WATER TREATMENT PLANT NO. 5 PRELIMINARY DESIGN REPORT

FOR

SEPTEMBER 2017

I hereby certify that this report was prepared by me or under my direct supervision and that I am a duly Registered Professional Engineer under the laws of the State of Minnesota.

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CHAPTER 1 INTRODUCTION

1.1 Background

The City of Edina is a community with approximately 50,770 people, located in Hennepin County along Interstate-494 in the southwest Minneapolis/St. Paul metropolitan area. The City experienced significant growth throughout the 1950's and 1960's, and considered itself fully developed by the mid-1970s. Focus has now shifted from new development to infill and redevelopment. The City of Edina is committed to offering a high quality of life to the residents of the City, which includes providing an adequate supply of safe drinking water at a reasonable cost.

The City operates two separate water systems. The first is the Morningside system in the northeast corner of the City that gets its water from the City of Minneapolis where it undergoes ultrafiltration, lime softening and multiple chemical treatments prior to reaching the distribution system. Edina's Utility Department is responsible for maintaining the system piping in the Morningside system service area.

The second is the Edina system that currently consists of eighteen (18) raw water supply wells, four (4) Water Treatment Plants (WTP), four (4) water towers, one (1) ground storage reservoir, and over 200 miles of water main. The City treats all well water with fluoride for dental health, chlorine for disinfection, and polyphosphates to inhibit pipe corrosion. In addition, ten (10) of the wells are pumped to one (1) of the four (4) existing WTPs where the primary treatment goal is removal of iron and manganese through oxidation and granular filtration. The newest of the facilities, WTP No. 6, includes air-stripping towers to remove vinyl chloride and provisions to install the equipment to feed hydrous manganese oxide (HMO) to remove the high radium levels from Well No. 9. The vinyl chloride entered the aquifer and created a chemical plume that migrated into Edina from St. Louis Park. WTPs No. 3 and No. 4 include the equipment to remove high levels of radium chemically from the raw water by the addition of hydrous manganese oxide (HMO).

Currently, based on the last 10 years of available data, the City provides an average of approximately 6.9 MGD of water supply to primarily residential, commercial and light industrial water users. Maximum day water demands have exceeded 18.8 MGD (2009 peak) in the past as the result of peak summer daily system demands. There are several smaller residential and commercial areas in Edina served by the cities of Bloomington, Eden Prairie, Minneapolis, and St. Louis Park. The City of Edina has inter-community water use agreements with the cities of Bloomington, Eden Prairie, Hopkins, and Minneapolis that provides the ability to interconnect in the event of emergency.

The iron and manganese that naturally occurs in the groundwater is a common source of complaints from the City's customers due to their staining and color effects in the water supply. These complaints are elevated during the summer when the City uses the unfiltered wells to meet the peak demand. Edina is looking to add more treatment capacity to its system to make improvements in the quality of water supplied to its customers.

In addition to the aesthetic issues related to manganese, the Minnesota Department of Health (MDH) published a Health Risk Assessment Unit related to manganese and drinking water in February 2016. In this document, MDH suggests a guidance value for manganese of 100 parts per billion (ppb) for formula-fed infants that drink tap water and 300 ppb for children and adults. The document states that too much manganese in drinking water may affect learning and behavior in infants and the potential for neurological problems over time for children and adults.

As the City looks to redevelop the Southdale District into higher density and mixed-use development over the next few decades, increased water demand will accompany this change and growth. Wells 5 and 18, which are currently unfiltered, serve this southeastern portion of the City. These two (2) wells currently provide water during peak summer demands, but would likely require more frequent use as redevelopment of the area they serve occurs. Wells 5 and 18 have reported iron and manganese levels that exceed the EPA Secondary Maximum Contaminant Level (SMCL) for drinking water. There are no regulations enforced by the EPA for Secondary MCLs, but are in place as recommended values to mitigate the aesthetic concerns in drinking water.

Other contaminants of concern for WTP No. 5 are radionuclides including gross alpha and combined radium. The EPA regulates these contaminants and set maximum contaminant levels (MCL) for gross alpha at 15.0 pCi/L and for combined radium at 5.0 pCi/L. Historically, Wells 5 and 18 have reported gross alpha and combined radium levels below the regulated MCLs. In some instances, the combined radium results have indicated levels at approximately 80% of the MCL. As a conservative approach, the Project Team incorporated radium removal technologies in the design of WTP No. 5, acknowledging that radionuclides change over time, and have the potential of increasing for the facilities source wells with increased well pumping. The adverse health effects associated with exposure to radionuclides include radiotoxicity, which affects human tissue, and chemotoxicity, which affects human organs. Research links extended radionuclide exposure to cancer.

The City aims to construct WTP No. 5 to provide additional treatment capacity and meet the needs of the growing community and to produce more filtered water to reduce aesthetic complaints throughout the distribution system. The siting and planning for WTP No. 5 began over a decade ago. Within this decade, the City built WTP No. 6 and decommissioned WTP No. 1. As part of the planning, the City secured easements for the lot directly adjacent the

Southdale Tower and partially extended raw water pipes to connect the planned facility with its source wells: Well 5, 18 and a potential future Well 21.

In addition to the preferred site located near the Southdale Tower, the City wishes to explore alternative sites for consideration of economic development tradeoffs. Alternative sites include the Yorktown Site located near the YMCA and Fire Station No. 2, the Median Site located adjacent to Well No. 5 along West 69th Street, and the Fred Richards Site located near existing WTP No. 3 within the recently closed golf course parking lot. Each site offers unique opportunities related to integration into the City's development plans and environmental sustainability.

1.2 The Preliminary WTP Design Report Process

Ensuring the responsible management of annual operation and maintenance budgets, optimizing short-term capital improvement expenditures, and maximizing the benefits of long-term capital improvements requires a comprehensive direction. To establish a vision for the addition of WTP No. 5 to the Edina water treatment system, the City authorized preparation of the WTP Preliminary Design Report.

The process provides a means that the City of Edina can assess existing needs, project future needs, evaluate alternatives, and develop a priority and strategy for improvement options based on the best available information. The City retained Advanced Engineering and Environmental Services, Inc. (AE2S) to prepare a WTP Preliminary Design Report.

This WTP Preliminary Design Report (PDR) will inform policymakers and the public on the condition of existing infrastructure, requirements and alternatives for the water treatment process, opinions of cost, and the recommended steps to implement the preferred alternative. The City of Edina is especially interested in the evaluation of the future facility to provide economic and environmental sustainability. For this reason, the report includes the investigation of the economic and environmental tradeoffs for each of the site alternatives. Furthermore, this Design Report will provide for the implementation of desired improvements within the context of a comprehensive plan to ensure compatibility and prudent management of the water utility.

1.3 Purpose and Scope

AE2S developed this WTP No. 5 Preliminary Design Report through a collaborative planning process with representatives of the City and the Project Team. The following list summarizes the objectives of the WTP Preliminary Design Report:

1. Prepare and update water demand projections for the planning period.
2. Establish treatment capacity and water quality objectives based on source water analysis.
3. Evaluate the four (4) alternative sites for the proposed WTP No. 5 and consider economic development and environmental sustainability tradeoffs for each site.
4. Determine whether there is a need for additional system storage.
5. Review existing, proposed and anticipated regulations of the Safe Drinking Water Act (SDWA).
6. Identify strategic criteria for the evaluation of process treatment technologies.
7. Identify and evaluate treatment process technologies and alternatives on a planning level basis.
8. Provide planning level opinions of capital and life cycle costs for preferred treatment alternatives.
9. Based on evaluation with respect to established criteria and planning level opinions of cost, identify the recommended alternative(s) for a WTP.

The following chapters present the findings and results of the work performed to satisfy each of the stated objectives of the WTP Preliminary Design Report. Supplemental information comprises the Appendix.

CHAPTER 2 OVERVIEW OF EXISTING SYSTEM

2.1 Existing Water Supply Well Overview

The City of Edina has eighteen (18) active groundwater appropriation permits authorized by the Minnesota Department of Natural Resources (MnDNR). The cumulative appropriations provide for an instantaneous withdrawal rate of 17,650 gpm (approximately 25.42 MGD) and a total annual withdrawal volume of 3,000 MG/year (which equates to an average daily withdrawal of 8.22 MGD). Each of the eighteen (18) raw water supply wells is unique in its location and production capabilities. Table 2.1 presents the pumping rate for each well.

Table 2.1 Well Characteristics and Pumping Rates

Well Name	Unique Well No.	Well Use Status	Well Depth (ft.)	Source Aquifer[2]	Pumping Rate (gpm)
No. 2	208399	Active	448	OPDCCJDN	850
No. 3	240630	Active	496	OPDCCJDN	1,000
No. 4	200561	Active	500	OPDCCJDN	1,000
No. 5	206377	Active	443	OPDCCJDN	1,000
No. 6	200564	Active	505	OPDCCJDN	1,000
No. 7	206474	Active	547	OPDCCJDN	1,000
No. 8	204884	Active	472	OPDCCJDN	800
No. 9	206588	Inactive[1]	1,130	CMTS	1,000
No. 10	206184	Active	1,001	CMTS	1,000
No. 11	206183	Active	403	CJDN	1,000
No. 12	203614	Active	1,080	CMTS	1,000
No. 13	203613	Active	495	CJDN	1,000
No. 15	207674	Active	475	OPDCCJDN	1,000
No. 16	203101	Active	381	OPDCCJDN	1,000
No. 17	200914	Active	461	CJDN	1,000
No. 18	200918	Active	446	CJDN	1,000
No. 19	505626	Active	520	CJDN	1,000
No. 20	686286	Active	467	CJDN	1,000
				Total	**17,650 (25.42 MGD)**

[1] Inactive due to high levels of radium in the raw water. Well 9 inactive since 2010.
[2] OPDCCJDN: Prairie du Chien – Jordan, CMTS: Mt. Simon, CJDN: Jordan.

As the City of Edina continues to redevelop into higher density land use, additional water supply resources may be required to provide for increased water demand. The MnDNR will heavily scrutinize any future request for additional raw water appropriation. Groundwater resources throughout the Minneapolis Metropolitan area are limited, and recent activities have increased the sensitivity to the availability of the water supply. It will be imperative that the City of Edina demonstrate that every effort has been made to optimize the performance of each existing well and that steps have been taken to ensure efficient use of the current water resources, prior to any future requests for new/additional groundwater appropriation.

2.1.1 Well Performance Considerations

Performance of each well is a function of many separate variables that can have varying effects. Physical characteristics, proximity to other wells, and general maintenance are all important considerations of optimizing the production of each ground water source.

When pumping a well, the level of the groundwater surface near the well lowers. Drawdown is the difference between the static water level and the pumping water level at the well. The pumping of groundwater from a well causes the groundwater surface near the well to take the shape of an inverted cone, called the well's cone of depression. As the distance from the well increases, the influence from the pumping decreases until the slope from the cone of influence merges with the static water level in the aquifer. The radius of influence of the well is the distance of impact. A well's cone of depression and its subsequent radius of influence are dependent upon the pumping rate of the well and the hydraulic characteristics of the aquifer including transmissivity, aquifer thickness, recharge sources, and lateral extent of the aquifer.

