be Required.

Raw Water Pump Station

ITEM	QUANTITY	UNIT	UNIT PRICE	EXTENSION
[1]Pumps Room for Future Pump	2	EA	$ 150,000	$ 300,000
Variable Frequency Starters	2	EA	$ 35,000	$ 70,000
Elevated Building	1	LS	$ 250,000	$ 250,000
Pipe/Fittings/ Valves	1	LS	$ 88,000	$ 88,000
Chemical Feed Pump Skid	2	LS	$ 23,500	$ 47,000
Fence	200	LF	$ 65	$ 13,000
Grading/Access	1	LS	$ 45,000	$ 45,000
Installation	25%	% ls	-	$92,500
Electrical I&C	30%	% ls	-	$200,100
			Subtotal	$ 1,105,600
			Contingency (20%)	$ 221,120
			Engineering[2] (25%)	$ 331,680
			Total Project Cost (2018$)	$ 1,658,400
			Rounded Total (2018$)	**$ 1,659,000**

[1] Assumes 2 pumps (1 firm, 1 standby), Installation not included.

[2]Engineer to Provide RPR

Construct Valve Vault and 500 Feet of 16-inch Pipe

ITEM	QUANTITY	UNIT	UNIT PRICE	EXTENSION
Pipe/Fittings/ Valves 16"	500	LF	$ 288	$ 144,000
[3]Valve Vault at Connection Points in Dam	2	EA	$ 50,000	$ 100,000
			Subtotal	$ 244,000
			Contingency (20%)	$ 48,800
			Engineering[2] (25%)	$ 73,200
			Total Project Cost (2018$)	$ 366,000
			Rounded Total (2018$)	**$ 366,000**

[2] Assumes Limited RPR Services and Uncontested Cooperation with the USACE.

[3] This work requires work on the side slope of the dam. Actual costs maybe higher due to liability insurance costs or USACE

Maintenance Improvements (2018)	Cost
Lime Feed Equipment Improvements	$ 100,000
General Plant Improvements	$ 101,000
Chemical Building Improements	$ 235,000
Residual Removal	$ 594,000
Total (2018$)	**$ 1,030,000**

Lime Feed Equipment

ITEM	QUANTITY	UNIT	UNIT PRICE	EXTENSION
Replace Lime Feed [1]	1	ls	$ 25,000	$ 25,000
Escalation		%	0.03	-
			Subtotal (2018$)	$ 31,669
New Platform and Pallet Lift	1	ls	$ 23,000	$ 23,000
			Subtotal (2018$)	$ 54,669
Installation	25%	% ls	-	$7,917
Electrical I&C	30%	% ls	-	$9,501
			Subtotal (2018$)	$ 72,087
			Contingency (20%)	$ 14,417
			Engineering[4] (15%)	$ 12,976
			Total Project Cost (2018$)	$ 99,481
			Rounded Total (2018$)	**$ 100,000**

[1] Assumes 40lb bag feeder based on Neosho WTP Improvments (2010$)

General Plant Improvements

ITEM	QUANTITY	UNIT	UNIT PRICE	EXTENSION
Site Improvements [1]	1	ls	$ 8,000	$ 8,000
Filter Pipe Gallery				
Valve Replacement [2]				
Butterfly valves (6-in)	10	ea	$ 3,000	$ 30,000
Globe valves (8-in)	4	ea	$ 6,000	$ 24,000
Pipe Improvements	1	ls	$ 6,000	$ 6,000
Pipe Labels [3]	1	ls	$ 5,000	$ 5,000
			Subtotal	$ 73,000
			Contingency (20%)	$ 14,600
			Engineering[4] (15%)	$ 13,140
			Total Project Cost (2018$)	$ 100,740
			Rounded Total (2018$)	**$ 101,000**

[1] General concrete replacement (splash block, sidewalks) for erosion control

[2] Unit price includes installation cost.