When two (2) or more wells have cones of depression that overlap, the groundwater surface at each of these wells lowers cumulatively. The impact of this overlap is well interference. Under these conditions, the impact from well interference on the head conditions for the individual well to be developed shall be included within the design and sizing of the well. Consequently, it is good practice to have a wider distribution of well installations throughout the lateral extent of the aquifer to reduce pumping costs and minimize well interferences. The well separation should be determined from a distance – drawdown plot as part of a well development program for the aquifer.

The decentralized layout of the City's water supply and treatment systems provide for maximum distances between the wells and therefore mitigate any adverse effects of well interference. However, each of the four (4) existing WTP sites contain at least two (2) water supply wells within a relatively close proximity. The presence of multiple wells at each WTP provides both redundancy and reliability, as well as operational flexibility. Original installation of wells was at separation distances adequate to reduce interference, but the City should monitor well performance and drawdown over time to ensure continued performance of the wells.

When considering the placement of future Well 21, the City should establish a well location to maintain the appropriate separation between existing and potential sources of contamination and to minimize the potential impact on other wells, according to Ten States Standards. Minnesota Rules require that wells be located no less than 50 feet from a source of contamination, including buried sewers, and that the public water system (PWS) maintain a permanent easement of the property within a 50-foot radius of the well. Minnesota Rules also require that PWSs develop a Wellhead Protection/Source Water Protection Program to prevent human-caused contaminants from entering the source waters used for public water supplies.

The City of Edina completed updates to their original Wellhead Protection Plan in June 2011 (Part I) and April 2013 (Part II). Part I delineated the wellhead protection area (WHPA) and the drinking water supply management area (DWSMA). This part provides vulnerability assessments for each City well and source water aquifer within the DWSMA. The City's DWSMA increased with the update to include almost the entire City of Edina, all of Hopkins, and portions of Minneapolis, Richfield, Bloomington, Eden Prairie, Minnetonka, Plymouth, and Golden Valley. Part II identified 183 potential contaminant sources within the DWSMA. According to Part I, Well No. 5 is not vulnerable due to the presence of a confining geologic layer, and Well No. 18 is vulnerable due to a high DNR geologic sensitivity rating.

According to Ten States Standards, the capacity of a developed groundwater supply system shall equal or exceed the projected maximum day demand (MDD) with the largest producing well out of service. The amount of groundwater an individual production well can pump is dependent upon the hydraulic characteristics of the aquifer, the recharge rate of the aquifer, and the well construction itself. Characteristics of the aquifer may not be conducive to providing the desired well capacity objectives. In these situations, the supplier can develop individual wells at reduced capacities or operate the well system on a rotating cycle. Under these scenarios, additional wells may be necessary to meet the water supply objectives.

The efficiency of the well can also limit the yield from a well. The efficiency of the well is largely dependent upon the design and construction of the well. AWWA Standard A100-97 includes guidelines to consider when constructing production water wells. A better-designed and constructed well provides greater well efficiency, or ease of the flow of groundwater from the aquifer into the well. Specific capacity is the basic measure of the performance of a well, with higher values signifying a greater yield capability. The pumping rate divided by the stabilized drawdown in the production well is the specific capacity of a pumping well. The units of gallons per minute per foot (gpm/ft) of drawdown expresses this relationship.

2.1.2 Existing Raw Water Supply Wells

The City of Edina has eighteen (18) wells in total that provide water to the customers within the Edina water utility. The wells vary in depth from 391 to 1,130 feet and have capacities ranging from 800 gpm to 1,000 gpm. Although well construction was similar for all groundwater wells throughout the City, each of these wells has its own unique characteristics. This section summarizes general information related to each well within the current water supply system.

Table 2.2 summarizes which wells supply each of the existing treatment facilities. Past studies and reports on the Edina water system, as well as data provided by the City helped identify the historical contaminant ranges for each well. These characteristics help the City identify required treatment technologies within each facility and indicate wells that have limited use due to untreated contaminants. Bolded concentrations indicate that the contaminant level is above the EPA regulated MCL or non-regulated SMCL.

High contaminants are those that have historic concentrations greater than the EPA primary (MCL) or secondary maximum contaminant level (SMCL). The MCL for Vinyl Chloride is 0.002 mg/L, for combined Radium-226 and Radium-228 is 5 pCi/L, and for alpha particles is 15 pCi/L. While the EPA does not regulate SMCL's, the concentrations are recommendations to mitigate customer complaints related to finished water aesthetics. The SMCL for iron is 0.3 mg/L and 0.05 mg/L for manganese.

Table 2.2 Treatment and Well Contaminant Summary for Active Wells

Well Name	Treatment Facility	Iron (mg/L)	Manganese (mg/L)	Radium-226 and Radium-228 (pCi/L)	Gross Alpha (pCi/L)
No. 2	WTP No. 6	**0.70 – 0.95**	**0.053 – 0.21**	2.03	3.9
No. 3	NA	<0.01 – **0.65**	<0.01 – **0.072**	1.9	5.4
No. 4	WTP No. 2	**0.61 – 0.71**	0.033 – **0.05**	**5.3**	13.6
No. 5	NA	**0.39 – 0.51**	**0.05 – 0.375**	1.3 - 4.1	5.0 – 8.1
No. 6	WTP No. 2	0.10 – **0.56**	<0.02 – **0.085**	2.26	4.6
No. 7	WTP No. 6	<0.01	<0.01 – **0.065**	2.5	10.1
No. 8	NA	**0.40 – 0.57**	**0.24 – 0.322**	1.71	7.3
No. 9	Inactive	**1.00**	**0.05**	**7.3**	**23.8**
No. 10	WTP No. 3	**0.42 – 0.82**	<0.02 – 0.021	**16.4**	**21.3**
No. 11	WTP No. 3	**0.46 – 0.59**	0.04 – **0.054**	**8.6**	11.3
No. 12	WTP No. 4	**0.51 – 4.0**	0.01 – **0.086**	**8.7**	**22.5**
No. 13	WTP No. 4	**0.55 – 0.69**	0.041 – **0.055**	**5**	**22.9**
No. 15	WTP No. 6	**0.48 – 1.5**	0.03 – **0.16**	3.8	7.5

Well Name	Treatment Facility	Iron (mg/L)	Manganese (mg/L)	Radium-226 and Radium-228 (pCi/L)	Gross Alpha (pCi/L)
No. 16	NA	**0.05 – 0.52**	<0.02 – 0.04	2.0	6.3
No. 17	WTP No. 2	**0.53 – 1.1**	<0.02 – 0.048	4.6	**24.3**
No. 18	NA	**0.32 – 0.53**	**<0.02 – 0.22**	3.2 – 4.0	6.2 – 7.9
No. 19	NA	**0.51 – 0.58**	0.033 – 0.038	3.0	10.9
No. 20	NA	**0.42**	0.031 – 0.041	-	-

One (1) of the wells, Well 9, is currently inactive due to high levels of radium. Based on annual water use data provided by the City, the last time Well 9 was online was in 2010. Design of WTP No. 6 included treatment of Well 9. The City has not installed the HMO feed equipment for this well to date, but plan to have the well and HMO system online within the next year.

This preliminary design report does not include an extensive analysis on all wells. The existing well data identifies the range of constituents reported for the City's system. The Project Team used the ranges to make conservative design considerations for WTP No. 5 without knowing the water quality of the future Well 21.

2.1.3 Wells to Service WTP No. 5

Well 5, Well 18, and future Well 21 will provide raw water to the future WTP No. 5. The following sections summarize detailed information regarding conditions and water quality of the two (2) existing wells.

2.1.3.1 *Well No. 5*

Well No. 5 is located south of the Southdale Tower within the median of W 69th Street on the east side of the France Ave S and W 69th St intersection. Original drilling of the well occurred in 1950. Bergerson-Caswell reconstructed the well in 2002. The water level during pumping is approximately 90 feet below the surface. During reconstruction of the well, test pumping indicated that the well has a specific capacity of approximately 24.0 gpm/ft.

The existing pump is a 100 horsepower (Hp) J-Line vertical turbine pump designed to pump 1,000 gpm at an estimated total dynamic head (TDH) of 310 feet. The TDH was approximated based on the 90 foot pumping level and an assumed distribution system pressure of 90 psi, which was identified in the water distribution system analysis completed for the City's 2008 Comprehensive Plan. The existing average day pressure for the area surrounding Well No. 5 was reported to be between 90 and 100 psi. The 90 psi corresponds to 210 feet of TDH. An additional 10 feet of head was assumed to account for minor losses through the pump column and discharge head. This TDH assumption corresponds well with the pump performance curve that is provided in ***Appendix A***. The well characteristics for Well No. 5 are provided in Table 2.3.

Table 2.3 Well No. 5 Characteristics

Well Characteristic	**Well No. 5**
Unique Well No.	206377
Date Reconstructedh	6/13/2002
Formation	Prairie du Chien - Jordan
Pump Hp	100
Setting Depth (ft.)	153
Depth (ft.)	443
Diameter (in.)	16
Outer Casing (in.)	16/20/24
Open Hole Depth (ft.)	186
Pumping Rate (gpm)	1,000
Static Level (ft.)	78
Pumping Level (ft.)	90
Specific Capacity (gpm/ft.)	83.3

Table 2.4 summarizes the raw water quality of this well. The range of concentrations for each contaminant ensures the Project Team is basing treatment sizing considerations on the highest

historical concentration. The well house includes systems to dose fluoride for public wellness, chlorine for disinfection, and polyphosphates for pipe corrosion inhibition.

Well No. 5 has elevated concentrations of iron and manganese. The well also has combined Radium-226 and Radium-228 close to the EPA regulated MCL of 5 pCi/L. The well is currently only used during peak summer demand and emergencies. The proposed facility will include treatment technologies to oxidize and filter out the iron and manganese and provisions to add radium removal equipment in the future if radium concentration increases over time.

Table 2.4 Well No. 5 Water Quality

Analyte	Concentration Range
Ammonia (mg/L)	0.12
Iron (mg/L)	0.39 – 0.57
Manganese (mg/L)	0.05 – 0.409
Nitrate + Nitrite as Nitrogen (mg/L)	<0.05
pH	7.8
Sulfate (mg/L)	44.9 – 46.2
Sodium (mg/L)	11.8 – 12.4
Radium-226 + Radium-228 (pCi/L)	1.3 – 4.1
Gross Alpha (pCi/L)	5.0 – 8.1

2.1.3.2 *Well No. 18*

Well No. 18 is located along York Ave S in the parking lot of Edina Fire Station No. 2. Keys Well Drilling Company drilled the well back in 1973. The water level during pumping is approximately 90 feet below the surface. The well has a specific capacity of approximately 83.3 gpm/ft.