[3] Assumes amterials only, work completed by staff

[4] Assumes no RPR

Existing Chemical Feed Building Improvements

ITEM	QUANTITY	UNIT	UNIT PRICE	EXTENSION
Secondary Containment Improvements	8	CY	$ 800	$ 6,000
Painting [1]	1	LS	$ 20,000	$ 20,000
Electrical Demo	1	LS	$ 20,000	$ 20,000
Pipe Improvements [2]	1	LS	$ 20,000	$ 20,000
Electrical Panel Replacement	1	LS	$ 25,000	$ 25,000
Pumps Skids[3,4]	2	EA	$ 23,500	$ 47,000
Standby Pump	1	EA	$ 8,000	$ 8,000
I&C	30%	% ls	-	$ 24,000
			Subtotal (2018$)	$ 170,000
			Contingency (20%)	$ 34,000
			Engineering[5] (15%)	$ 30,600
			Total Project Cost (2018$)	$ 234,600
			Rounded Total (2018$)	**$ 235,000**

[1] Assumes no lead paint

[2] Feed pipe to be replaced (less than 3-in diameter)

[3] Assumes 2 skids (1 pump per basin - 4 total pumps), misc valve, piping, etc. for poly, chlorine dioxide, and sodium hypo

[4] Assumes existing chemical storage tanks will be reused

[5] Assumes RPR services provided by the City

Residual Removal

ITEM	QUANTITY	UNIT	UNIT PRICE	EXTENSION
Residuals Removal from East and West Residual Basins[1]	642	DT	$ 670	$ 430,140
			Subtotal (2018$)	$ 430,140
			Contingency (20%)	$ 86,028
			Engineering[2] (15%)	$ 77,425
			Total Project Cost (2018$)	$ 593,593
			Rounded Total (2018$)	**$ 594,000**

[1] Dry Tons are Estimated using Date from the 2014 Project

[2] Assumes RPR services to be provided by the City of Smithville.

WTP Capacity Improvements (2025)	Cost
New Primary and Secondary Sedimentation Basins for 5 MGD Plant	$ 5,450,000
Increase Filter Capacity	$ 4,200,000
Total (2018$)	**$ 9,650,000**

New Primary and Secondary Sedimentation Basins

ITEM	QUANTITY	UNIT	UNIT PRICE	EXTENSION
Rapid Mix Basins [1]	8	LS	$ 18,000	$ 144,000
Mixers	12	EA	$ 10,000	$ 120,000
Sedimentation Basins [2]	3	EA	$ 720,000	$ 2,160,000
Demo Existing Secondary Clarifier	1	EA	$ 100,000	$ 100,000
Lime Feed Equipment (Liquid Feed System)[3]	1	LS	$ 185,000	$ 185,000
Flow Meter and Vault	3	LS	$ 30,000	$ 90,000
Splitter Box	2	LS	$ 20,000	$ 40,000
Site Piping	2400	LF	$ 160	$ 384,000
Site Grading/Access	1	LS	$ 80,000	$ 80,000
Site Electrical and Instrumentation & Controls (10%)	1	LS	$ 330,000	$ 330,000
			Subtotal (2018$)	$ 3,633,000
			Contingency (20%)	$ 726,600
			Engineering (25%)[4]	$ 1,089,900
			Total Project Cost (2018$)	$ 5,449,500
			Rounded Total (2018$)	**$ 5,450,000**

[1] Assumes 1 rapid basin per chemical dosed. Primarys require 3 total. Secondaries will rquire 3 total (shared).