The existing pump is a 125 horsepower (Hp) Peerless vertical turbine pump designed to pump 1,000 gpm at an estimated total dynamic head (TDH) of 320 feet. The TDH was approximated based on the 103 foot pumping level and an assumed distribution system pressure of 90 psi, which was identified in the water distribution system analysis completed for the City's 2008 Comprehensive Plan. The existing average day pressure for the area surrounding Well No. 18 was reported to be between 90 and 100 psi. The 90 psi corresponds to 210 feet of TDH. An additional 7 feet of head was assumed to account for minor losses through the pump column and discharge head. This TDH assumption corresponds well with the pump performance curve that is provided in ***Appendix B***. The well characteristics for Well No. 18 are provided in Table 2.5.

Table 2.5 Well No. 18 Characteristics

Well Characteristic	**Well No. 18**
Unique Well No.	200918
Date Drilled	10/16/1973
Formation	Jordan
Pump Hp	125
Setting Depth (ft.)	120
Depth (ft.)	446
Diameter (in.)	16
Outer Casing (in.)	16/20/24
Open Hole Depth (ft.)	186
Pumping Rate (gpm)	1,000
Static Level (ft.)	61.3
Pumping Level (ft.)	103
Specific Capacity (gpm/ft.)	24.0

Table 2.6 shows the water quality of this well. The range of concentrations for each contaminant ensures the Project Team is basing treatment sizing considerations on the highest historical concentration. The well house includes systems to dose fluoride for public wellness, chlorine for disinfection, and polyphosphates for pipe corrosion inhibition.

Well No. 18 has elevated concentrations of iron and manganese. The well also has combined Radium-226 and Radium-228 close to the EPA regulated MCL of 5 pCi/L. The well is currently only used during peak summer demand and for emergencies. The proposed facility will include treatment technologies to oxidize and filter out the iron and manganese and will include provisions to add radium removal equipment in the future if radium concentration increases over time.

Table 2.6 Well No. 18 Water Quality

Analyte	Concentration Range
Ammonia (mg/L)	0.17
Iron (mg/L)	0.32 – 0.53
Manganese (mg/L)	<0.02 – 0.26
Nitrate + Nitrite as Nitrogen (mg/L)	<0.05
pH	7.8
Sulfate (mg/L)	44.1 – 45.7
Sodium (mg/L)	18.5 – 20.5
Radium-226 + Radium-228 (pCi/L)	3.2 – 4.0
Gross Alpha (pCi/L)	6.2 – 7.9

2.2 Existing Treatment System Overview

The existing water treatment system consists of four (4) regional WTPs located near the groundwater supply wells. The following sections summarize the existing treatment technologies and facilities. Site visits and conversations with City staff aided in providing the information contained in these sections.

2.2.1 Existing Water Treatment Technologies

The City's existing system includes many comparable treatment technologies throughout their facilities. This is in part due to relatively similar water quality seen throughout the City's existing treated wells. The City installed equipment and chemical feed systems to treat additional contaminants as required. The following is a list of water treatment technologies present throughout the existing facilities:

- Pre-oxidation of iron and manganese with chlorine fed upstream of the filters.
- Pressure filtration to remove the oxidized iron and manganese.
- Chlorine addition to provide disinfection. Until recently, the City used breakpoint chlorination. Within the last year, the City switched to chloramination where chlorine consumes the raw water ammonia and forms chloramines that provide disinfection.
- Baseline chemical feed systems including chlorine, fluoride, and orthophosphate / polyphosphate blend.
- Air-stripping units to remove vinyl chloride (WTP No. 6 only).

- Radium removal by addition of HMO (WTP No. 3 and No. 4 only).

Manganese concentrations for the currently treated wells is relatively low, so manganese removal is not a primary treatment goal. The facilities do not provide additional detention upstream of the filters, and due to the time required to oxidize manganese using chlorine, it is unlikely that the existing facilities remove manganese with the current treatment technologies.

The existing water treatment plants treat ten (10) of the City's wells within the four (4) existing WTPs where the primary treatment goal is removal of iron and manganese through oxidation and granular filtration. The newest of the facilities, WTP No. 6, includes air-stripping towers to remove vinyl chloride and provisions to install the equipment to feed HMO within the Well No. 6 well house to remove the well's high radium levels. WTPs No. 3 and No. 4 already include the HMO feed equipment to remove high levels of radium chemically from the raw water. Table 2.7 summarizes the source water wells, well pumping capacities, and design and current capacities of the four (4) existing facilities.

Table 2.7 Summary of Existing Water Treatment Plant Capacities

Treatment Facility	Well ID	Pumping Capacity (gpm)	Combined Plant Capacity (gpm) Design/Current	Combined Plant Capacity (MGD) Design/Current
WTP No. 2	No. 4	850	2,850	4.10
	No. 6	1,000		
	No. 17	1,000		
WTP No. 3	No. 10	1,000	2,000	2.88
	No. 11	1,000		
WTP No. 4	No. 12	1,000	2,000	2.88
	No. 13	1,000		
WTP No. 6	No. 2	1,000	4,000 / 2,500[2]	5.76 / 3.60
	No. 7	1,000		
	No. 9[1]	1,000		
	No. 15	1,000		
Currently Unfiltered	No. 3	1,000		
	No. 5	1,000		
	No. 8	800		
	No. 16	1,000		
	No. 18	1,000		
	No. 19	1,000		
	No. 20	1,000		
Existing Filter Water Capacity			**10,850 / 9,350**	**15.6 / 13.5**

[1] Well No. 9 currently inactive.
[2] Combined plant capacity is limited by the facility effluent piping. Distribution pressure is too high with all wells operating at full pumping capacity.

2.2.2 Existing Water Treatment Plants

All four (4) existing facilities have dual pressure filters with silica sand filtration media originally designed to remove raw water iron and manganese. The exception to the silica sand media is in WTP No. 6, which has conventional dual sand and anthracite media. Pre-chlorine feed at the well sites oxidizes the iron and manganese prior to filtration. All facilities are equipped with chemical feed systems to dose chlorine for oxidation and disinfection, fluoride for public health and wellness, and an orthophosphate / polyphosphate blend to inhibit pipe corrosion. Post-chlorination equipment exists, but the City does not use it for current operation.

Every facility includes a backwash reclaim system that reclaims backwash water to the front of the system and wastes sludge to sanitary. Current operations base filter backwash frequency on an iron breakthrough concentration of 0.1 to 0.15 mg/L, depending on the facility.

In addition to the common components of all facilities, each facility has its own unique components and operational characteristics. A brief description of these items follows.

2.2.2.1 *WTP No. 2*

WTP No. 2 is located off Highway 100, just north of County Road 62, near Southview Middle School and the Kuhlman Field complex. The Community Center Tower is located just south of the facility. This facility treats the water from Well No. 4, No. 6, and No. 17. Historical raw water characteristics include iron concentrations above the SMCL of 0.3 mg/L for all three wells and manganese concentrations at or above the SMCL of 0.05 mg/L for Well No. 4 and No. 6. For data related to radionuclides, historical data reports combined radium above the MCL of 5 pCi/L for Well No. 4 and gross alpha above the MCL of 15 pCi/L for Well No. 17. No additional chemical feeds exist for WTP No. 2. Operations staff noted that Well No. 17 is never operated standalone due to high concentrations of combined radium and gross alpha.

2.2.2.2 *WTP No. 3*

WTP No. 3 is located in the southeast corner of the City off Parklawn Avenue. The City built the facility within the Fred Richards Golf Course clubhouse, cart storage, and maintenance building complex, blending it into the structures through common architecture. Wells No. 10 and No. 11 provide influent to the facility. Historically both wells have iron concentrations above the SMCL and only

Well No. 11 and low levels of manganese. Raw water manganese levels are just above the SMCL, so no additional treatment measures for manganese removal exist. Both wells have combined radium above the MCL and Well No. 10 has gross alpha above the MCL. This facility also has an HMO feed system for radium removal.

This facility is one (1) of the four (4) site alternatives investigated for the future WTP No. 5. During the site visits and treatment alternative workshop, City operation staff brought up the idea of integrating the proposed WTP No. 5 into the existing WTP No. 3. The City already owns this property, and when considering the aging existing infrastructure and operations staff request to limit the number of facilities to maintain and operate, this became a viable alternative to investigate in this preliminary design report. WTP No. 5 would initially be a standalone facility with a capacity of 3,000 gpm. Once WTP No. 3 needs replacement, the new WTP No. 3 would be an addition to WTP No. 5, bringing the total capacity up to 5,000 gpm.

2.2.2.3 WTP No. 4

WTP No. 4 serves the northwestern part of the City and is located within Alden Park near the crossing of Highway 169 and Excelsior Boulevard. This facility gets water from Wells No. 12 and No. 13. Both wells have historical data indicating iron, manganese, combined radium, and gross alpha results all above the SMCLs and MCLs. Raw water manganese levels are just above the SMCL, so no additional treatment measures for manganese removal exist. Like WTP No. 3, No. 4 has an HMO feed system for radium removal.

2.2.2.4 WTP No. 6

The newest addition to Edina's water treatment system is WTP No. 6. The City selected a local grocery store's existing parking garage as the site for this facility. According to the City's website, Edina selected the site because, the Park, Police, and Public Works Departments previously used the space for maintenance and storage, so the City endured no additional cost to procure the property, and the site is centrally located to the City's northern water wells. This includes Wells No. 2, No. 7 and No. 15, with plans to bring Well No. 9 back online in the near future. Wells No. 2, No. 9, and No. 15 have historical iron concentrations above the SMCL. Manganese concentrations are at or above the SMCL for all four wells. Well No. 9 has combined radium and gross alpha above the MCLs.

The facility includes air-stripping towers to remove vinyl chloride from Well No. 7. The vinyl chloride entered the aquifer and created a chemical plume that migrated into Edina from St. Louis Park. The City postponed the connection of Well No. 9 because the facility's effluent piping cannot handle the full original design capacity, which was approximately 4,000 gpm with all four (4) wells operational. Plant operators currently limit the peak production to approximately 2,500 gpm, because at that capacity, the distribution pressure is over 115 psi.

2.2.3 Existing Chemical Feed Systems

As identified previously, chemical feed systems used throughout the City's existing treatment system include chlorine for oxidation and disinfection, fluoride for public health and wellness, and an orthophosphate / polyphosphate blend to inhibit pipe corrosion. Additionally, WTPs No. 3 and No. 4 include HMO feed for radium removal. The following sections discuss the chemical feed systems present throughout the existing facilities.

2.2.3.1 *Chlorine*

The City of Edina currently utilizes gas chlorine as the primary and secondary disinfectant for the water system. For filtered wells, pre-chlorine feed is located at the well sites, so residence times vary between wells depending on the distance a well is from the treatment facility. Chlorine feed for unfiltered wells is also located at each well site. The treatment facilities have post-chlorine feed equipment, but the City does not currently use it. All chlorine feed occurs upstream of the filters.

Prior to the summer of 2016, the City operated at breakpoint chlorination for disinfection. The City now uses chloramination as the disinfection strategy, with a goal of 2.0 mg/L total chlorine residual leaving each facility. With the ammonia levels present in the raw water for the existing treated wells, the City does not need supplemental ammonia addition to provide adequate chloramines to inhibit microbiological growth.