[2] Assumes 2 primary and 1 secondary sedimentation basins and Electrical and I&C

[3] Includes Bulk Tank, 2 Day Tanks, 2 Pump Skids, Feed Piping and Building

[4] Includes RPR

Increase Filter Capacity

ITEM	QUANTITY	UNIT	UNIT PRICE	EXTENSION
New Filter and High Service Pump Building [1&2]	1	LS	$ 2,355,000	$ 2,355,000
Site Piping	1	LS	$ 91,200	$ 91,200
Site Grading/Drive Changes/ Fence	1	LS	$ 98,600	$ 98,600
Electrical, Instrumentation and Controls (10%)	1	LS	$ 255,000	$ 255,000
			Subtotal (2018$)	$ 2,799,800
			Contingency (20%)	$ 559,960
			Engineering[3] (25%)	$ 839,940
			Total Project Cost (2018$)	$ 4,199,700
			Rounded Total (2018$)	**$ 4,200,000**

[1] Assumes 5 filter bays for future 5 MGD Flow. Only two of the Bays need media at this time.

[2] Includes Three New High Service Pumps

[3] Includes RPR

Appendix B, MDNR Chlorine “CT” Tables

GUIDANCE MANUAL
FOR
SURFACE WATER SYSTEM
TREATMENT REQUIREMENTS

January 1992

MISSOURI DEPARTMENT OF NATURAL RESOURCES
Public Drinking Water Program

PUB416

TABLE-15

CT VALUES FOR INACTIVATION OF VIRUSES BY FREE CHLORINE[1,2]

Log Inactivation	2.0		3.0		4.0	
pH	6-9	10	6-9	10	6-9	10
Temperature						
0.5°C	6	45	9	66	12	90
5.0°C	4	30	6	44	8	60
10.0°C	3	22	4	33	6	45
15.0°C	2	15	3	22	4	30
20.0°C	1	11	2	16	3	22
25.0°C	1	7	1	11	2	15

Notes:

1. Data adapted from Sobsey (1988) for inactivation of Hepatitis A virus (HAV) at pH = 6, 7, 8, 9, and 10 and temperature = 5°C. CT values include a safety factor of 3.

2. CT values adjusted to other temperatures by doubling CT for each 10°C drop in temperature.

TABLE-16

CT VALUES FOR INACTIVATION OF GIARDIA CYSTS BY CHLORINE DIOXIDE pH 6-9

Inactivation	Temperature					
	≤1°C	5°C	10°C	15°C	20°C	25°C
0.5 log	10.0	4.3	4.0	3.2	2.5	2.0
1.0 log	21.0	8.7	7.7	6.3	5.0	3.7
1.5 log	32.0	13.0	12.0	10.0	7.5	5.5
2.0 log	42.0	17.0	15.0	13.0	10.0	7.3
2.5 log	52.0	22.0	19.0	16.0	13.0	9.0
3.0 log	63.0	26.0	23.0	19.0	15.0	11.0

TABLE-10 CT VALUES FOR INACTIVATION OF GIARDIA CYSTS BY FREE CHLORINE AT 5°C

Cl_2 CONC. mg/L	pH<=6 LOG INACTIVATIONS 0.5	1.0	1.5	2.0	2.5	3.0
≤0.4	16	32	49	65	81	97
0.6	17	33	50	67	83	100
0.8	17	34	52	69	86	103
1.0	18	35	53	70	88	105
1.2	18	36	54	71	89	107
1.4	18	36	55	73	91	109
1.6	19	37	56	74	93	111
1.8	19	38	57	76	95	114
2.0	19	39	58	77	97	116
2.2	20	39	59	79	98	118
2.4	20	40	60	80	100	120
2.6	20	41	61	81	102	122
2.8	21	41	62	83	103	124
3.0	21	42	63	84	105	126

Cl_2 CONC. mg/L	PH=6.5 LOG INACTIVATIONS 0.5	1.0	1.5	2.0	2.5	3.0
≤0.4	20	39	59	78	98	117
0.6	20	40	60	80	100	120
0.8	20	41	61	81	102	122
1.0	21	42	63	83	104	125
1.2	21	42	64	85	106	127
1.4	22	43	65	87	108	130
1.6	22	44	66	88	110	132
1.8	23	45	68	90	113	135
2.0	23	46	69	92	115	138
2.2	23	47	70	93	117	140
2.4	24	48	72	95	119	143
2.6	24	49	73	97	122	146
2.8	25	49	74	99	123	148
3.0	25	50	76	101	126	151

Cl_2 CONC. mg/L	pH=7.0 LOG INACTIVATIONS 0.5	1.0	1.5	2.0	2.5	3.0
≤0.4	23	46	70	93	116	139
0.6	24	48	72	95	119	143
0.8	24	49	73	97	122	146
1.0	25	50	75	99	124	149
1.2	25	51	76	101	127	152
1.4	26	52	78	103	129	155
1.6	26	53	79	105	132	158
1.8	27	54	81	108	135	162
2.0	28	55	83	110	138	165
2.2	28	56	85	113	141	169
2.4	29	57	86	115	143	172
2.6	29	58	88	117	146	175
2.8	30	59	89	119	148	178
3.0	30	61	91	121	152	182

Cl_2 CONC. mg/L	pH=7.5 LOG INACTIVATIONS 0.5	1.0	1.5	2.0	2.5	3.0
≤0.4	28	55	83	111	138	166
0.6	29	57	86	114	143	171
0.8	29	58	88	117	146	175
1.0	30	60	90	119	149	179
1.2	31	61	92	122	153	183
1.4	31	62	94	125	156	187
1.6	32	64	96	128	160	192
1.8	33	65	98	131	163	196
2.0	33	67	100	133	167	200
2.2	34	68	102	136	170	204
2.4	35	70	105	139	174	209
2.6	36	71	107	142	178	213
2.8	36	72	109	145	181	217
3.0	37	74	111	147	184	221

Cl_2 CONC. mg/L	pH=8.0 LOG INACTIVATIONS 0.5	1.0	1.5	2.0	2.5	3.0
≤0.4	33	66	99	132	165	198
0.6	34	68	102	136	170	204
0.8	35	70	105	140	175	210
1.0	36	72	108	144	180	216
1.2	37	74	111	147	184	221
1.4	38	76	114	151	189	227
1.6	39	77	116	155	193	232
1.8	40	79	119	159	198	238
2.0	41	81	122	162	203	243
2.2	41	83	124	165	207	248
2.4	42	84	127	169	211	253
2.6	43	86	129	172	215	258
2.8	44	88	132	175	219	263
3.0	45	89	134	179	223	268

Cl_2 CONC. mg/L	pH=8.5 LOG INACTIVATIONS 0.5	1.0	1.5	2.0	2.5	3.0
≤0.4	39	79	118	157	197	236
0.6	41	81	122	163	203	244
0.8	42	84	126	168	210	252
1.0	43	87	130	173	217	260
1.2	45	89	134	178	223	267
1.4	46	91	137	183	228	274
1.6	47	94	141	187	234	281
1.8	48	96	144	191	239	287
2.0	49	98	147	196	245	294
2.2	50	100	150	200	250	300
2.4	51	102	153	204	255	306
2.6	52	104	156	208	260	312
2.8	53	106	159	212	265	318
3.0	54	108	162	216	270	324

Cl_2 CONC. mg/L	pH=9.0 LOG INACTIVATIONS 0.5	1.0	1.5	2.0	2.5	3.0
≤0.4	47	93	140	186	233	279
0.6	49	97	146	194	243	291
0.8	50	100	151	201	251	301
1.0	52	104	156	208	260	312
1.2	53	107	160	213	267	320
1.4	55	110	165	219	274	329
1.6	56	112	169	225	281	337
1.8	58	115	173	230	288	345
2.0	59	118	177	235	294	353
2.2	60	120	181	241	301	361
2.4	61	123	184	245	307	368
2.6	63	125	188	250	313	375
2.8	64	127	191	255	318	382
3.0	65	130	195	259	324	389

NOTE: $CT_{99.9}$=CT for 3-log inactivation

Appendix C, Water Distribution System Map