The amount of online and offline cylinders varies for each facility, but in general, each online cylinder includes a gas chlorinator, manual feeder and chlorine ejector. In some circumstances, an automatic switchover manifolds two (2) or three (3) cylinders together to reduce the cylinder change frequency. Regardless, the operator manually adjusts the chlorine dosage depending on flow. This method of operation is common but labor intensive if well operation varies. Current technology and instrumentation systems would provide a flow paced chemical feed to automate this process as long as the facility includes a meter for each raw water source and the chlorine feed system includes an automatic chlorine feed controller.

The current method of chlorine storage is not up to recent regulatory requirements, but the City is exempt because installation occurred before these requirements existed. If the City makes any major modifications to their chlorine feed systems at existing facilities, the following would be required:

1. Ventilation improvements to achieve 1 air exchange per minute;
2. Chlorine leak detectors;
3. Automatic shutoff valves;
4. Exterior warning lights, and;
5. Separate storage areas for each pair of cylinders.

Actual chlorine storage requirements for the proposed facility will incorporate recommendations of the City fire marshal and MDH reviewers. These agencies will approve the final design.

<u>2.2.3.2 *Fluoride*</u>

Fluoride serves to reduce tooth decay. Inadequate levels of fluoride in water can result in an increased number of cavities in the population served, while excessive amount of fluoride can mottle tooth enamel, usually producing a yellowish color.

Fluoride addition to the system occurs either after filtration within the treatment facilities or at the well sites for unfiltered wells. The City doses fluoride in the form of fluorosilicic acid. The size of the system varies between the existing facilities based on treatment capacity, but operators adjust the feed rate to maintain a fluoride residual ranging from 0.5 to 0.9 mg/L, with a 0.7 mg/L target.

Fluorosilicic acid is extremely corrosive. For WTPs No. 2 and No. 4, the storage tanks are not within a separate containment area. WTPs No. 3 and No. 6 include containment systems that separate the fluorosilicic acid from other chemicals in the room. For the proposed facility, MDH will require that chemical containment be in accordance with the guidance provided in Ten States Standards. These standards urge the designer to provide a separate room for fluorosilicic acid storage and feed.

2.2.3.3 *Hydrous Manganese Oxide*

The City feeds HMO upstream of the pressure filters in WTPs No. 3 and No. 4. The pre-formed HMO particles adsorb the radium, and then the filters catch and remove the radium during backwash. Edina purchases a product called TonkaZorb™ by Tonka Water that comes as a pre-formed 3% HMO solution. This pre-mixed and pre-formed solution eliminates the need for operators to mix the solution onsite, which is another method of creating HMO. The storage tank has a mixer mounted to the top that keeps the HMO particles uniformly suspended in solution.

HMO is the same chemical as the coating on manganese greensand filter media, creating the potential for adsorption of radium to the filter media. As a result, the filter media may become radioactive. The Project Team suspects that designers of the other facilities with high radium concentrations did not install greensand filtration media for this reason.

2.2.3.4 *Phosphate*

Water systems use an orthophosphate/polyphosphate blend to inhibit corrosion of iron pipe and other metals in the distribution system and sequester iron and manganese. The City currently doses a 50/50 blend of orthophosphate and polyphosphate to provide corrosion control. Phosphate addition to the system occurs either after filtration within the treatment facilities or at the well sites for unfiltered wells. The product the City uses is Carus™ 8500, which is a liquid concentrate of a broad spectrum of phosphates for better sequestration and corrosion control, according to the products data sheet.

2.3 Distribution System and Storage Capacity

Edina's distribution system includes approximately 200 miles of water main, five (5) finished water storage structures, and various interconnections with neighboring cities for emergency use. The following sections describe the existing distribution system and storage capacity in more detail.

2.3.1 Distribution System

The City's distribution system totals approximately 200 miles of water main ranging from four (4) to sixteen (16) inches in diameter. In general, the distribution system is well looped throughout the City, but a few un-looped areas still exist. The City continuously pursues maintenance of old, unlined cast-iron mains and it is typically in the form of full pipe replacement or restoration by pipe lining. Providing the water main looping also ensures adequate water supply for fire flow.

When looking at areas of the distribution system near the proposed facility, the Southdale area includes a large network of 12 inch trunk water main that provides adequate flow during average daily, maximum daily, and emergency demand situations. As part of the Water System Demand and Capacity Analysis conducted in 2013, the City analyzed the impacts of pumping Wells No. 5 and No. 18 into the distribution system. The analysis completed indicated that the two wells provide over 50% of the water during maximum demand scenarios for the southeastern portion of the City, including the highly commercial Southdale area, and generally bounded by Highway 62 to the north, Xerxes Ave to the east, West 77th St to the south, and Highway 100 to the west.

The Edina System includes areas with average day pressures ranging from 40 psi up to over 100 psi. In general, the southern third of the City has high pressures above 90 psi in the highly commercial areas.

In addition to the Edina water system, the City also operates the Morningside system in the northeast corner of the City that gets its water from the City of Minneapolis where it undergoes ultrafiltration, lime softening and multiple chemical treatments prior to reaching the distribution system. Edina's Utility Department is responsible for maintaining the system piping in the Morningside system service area.

2.3.2 Finished Water Storage

The City has five (5) finished water storage structures located throughout the distribution system. The Dublin Reservoir and the Gleason Road Tower are located in the southwest quadrant of the City, the Community Center Tower is directly adjacent to WTP No. 2, the Van Valkenburg is near WTP No. 4, and the Southdale Tower is by Well No. 5.

Table 2.8 summarizes the type of structure and storage capacity for each of the existing storage facilities in Edina. Conversations with City operations staff identified that the tower levels control well operation.

Table 2.8 Edina Storage Structure Summary

Storage Structure Name	Year Constructed	Type	Storage Capacity (MG)
Dublin Reservoir	1960	Ground	4.0
Gleason Road Tower	1970	Elevated	1.0
Community Center Tower	1955	Elevated	0.5
Van Valkenburg Tower	1989	Elevated	1.0
Southdale Tower	1956	Elevated	0.5
		Total	**7.0**

Source: 2008 Edina Comprehensive Plan

2.3.3 Inter-Community Water Service Agreements

According to the 2008 Edina Comprehensive Plan, the cities of Bloomington, Eden Prairie, Minneapolis, and St. Louis Park serve portions of small residential and commercial areas within city limits. In addition, the City has the ability to interconnect with Bloomington, Eden Prairie, Hopkins, and Minneapolis in the event of an emergency. Edina can both send and receive water from these municipalities.

2.4 Back-Up Power

The City of Edina strives to provide its water customers with a reliable water system that delivers a safe supply of water during emergencies. Part of this commitment includes back-up power generation to keep WTP's and wells operational during outages.

All four (4) existing WTP's have onsite generators with automatic transfer switches. Wells located within the facilities also have back-up power through the onsite generators. This includes Well No. 6 within WTP No. 2, Wells No. 10 and No. 11 within WTP No. 3, and Wells No. 12 and No. 13 within WTP No. 4. Well No. 2 also has an onsite generator with an automatic transfer switch. The City provides back-up power to the remaining well sites with two (2) portable generators. Generator hookups at each of these sites allows the City to get wells back online within hours, or sooner, depending on generator transport time. In addition, the Dublin Reservoir high service booster pumps have back-up power through an onsite generator with automatic transfer. The design of WTP No. 5 includes back-up power.

2.5 Previous Study Efforts

The City of Edina has previously completed various studies and planning documents to address specific components of the need for Water Treatment Plant No. 5. The Project Team reviewed and summarized relevant information from these studies in the sections to follow. This WTP No. 5 Preliminary Design Report considers and incorporates these recommendations as necessary.

2.5.1 Feasibility Study for Water Treatment Plants No. 5 and No. 6

In 2007, the City's consultant completed a feasibility study for Water Treatment Plants No. 5 and No. 6. The report evaluated various treatment options to meet the treatment goals, developed a conceptual design, and provided an opinion of probable cost for the two facilities.

The feasibility study assumed WTP No. 5 to have a treatment capacity of 2,000 gpm, with raw water coming from existing Wells No. 5 and No. 18 and location of the site at the Southdale Site. Upon completion of the alternative treatment option and financial analyses, recommendations included that WTP No. 5 use HMO with iron and manganese removal, pressure filtration, and greensand filtration media. The anticipated construction cost at that time was $6,083,000.

In addition to the recommended treatment approach to remove radium, the consultant evaluated lime softening, ion exchange, reverse osmosis, and a proprietary system from Water Remediation Technology (WRT) using patented Z-88 media.

The conceptual design included two, 10-foot diameter, 40-foot long pressure filters split into four cells, a gas chlorination feed system, and chemical feed systems for premixed HMO, phosphate, and fluoride. The plant also included a backwash reclaim system with a 450,000-gallon basin that is capable of holding 1.5 backwashes from both filters.

2.5.2 City of Edina Comprehensive Plan

The City last updated their Comprehensive Plan in 2008. The water supply portion of the plan details the current water supply and distribution system, the water demand trends and challenges, the City's goals and policies related to water supply, and the implementation plans for reaching those goals.

At that time, the City operated eighteen (18) wells, with eight (8) treated by oxidation and filtration, seven (7) that provided unfiltered water during periods of high demand, and three (3) inactive due to having contaminants that exceeded the current EPA Maximum Contaminant Level (MCL). Four Water Treatment Plants treated various wells and five storage facilities provided 2.0 million gallons of usable storage.

The average daily demand in 2006 was 7.62 MGD, with variation over the previous five-year period of 6.78 to 8.16 MGD. The maximum daily demand varied from 14.5 to 21.8 MGD. The peaking factor between maximum and average daily demand ranged from 1.9 to 3.0.

The future demand projections through 2030 indicated no additional need for storage in the system. Based on a peaking factor of 3.0, the City would need two additional wells to meet the Metropolitan Council projections at that time. Additional projections considered ultimate build out of the City and found that if the City can reduce the peaking factor to 2.75 with water conservation and major capital improvements, they may need an additional five new wells on top of the proposed wells 20 and 21.

The goal of the water supply plan was to implement plans and policies that would continue to provide water customers with safe, high quality potable water, to ensure sustainability of the City's water system through preservation and conservation, and to maintain a reliable water system that can provide safe drinking water during emergencies.

Plans for implementation of these goals at that time included the addition of two filter plants to reduce the amount of unfiltered water in the system during peak demands and the replacement or lining of old, unlined water mains. To promote water conservation, the City adopted a tiered inclining block rate structure and implemented other water conservation measures identified in a separate Water Supply Plan.

2.5.3 Water System Demand and Capacity Analysis

In 2013, the City's consultant completed a water system demand and capacity analysis for the addition of Water Treatment Plant No. 5 into the City's system. The analysis assumed the facility would be located at the Southdale Site and would have a 2,000 gpm capacity.

Part of the analysis included review of daily pumping records and determined that with current treatment capacity, the City utilizes unfiltered wells 120 days of the year. Looking at the population projections for the City through 2030, if the City does not add WTP No. 5, the calendar days of unfiltered water in the system would expand to 140 days of the year. With the addition of WTP No. 5, the City would reduce the unfiltered water days to 5 days per year.

The analysis concluded that the WTP addition increases pressures in the vicinity by around 1-2 psi with the additional pumping at that location during average day demand simulations. During this average day analysis, the WTP No. 5 production assumed was 2,000 gpm with water towers at 2 feet below overflow, with WTP No. 6 producing about 2600 gpm, and Well No. 4 on at WTP No. 2.

The analysis also examined the effect of the WTP addition near the Southdale Tower for peak demands, and they found that due to the distribution of water demand in the model, the Southdale Tower lags other towers, so the increased pumping at the Southdale Site actually helps balance the tower levels.

The report also recommended that during preliminary design, the design engineer should consider the costs and benefits of increasing WTP No. 5 capacity to accommodate an additional future well.

CHAPTER 3 PLANNING HORIZON AND WATER DEMAND PROJECTIONS

3.1 Planning Horizon

The ultimate goal of this Preliminary Design Report (PDR) is to inform City staff on the potential long term outcomes various treatment decisions may result in. There are many ways to approach the ultimate goal of additional water treatment for the City of Edina. All of these approaches will have different cost, operational, and planning level implications. It is important that the Project Team take a long term view of this decision to understand the impact they may have on the City of Edina. For the purposes of this study, the Project Team reviewed water demand and population projections through 2040. We chose the year 2040 because it aligns with many of the comprehensive planning efforts currently underway within the City of Edina and the Metropolitan Council established 2040 as an appropriate planning level timeline for use when making large capital investments such as a water treatment plant.

In addition to the 2040 population projections, the Project Team utilized 30-year capital, and operation and maintenance (O&M) cost projections to understand the long term impacts of various treatment alternatives better. Chapter 7 reviews the analysis of these types of cost implications. In many cases, there is a significant difference in chemical and capital costs for equivalent treatment technologies. It is of significant importance that the City of Edina fully understand the capabilities of each alternative and long term cost implications of these decisions, as they will have impacts on the financial needs of the City.

3.2 Historical Population and Growth Projections

The City of Edina experienced limited growth from 2000 through 2010, with a slight increase in growth from 2010 through 2016. According to the Metropolitan Council's Thrive 2040 population forecasts, Edina is likely to experience moderate growth through 2040. Edina's 2000 population was 47,425 and its projected 2040 population is 54,400. Table 3.1 provides a complete summary of the historical and projected populations for Edina. In addition to these population projections, Edina is currently conducting additional water supply analysis that provides a more detailed review of the potential water demands and commercial development in various portions of the City. These projections may further inform policy makers of the overall water supply needs. For the purposes of this report, the Project Team utilized the Metropolitan Council population projections and the historical water use data to determine the adequacy of the water supply system for Edina.

Table 3.1 Edina Population Projections

	YEAR	Edina Population	Edina Growth Rate
HISTORICAL DATA	**2000**	**47,425**	
	2001	47,477	0.11%
	2002	47,528	0.11%
	2003	47,580	0.11%
	2004	47,631	0.11%
	2005	**47,683**	**0.11%**
	2006	47,734	0.11%
	2007	47,786	0.11%
	2008	47,837	0.11%
	2009	47,889	0.11%
	2010	**47,940**	**0.11%**
	2011	48,380	0.92%
	2012	48,819	0.91%
	2013	49,259	0.90%
	2014	49,698	0.89%
	2015	**50,138**	**0.88%**
	2016	50,350	0.42%
PROJECTED DATA	2017	50,563	0.42%
	2018	50,775	0.42%
	2019	50,988	0.42%
	2020	**51,200**	**0.42%**
	2025	**52,550**	**0.52%**
	2030	**53,900**	**0.50%**
	2035	**54,150**	**0.09%**
	2040	**54,400**	**0.09%**

3.3 Historical Water Demand and Future Projections

The Project Team conducted an in depth review of Edina's historical pumping records to better understand the previous year experienced water demands. Daily, monthly, and annual usage data was available to conduct the review. Since 2000, the average gallon per capita per day water usage has been approximately 147.12 gpcd and has shown to be decreasing over the past 5 to 10 years. The future water use projections use a 158.62 gpcd average day water demand as a conservative estimation of average occurrences in the past 10 years. Based on this assumption, the estimated average day demand for the City of Edina will be approximately 8.63 MGD in 2040.

The Project Team also projects an approximate maximum day demand of 25.89 MGD for 2040. The maximum day demand is critical to the long term planning of a utility because it determines the highest demand likely experienced by the water utility. In 2016, the City of Edina experienced a peaking factor of approximately 2.15. Previous Edina water demand analysis utilized a peaking factor of 3.0. A 3.0 peaking factor is reasonable for the City to use for future water demand planning for the water utility, and is in the range of peaking factor experienced by other communities throughout the region. Table 3.2 below summarizes the historical and projected water demands for the City of Edina.

Table 3.2 Edina Water Demand Projections

YEAR	Edina Population	Edina Growth Rate	Total Water Pumped (MG)	Average Day Demand (MGD)	Maximum Day Demand (MGD)	Peaking Factor	Total Annual Gallons /Capita /Day (gpcd)
2000	**47,425**		**2,675**	**7.33**	**21.98**	**3.00**	**154.52**
2001	47,477	0.11%	2,713	7.43	22.30	3.00	156.57
2002	47,528	0.11%	2,473	6.78	20.33	3.00	142.57
2003	47,580	0.11%	2,978	8.16	24.48	3.00	171.50
2004	47,631	0.11%	2,649	7.26	21.78	3.00	152.39
2005	**47,683**	**0.11%**	**2,583**	**7.08**	**21.23**	**3.00**	**148.42**
2006	47,734	0.11%	2,798	7.67	23.00	3.00	160.61
2007	47,786	0.11%	2,691	7.37	22.11	3.00	154.26
2008	47,837	0.11%	2,615	7.16	21.49	3.00	149.74
2009	47,889	0.11%	2,773	7.60	18.75	2.47	158.62
2010	**47,940**	**0.11%**	**2,478**	**6.79**	**13.13**	**1.93**	**141.64**
2011	48,380	0.92%	2,522	6.91	14.12	2.04	142.81
2012	48,819	0.91%	2,779	7.61	17.08	2.24	155.93
2013	49,259	0.90%	2,428	6.65	15.78	2.37	135.03
2014	49,698	0.89%	2,368	6.49	15.45	2.38	130.56
2015	**50,138**	**0.88%**	**2,302**	**6.31**	**12.70**	**2.01**	**125.81**
2016	50,350	0.42%	2,207	6.05	12.99	2.15	120.10
2017	50,563	0.42%	2,927	8.02	24.06	3.00	158.62
2018	50,775	0.42%	2,940	8.05	24.16	3.00	158.62
2019	50,988	0.42%	2,952	8.09	24.26	3.00	158.62
2020	**51,200**	**0.42%**	**2,964**	**8.12**	**24.36**	**3.00**	**158.62**
2025	**52,550**	**0.52%**	**3,042**	**8.34**	**25.01**	**3.00**	**158.62**
2030	**53,900**	**0.50%**	**3,121**	**8.55**	**25.65**	**3.00**	**158.62**
2035	**54,150**	**0.09%**	**3,135**	**8.59**	**25.77**	**3.00**	**158.62**
2040	**54,400**	**0.09%**	**3,150**	**8.63**	**25.89**	**3.00**	**158.62**

Peaking factors vary on an annual basis for a number of reasons. The 3.0 peaking factor applied from 2000 through 2008 and 2017 through 2040 estimates the anticipated past and future maximum day demands. Industrial or commercial users typically reduce a utility's peaking factor as they are less weather dependent and tend to use consistent volumes of water. Weather conditions typically have the most significant influence on peaking factor. Warm, dry years typically have higher peaking factors as the result of increased seasonal irrigation, while wet years typically have lower peaking factors.

Figure 3.1 below illustrates the daily water use trend for 2016. This trend varies from year to year. Some years' experience more extreme peaks than others. Included in Figure 3.1 are the current capacities of each WTP. The graph illustrates these capacities as a cumulative effect to illustrate to what degree they are able to meet the annual maximum day demands experienced by Edina. As noted previously, WTP No. 6 capacity is currently limited to 3.6 MGD due to distribution system sizing. As illustrated below, WTP No. 5 would likely provide enough treatment capacity to meet the maximum day demands seen by Edina in 2016. The graph also indicates that even with full capacity of WTP No. 6, the system does not meet maximum day demands for this given year. Addition of WTP No. 5 prior to increasing WTP No. 6 capacity appears to meet this demand. Note that the firm well and well capacities represented in the figure do not include the 1,000 gpm additional capacity planned for future Well No. 21, anticipated for inclusion in the treated capacity of WTP No. 5.

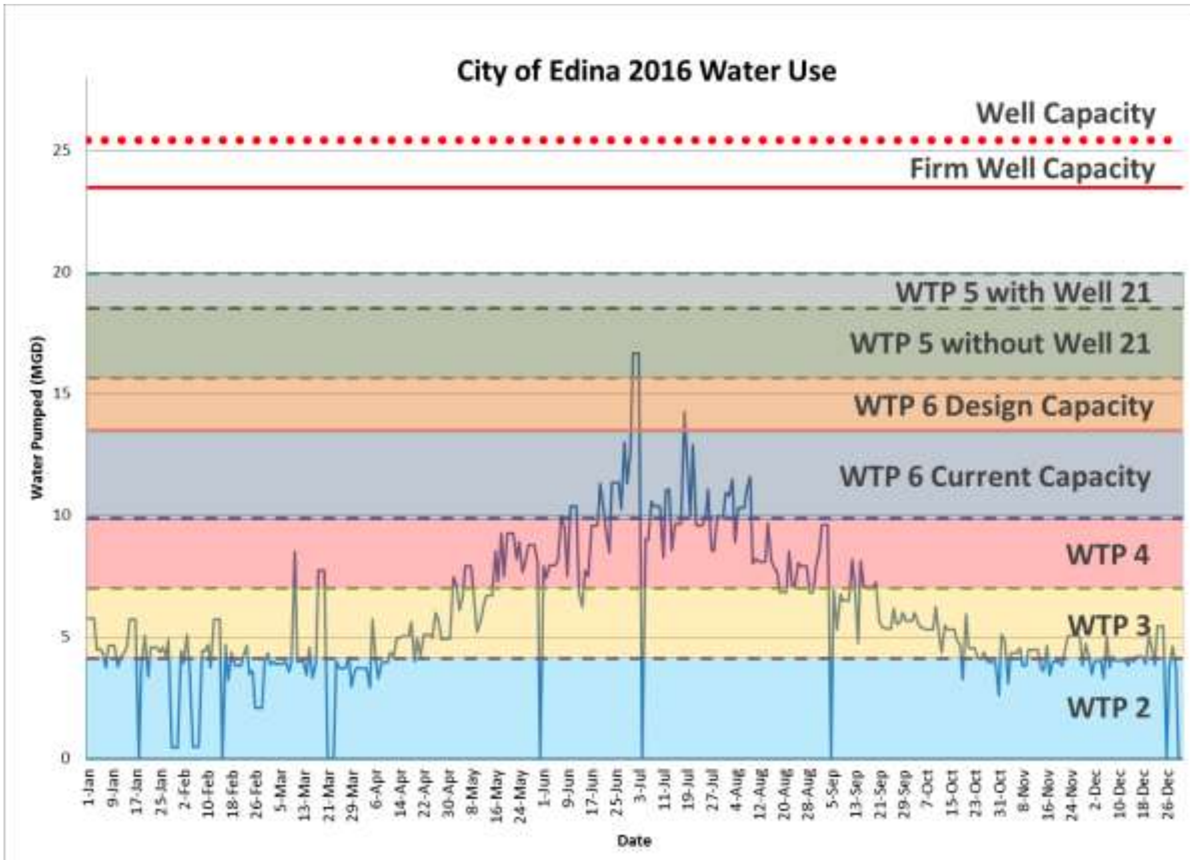

Figure 3.1 2016 Daily Water Use Analysis

Figure 3.2 graphically illustrates the use of the data in Table 3.2 and a similar cumulative impact approach as Figure 3.1 applied to the future water demand projections. Note that the Project Team gathered historical population for 2000 from the US Census Bureau's database and confirmed the data within Edina's 2008 Comprehensive Plan update. Retrieval of historical populations for 2010 and 2015 came from the US Census Bureau quick facts online database. As noted previously, the Metropolitan Council "Thrive MSP 2040" update as of January 1, 2017 provided population estimates for 2020 and beyond.

This graph shows that the current firm well capacity is below the projected maximum day demand through 2040, indicating that additional wells may be necessary in the future to meet system demands. The firm well and well capacities represented in the figure do not include the 1,000 gpm additional capacity planned for future Well No. 21, anticipated for inclusion in the treated capacity of WTP No. 5. Recall that this projection uses a total annual gallon per capita per day of 158.62 gpcd for calculation of average day demand based on population projections and a 3.0 peaking factor for determining maximum day demands.

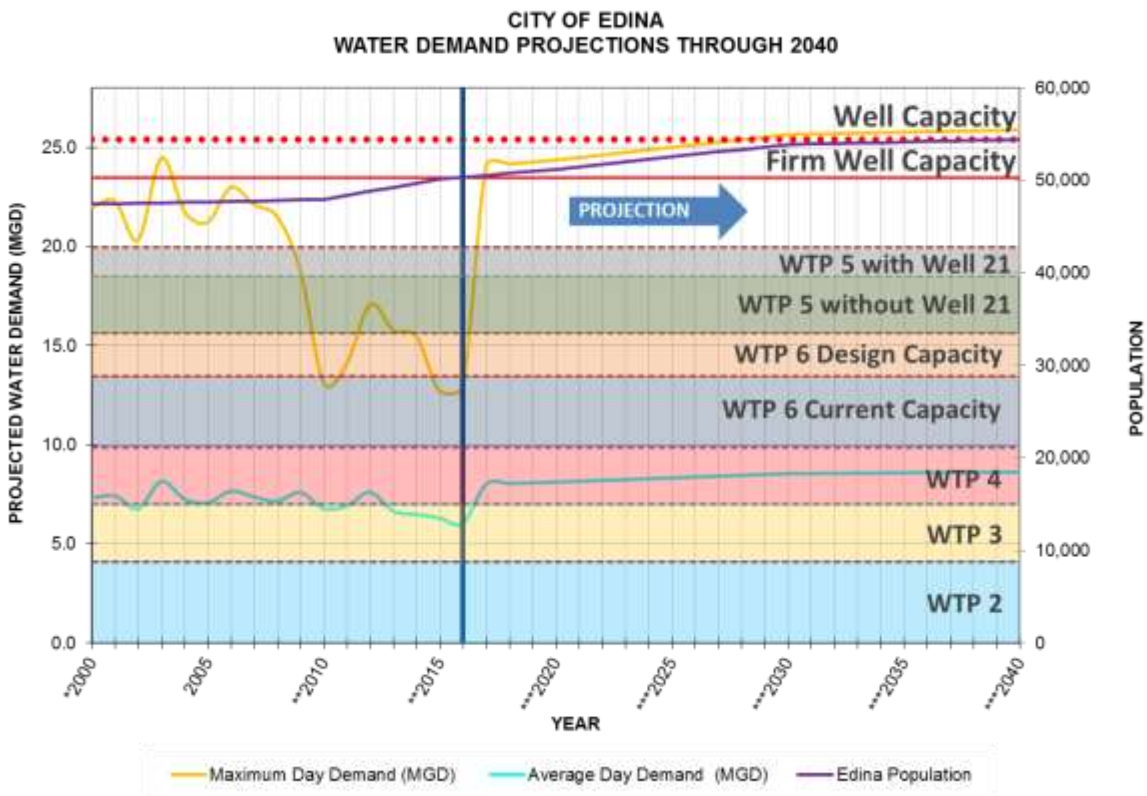

Figure 3.2 Projected Water Use Analysis

The review of the historical water use and future projections indicates that WTP No. 5 will likely provide the City of Edina with additional treatment capacity that will further eliminate the need to utilize the unfiltered wells during peak day scenarios in the summer months. WTP No. 5 will also provide additional treatment redundancy to allow City staff more operational flexibility during maintenance or emergencies.

In addition to the filtered capacity of Edina's water, the Project Team evaluated firm well capacity. The firm capacity was determined by removing on of the 1,000 gpm wells from the system, representing the largest well for the City of Edina. Additional firm capacity assumptions include limiting the WTP No. 6 plant production from the available wells (Well No. 2, No. 7, and No. 15) to 2,500 gpm due to current distribution system limitations. The reconstruction of Well No. 9 is currently underway, so the firm capacity assumes the well is active.

As illustrated in Figure 3.2 Edina's current firm well capacity is 23.47 MGD, which is below the projected maximum day demands in 2040 of 25.89 MGD. In fact, it may likely be below current maximum day demands in the event that Edina experiences a warm summer. It is prudent for Edina to investigate the addition of another well. As discussed in the remaining chapters, this well would likely be Well No. 21, which would provide WTP No. 5 with the final 1,000 gpm of planned capacity. If Well No. 21 produces 1,000 gpm, the need for a well in addition to Well No. 21 is likely to provide a firm well capacity equivalent to the projected 2040 maximum Day demand.

3.4 Water Storage Volume Considerations

When evaluating the water demands of a community, the storage capability of the distribution system is critical to this evaluation. Storage facilities typically provide:

1. Equalization Storage – to meet hourly system water demands exceeding supply pumping capacity
2. Fire Protection Storage – to meet the demands of fire fighting
3. Emergency Storage – to provide water reserves for contingencies such as system failures, power outages, and other emergencies

3.4.1 Equalization Storage

A primary function of storage facilities within the distribution system is equalization. Water demand in most utilities varies significantly throughout the course of the day, and treatment facilities tend to operate most efficiently at a constant rate. In order to meet these variations in demand, the water utility can vary the source, vary the pumping rate, or provide equalization through the process of filling and draining storage reservoirs within the distribution system. Equalization storage enables the treatment facility operation at a predetermined rate, depending on the utility's preference. Additionally, equalization storage is generally less expensive than increased capacities of high service pumps beyond what is required to meet the maximum day demand (MDD). Consequently, it is desirable to size the source and pumping facilities to serve the water needs up to the MDD and provide equalization storage for meeting peak instantaneous water demands (such as the peak hour or peak two hour demands). The amount of equalization storage required is a function of the source, pumping capacity, distribution piping capacity, and system demand characteristics.

The fraction of water production that must be stored during a maximum day as equalization storage depends on the individual utility, and utility's operational pumping practices.

Options for operational pumping modes include the following:

1. Operate at a constant rate to simplify operation and reduce demand charges;
2. Adjust flow to roughly match demand and minimize use of storage;
3. Pump during off peak hours to take advantage of reduced energy costs; and
4. Operate with a reasonable number of starts per unit time.

Table 3.3 provides typical values for the equalization storage needed as a fraction of the maximum daily demand for the various operational pumping modes. The values range from a low of zero for variable speed pumping, to a high of 0.50 for off-peak pumping. The upper range of values are typical for those systems with higher peak demands, while the lower values are typical for those systems with a flatter daily demand curve.

Table 3.3 Typical Equalization Volume Fractions for Various Operational Pumping Modes

Type of Operation	**Equalization volume needed as a fraction of maximum daily demand**
Constant Pumping	0.10 - 0.25
Follow Demand (Constant)	0.05 - 0.15
Off Peak Pumping	0.25 - 0.50
Variable Speed Pumping	0

Determining the volume of required equalization storage can equalize the demand variations with the pumping sequence that occurs during the MDD. Based on experience with water distribution systems similar to Edina, the Project Team assumed a volume of equalization storage of 15 to 20 percent of the MDD. Additionally, storage tanks/reservoirs should provide equalization storage within the top 50 percent of the tank, enabling operators to have an operating range that maintains adequate system pressures and adequate fire and emergency storage within the distribution system. As discussed in section 3.3, the Project Team projects Edina's 2040 MDD to be 25.89 MGD. The required 20-percent equalization storage at this maximum day demand is 5.17 MG.

3.4.2 Fire Flow Requirements and Storage Recommendations

In addition to the maximum day demand and equalization storage, Edina should consider the fire flow requirements of their distribution system users. Public water systems consider fire protection a secondary purpose, and is an issue typically addressed at the policy level within each community. No laws or legal implications exist if a water system does not provide water for fire protection. The decision to provide water for fire protection requires careful consideration of fire flow requirements when sizing pipelines, pumps, and storage tanks because it results in higher capital and O&M costs than a distribution system that provides only potable water to residents. Provisions for fire flows also provide a valuable public service, however, by reducing the potential loss of human life and property, and improving insurance ratings within the community.

The Pilot Team used the following recommended general guidelines in evaluating the capacity of future system improvements:

- Single family residential = 1,500 gpm for a two-hour duration
- Apartment Residential = 2,000 gpm for a two-hour duration
- Educational Building = 1,500 gpm for a two-hour duration
- Commercial/Industrial = range of 2,000 to 3,000 gpm for two to three-hour duration

Commercial and industrial buildings assume presence of designed and installed sprinkler systems. A safe (conservative) fire demand for a commercial/industrial building is 1,500 gpm on an external hydrant plus an additional 1,500 gpm load from the building sprinkler system. The building sprinkler system load may be less than 1,500 gpm depending on the uses and contents of the building. An important component in providing adequate fire protection is retaining sufficient fire storage volume within the distribution system. Thus, the maximum recommended fire load for a commercial/industrial building is 3,000 gpm for a three-hour duration or approximately 500,000 gallons of storage.

3.4.3 Emergency Storage

Emergency storage provides water for domestic consumption during events such as transmission or distribution main failures, raw water contamination events, extended power outages, failure of raw water transmission facilities, failure of WTP facilities (including high service pumps), or a natural disaster. There are no existing formulas for determining the amount of emergency storage required by a utility. Rather, the amount of emergency storage is a policy decision based on an assessment of the perceived vulnerability of the utility's water supply, risk of failures, and the desired degree of system reliability. If a utility has redundant sources and treatment facilities with auxiliary power, or power supplied from multiple sources, the need for emergency storage may be relatively small. This is the case for Edina. However, enough emergency storage should be available to handle a catastrophic pipe break that cannot be isolated easily.

3.4.4 Water Storage Volume Evaluation

Currently Edina operates four (4) water towers and one (1) ground Storage reservoir.

Table 3.4 Edina Water Storage Tanks

Storage Structure Name	Year Constructed	Type	Storage Capacity (MG)
Dublin Reservoir	1960	Ground	4.0
Gleason Road Tower	1970	Elevated	1.0
Community Center Tower	1955	Elevated	0.5
Van Valkenburg Tower	1989	Elevated	1.0
Southdale Tower	1956	Elevated	0.5
		Total	**7.0**

Engineers base the total volume of required storage on a combination of equalization, fire, and emergency storage. Some engineers use the sum of the three types of storage, while others base designs on the sum of equalization storage and the larger of either the fire protection storage or emergency storage. The logic in such cases is that the fire is not likely to occur at the same time as a critical pipe break or power outage. For the purposes of this analysis, the Project Team assumed that the total storage needed is equalization and the greater of either fire flow storage or emergency storage. Given the developed demographic of Edina and the current buildings within the city, fire storage controls for this analysis. Table 3.5 shows a preliminary assessment of storage volumes required based on an initial water demand and fire flow assessment of the water system. Based on this analysis, Edina has a prudent amount of storage volume within the City. The Project Team suggests that the City conduct a more specific pressure zone evaluation to analyze zone specific storage requirements and fire flow demands.

Table 3.5 Water Storage Volume Requirements

Equalization Storage	Based on 20 percent of the 2040 MDD for Edina.	5.17 MG
Fire Storage	Based on 3,000 gpm fire demand for 3 hr duration	0.5 MG
	Total	**5.67 MG**

Additionally, the Project Team suggests evaluation of the Dublin ground storage reservoir pumping capacity to confirm that it can meet the maximum hourly demand for its service area. If it is unable to meet the projected maximum hourly demand, we suggest removal of the 4.0 MG from the total 7.0 MG total available storage, leaving 3.0 MG remaining available storage.

CHAPTER 4 TREATMENT PROCESS OBJECTIVES

4.1 Standard Engineering Design Criteria

The City of Edina will measure the performance of WTP No. 5 against established criteria and drinking water regulations. The City is planning to construct a new WTP to continue to provide an abundant and reliable supply of safe, quality water to system customers. Primary objectives include conformance with standard engineering design criteria, compliance with existing and anticipated drinking water regulations and the ability to achieve the specific established target treatment goals.

4.1.1 MDH Standards (Ten State Standards)

The Minnesota Department of Health (MDH) establishes standards, formally and informally, through its engineering plan review process. In Minnesota, water system design follows the guidelines of the MDH and the Great Lakes–Upper Mississippi River Board of State and Provincial Public Health and Environmental Managers Standards for Water Works[1], or the Ten States Standards. Ten States Standards primarily consists of Policy Statements, Interim Standards and Recommended Standards for the design of water systems.

The Policy Statements address innovative treatment processes for which sufficient data does not yet exist to establish specific recommended design parameters. The Policy Statements also recommend approaches and considerations for addressing specific issues that may not develop into standards. The seven (7) Policy Statements provided in the most recent (2012) edition of Ten State Standards are as follows:

- Pre-Engineered Water Treatment Plants;
- Automated/Unattended Operation of Surface Water Treatment Plants;
- Bag and Cartridge Filters for Public Water Supplies;
- Ultraviolet Light for Treatment of Public Water Supplies;
- Infrastructure Security for Public Water Supplies;
- Arsenic Removal; and
- Design Considerations for the Optimization of Rapid Rate Filtration at Surface Water Treatment Plants.

The Interim Standards provide design criteria currently used for new process system design, but the data are limited and insufficient for recognition as a recommended standard. Currently, there are two (2) interim standards: 1) Use of Chloramine Disinfectant for Public Water Supplies and 2) Membrane Technologies for Public Water Supplies.

Proven technology developed the Recommended Standards, with the intent to serve as the guide for the design of public water systems. The Recommended Standards comprehensively address aspects of the following nine (9) primary areas of public water supplies:

- Submission of Plans;
- General Design Considerations;
- Source Development;
- Treatment;
- Chemical Application;
- Pumping Facilities;
- Finished Water Storage;
- Distribution System Piping and Appurtenances; and
- Waste Residuals.

4.1.2 Standard Industry Practices and Professional Judgement

Although Ten States Standards provides recommended guidelines for many aspects of drinking water systems, the standards are insufficient to address every aspect of detailed water system design comprehensively. Raw water quality characteristics and the variability of raw water quality are unique to each treatment facility. The performance of treatment processes may vary significantly depending on application and integration with other treatment processes. Equipment manufacturers offer competing products that, although similar, offer different size considerations, ancillary equipment and treatment characteristics. In addition, preferences of the Edina staff will influence specific aspects of system design. Where innovative or alternative technologies are considered and where recommended standards are not available, standard industry practices and best professional judgment of sizing and performance will be determined from manufacturers' data and available performance information from other installations.

4.1.3 Security and Redundancy

The safety of the public water supply to the City of Edina is a vital concern in this planning process. All WTP facilities employ special safety considerations. Ten States Standards identifies that water treatment plant design must comply with all applicable safety code and regulations which include, and may not be limited to, Uniform Building Code, Uniform Fire Code, National Fire Protection Association Standards, and state and federal Occupational Safety and Health Administration (OSHA) standards. Safety considerations include noise protection, confined space entry, personal protective equipment and clothing, safety showers and eyewashes, guardrails, warning signs, smoke detectors, and fire extinguishers.

Ten States Standards recognizes that water systems are vulnerable to intentional acts of vandalism, sabotage, or destruction. A few of the key items related to facility protection identified in the "Policy Statement on Infrastructure Security for Public Water Supplies" include:

1. Incorporate redundancy and enhanced security features in the design to eliminate single points of failure. Incorporate additional protection measures if redundancy is not feasible.
2. Maintain an inventory of critical parts for use in the event that damage or destruction occurs on a critical component.
3. Limit human and vehicle access to the facility through controlled locations only.
4. Secure computer based technologies such as SCADA from unauthorized access or cyber-attacks. Equip all automated control systems with manual overrides to provide the option to operate manually.
5. Encourage the addition of real time water quality monitoring with continuous reporting and alarms to provide early warning of possible intentional contamination events.
6. Design chemical delivery, handling and storage facilities to ensure that chemicals are safe from intentional release.

The site alternatives for the proposed facility are located in highly populated, centralized areas. Safety and security will be a major factor in the preliminary design. The design will incorporate enhanced safety measures to ensure protection of water plant operators and the public.

4.2 Drinking Water Regulations

Congress passed the Safe Drinking Water Act (SDWA) in 1974. Its purpose was to establish a uniform set of regulations and water quality standards for public water systems across the United States. The SDWA focused on identifying substances present in drinking water that had adverse public health effects. The City of Edina is currently required to meet the regulations of the SDWA under the enforcement responsibility of the Minnesota Department of Health (MDH), the Primacy Agency. Minnesota became one of the first (5) states to achieve primacy and to begin regulating public water supply systems at the state level in 1976.

To strengthen the SDWA, especially the regulation setting process, Congress amended most of the 1974 SDWA in 1986. Under the 1986 SDWA Amendments, the number of regulated contaminants increased from 23 to 89. Each standard consisted of a sampling frequency requirement and a maximum contaminant level (MCL). Congress originally mandated the USEPA to establish MCLs for 25 new parameters every three years under the amended 1986 SDWA.[2] Amendments to the SDWA in 1986 included several regulations that directly or indirectly affect the future WTP No. 5.

Congress signed a Reauthorization of the SDWA into law (Public Law 104-182) on August 6, 1996. The law repealed the original mandate established by Congress for the USEPA to regulate 25 new contaminants every three (3) years and replaced it with a new standard-setting process to identify contaminants for future regulation based on their occurrence, the health risk they pose and cost-benefit evaluations.[3] The 1996 SDWA Reauthorization made several additional important changes including: 1) establishing new requirements for selecting contaminants for regulation; 2) mandating the use of sound science; 3) allowing analyses of health risk reductions, costs and benefits; 4) establishing an occurrence database; and 5) evaluating permitting competing risks.

Under the Reauthorization, the USEPA selects at least five (5) new contaminants to consider for regulation every five (5) years with regulations geared toward those imposing the highest health risk. Surface water treatment facilities have been the focus of heightened regulations due to the concerns over microbiological contaminants and disinfection by-products (DBPs). The Reauthorization of the SDWA has provided a review of the original SDWA and a better understanding of the significance of providing regulations that emphasize the importance of maintaining proper disinfection while controlling the formation of DBPs. Recent discussions regarding future drinking water regulations include commercial and industrial chemicals, pesticides, biological toxins, additional disinfection byproducts and waterborne pathogens.

The City of Edina will achieve its goal of providing customers with quality water by complying with the primary drinking water regulations, satisfying secondary drinking water regulations and addressing the water quality issues not specifically addressed by primary or secondary regulations. Primary drinking water regulations control or will control filtration, turbidity, filter backwash, disinfection, DBPs, disinfectant residuals, total coliform bacteria, lead, copper and a long list of additional analytes in the water through MCLs. These regulations protect public health. The secondary drinking water regulations help provide water that is aesthetically and cosmetically pleasing. Secondary drinking water regulations and other considerations also address technical effects, a term coined by the SDWA Advisor, that address such issues as corrosivity.[4]

4.2.1 Primary Drinking Water Standards

Primary drinking water regulations address microbial contaminants, disinfectants and disinfection by-products (DBPs), maximum residual disinfectant levels (MRDLs), inorganic and organic compounds, radionuclides, treatment techniques (TT), maximum contaminant levels (MCLs) and other advisory objectives and parameters. The primary drinking water standards are legally enforceable standards that apply to public water systems. Primary standards protect public health by limiting the levels of contaminants in drinking water.

4.2.1.1 *Lead and Copper Rule*

The 1986 Amendments to the SDWA required USEPA to promulgate drinking water standards for contaminants that impose potential adverse health risks. Lead and copper were specifically listed in the 1986 SDWA amendments for mandatory development of a National Primary Drinking Water Regulation (NPDWR); USEPA responded by promulgating the LCR, which was published in 1991. The stated goal of the LCR is to "minimize lead and copper at users' taps while ensuring that treatment does not cause the system to violate any NPDWR".[2] This goal is intended to be accomplished through the application of corrosion control strategies (i.e. varying pH levels, alkalinity levels and inhibitor utilization). The LCR action levels for lead and copper are 0.015 mg/L and 1.30 mg/L, respectively, in the 90th percentile of samples measured at customer taps.

In November 2016, Edina had copper levels above the copper action level for one sampling site. MDH required the City to double their sampling frequency. The exceedance occurred at a residential sampling site built in the early 1980's, when use of lead containing plumbing pipe and fittings was common. This was the City's first instance of LCR action level exceedance. The first round of samples taken in 2017 resulted in no exceedances and the City will conduct the second round of sampling in October 2017.

The USEPA published the LCR Short-Term Revisions on October 10, 2007. The revisions included changes in both the health effects language and utility's public education requirements. The revisions intended to clarify and enhance implementation of the LCR in the areas of monitoring, treatment, customer awareness, and lead service line replacement. The revisions also aimed to improve compliance with public education requirements.

The USEPA is currently considering Long-Term Revisions to the LCR. Requirements under consideration for modification include sample site selection criteria, sampling procedures, water quality monitoring, continued emphasis on lead service line replacement and consecutive water system requirements.

Another recent related regulation is the Reduction of Lead in Drinking Water Act of 2011. Congress signed the Act into law on January 4, 2011, which became effective on January 4, 2014. Provisions of the federal law revise the SDWA definition of "lead free" for piping, pipe fittings, plumbing fittings and fixtures. The amendment reduces the lead limit from eight percent to 0.25% for brass and bronze. The limit for solder and flux remains at 0.2%. The federal law applies to the wetted surfaces of any product used in a drinking water system. The new requirement requires suppliers, contractors, the engineering community and water utilities to revise specifications for no-lead brass plumbing fittings and components such as curb stops, meters, regulators, check valves, and now fire hydrants. There is ongoing discussion regarding the USEPA's interpretation of the law regarding the inclusion of system fire hydrants.

Although continued compliance with developing regulations will be on-going, the City monitors lead and copper concentrations in the distribution system consistently. Edina adds an orthophosphate/polyphosphate blend to the water to inhibit corrosion and sequester lead and copper and further treatment in addition to this method is not currently required.

4.2.1.2 *Volatile Organic Chemicals Rule (VOC Rule)*

The VOC Rule became effective under the SDWA on January 9, 1989. The VOC Rule established MCLs for eight (8) volatile organic chemicals (VOCs) such as benzene, carbon tetrachloride, vinyl chloride, etc. that are suspected human carcinogens through ingestion. The VOC Rule is part of the Phase I Rules of the SDWA.

Based on review of the most recent water quality analysis by the MDH, all VOCs were determined to be below the Reporting Limit for Wells No. 5 and No. 18 that will feed into the proposed WTP No. 5. Well No. 7 located in the northwestern part of the City has reported levels of vinyl chloride that exceed the regulated MCL. WTP No. 6, the most recent addition to the City's system, treats and removes the vinyl chloride affecting this well by air-stripping towers. The source of vinyl chloride is likely from a chemical plume that entered the aquifer at the land surface from an unknown source in St. Louis Park. WTP No. 6 is typically online continuously to pump water from the impacted wells and stop the migration of the chemical plume.

4.2.1.3 *Phase II/IIb and Phase V Rules*

The Phase II and Phase IIb Rules became effective on July 1, 1991 and January 1, 1993, respectively. Phase II/IIb Rules nearly doubled the number of regulated drinking water contaminants by setting standards for 38 VOCs, synthetic organic chemicals (SOCs) and inorganic chemicals (IOCs). The Rules regulate Thirty-six (36) of the contaminants by MCLs and two (2), acrylamide and epichlorohydrin, by limiting their use in drinking water treatment chemicals.

Although a large number of Phase II/IIb chemicals result from human activity, others occur naturally in water. These contaminants have been shown to either be or are suspected to be carcinogenic through ingestion. Some of the other effects of these contaminants include damage to numerous organs in the body, circulatory system damage, bone damage, nervous system damage and disorders, thyroid damage, and decreased body weight.

PWSs are required to ensure the water they supply meets the MCL for each Phase II/IIb chemical. Phase II/IIb introduced a plan for synchronizing compliance monitoring across several existing and upcoming rules. Monitoring frequencies for most source-related contaminants were coordinated with compliance periods of three (3) years each. Phase II/IIb monitoring requirements also established:

1. Sampling locations for surface and groundwater systems;
2. The initial sampling frequency that is specific for a contaminant or contaminant group;
3. Lower repeat sampling frequencies for water systems that do not detect a specific contaminant or contaminant group during the initial monitoring;
4. Increased monitoring frequencies for water systems that do detect initial contaminants,
5. Monitoring waivers for reducing or eliminating the sampling frequencies; and,
6. One-time monitoring requirements for 30 other unregulated contaminants.

The Phase V Rule, effective on January 17, 1994, set standards for 23 more contaminants. Contaminants monitored under Phase V included five (5) IOCs, cyanide, three (3) VOCs, and fifteen (15) pesticides or SOCs. The EPA set different monitoring schedules for different contaminants, depending on the routes by which each contaminant enters the water supply. In general, surface water systems must take samples more frequently than groundwater systems because the source water is subject to more influences that are external. Systems that prove over several years that they are not susceptible to contamination can apply for a variance to reduce monitoring frequency. Results of sampling have not prompted any concerns regarding contaminants regulated by the Phase II/IIb and Phase V Rules for the City of Edina.

4.2.1.4 *Arsenic Rule*

The EPA based the 1975 arsenic standard of 50 ppb on a Public Health Standard dating back to 1942. The EPA proposed a revised Arsenic Rule in June 2000 and published the revision in the Federal Register on January 22, 2001. This revised rule applies to all community water systems and non-transient non-community water systems and requires compliance with an MCL of 10 ppb, based on samples obtained from all entry points to the distribution system. In addition to the MCL, the rule also specifies a non-enforceable MCLG of zero. The compliance date for the revised Arsenic Rule is January 23, 2006. Arsenic causes adverse health effects in humans at high exposure levels. High levels of arsenic typically lead to gastrointestinal irritation accompanied by difficulty in swallowing, thirst, hypertension, and convulsions. A range from 70 to 180 mg/L is the estimated lethal dosage for humans. Edina has never experienced concerns related to compliance with the Arsenic Rule.

4.2.1.5 *Radionuclides Final Rule*

The EPA proposed a NPDWR for six (6) radionuclides in 1991, which included combined radium 226, radium 228, (adjusted) gross alpha, beta particle and photon radioactivity, radon, and uranium. The EPA published a revision to this rule, promulgating the final drinking water standards for (non-radon) radionuclides in drinking water, in December 2000. This revised rule became effective on December 8, 2003. The revised rule finalized MCLG for all regulated radionuclides at zero. This rule, which applies to all community water systems, changes the monitoring requirements to include sampling from all distribution system entry points. The adverse health effects associated with exposure to radionuclides include radiotoxicity, which affects human tissue, and chemotoxicity, which affects human organs. Research links extended radionuclide exposure to cancer. Table 4.1 provides the MCLs and MCLGs for regulated radionuclides.

Table 4.1 Primary Drinking Water Regulations for Radionuclides (Excluding Radon)

Radionuclides	MCL	MCLG
Radium 226/228	5 pCi/L	0
Beta and Photon Emitters	4 mrem/year	0
Gross Alpha Emitters	15 pCi/L	0
Uranium	30 µg/L	0

The City annually tests for radionuclides to ensure customer safety compliance with drinking water regulations. Some of Edina's wells have reported total radium and gross alpha concentrations that exceed the regulated MCL. The City treats these wells by addition of preformed hydrous manganese oxide (HMO) at WTPs No. 3 and No. 4 currently, and blends the raw water from Well 17 with Wells 4 or 6 in WTP No. 2 to reduce the radionuclide concentrations below the MCL. Wells 5 and 18 will serve the proposed facility, which have total radium concentrations near the MCL. For this reason, the proposed facility includes provisions for removal of radium as part of the treatment process.

The EPA proposed a Radon Rule in November 1999. The Rule did not pass, so there is currently no federally enforceable drinking water standard for radon. The originally conceived Radon Rule applied to all public water suppliers that use groundwater or mixed ground and surface water. The rule proposed a MCLG, a MCL, an alternative maximum contaminant level (AMCL), and requirements for multimedia mitigation (MMM) program plans to address radon in drinking water. The proposed regulation provided two (2) options for the maximum level of radon that is allowable in community water supplies. The proposed MCL was 300 pCi/L of drinking water, and the proposed AMCL was 4,000 pCi/L of drinking water. The AMCL applied to States with enhanced indoor air programs and the lower MCL applied to States without enhanced indoor air programs. The State of Minnesota has a developed indoor air quality program, which would suggest an associated AMCL of 4,000 pCi/L for the City of Edina.

The U.S. Surgeon General and EPA recommend that radon be mitigated if the radon level is 4 pCi/L of air or higher. The existing WTPs in Edina have reported higher radon concentrations in the air above this threshold in the recent past. The City monitors and records daily facility radon measurement in WTPs No. 2, No. 3, and No. 4. Although not a primary regulatory concern, the presence of radon in the raw water supply will promote discussion of additional provisions for ventilation in the WTP and considerations of treatment alternatives for radon removal.

4.2.1.6 *Total Coliform Rule (TCR)*

The TCR became effective under the SDWA on December 31, 1990. This rule established microbiological standards and monitoring requirements that apply to all PWSs. The purpose of the TCR is to prevent outbreaks of waterborne microbial diseases by regulating a group of organisms that include fecal coliform and *Escherichia coli (E.coli)*. The potential health effects of microbial organisms include gastroenteric and Legionnaires' disease.

The US-EPA published revisions to the 1989 TCR rule on February 13, 2013 and made minor corrections on February 26, 2014. The Revised Total Coliform Rule (RTCR) targets greater public health protection. The RTCR:

1. Requires public water systems that are vulnerable to microbial contamination to identify and fix problems; and
2. Establishes criteria for systems to qualify for reduced monitoring.

The presence or absence of total coliform is the general indication used to measure the level of pathogenic contamination within the water. However, the RTCR removed and replaced the MCLG and MCL for fecal coliform with MCLG for E. coli of zero (0). While the basic monitoring requirements of the TCR remain unchanged, the RTCR established criteria for systems to stay on reduced monitoring frequencies and establishes increased monitoring for high-risk systems or systems with a history of noncompliance. Public water systems that exceed the specified frequency of total coliform occurrence are required to conduct additional assessment. All PWSs must comply with the RTCR starting April 1, 2016.

The City of Edina has never had a positive total coliform or E. coli result to date.

4.2.1.7 *Stage 1 Disinfectants-Disinfection By-Products Rule (Stage 1 D/DBPR)*

The Stage 1 D/DBPR established MCLs for eleven (11) DBPs, categorized into two (2) groups of organic by-products (four (4) trihalomethanes (THMs) and five (5) haloacetic acids (HAA5s)) and two (2) inorganic by-products (chlorite and bromate). The Stage 1 D/DBPR also established maximum residual disinfectant level goals (MRDLGs) and maximum residual disinfectant levels (MRDLs) for three (3) disinfectants: chlorine, chloramines and chlorine dioxide. Compliance was required by January 2002 for all community water systems (CWSs) (public water systems that are connected to 15 year-round residences or serve 25 people in a residential setting on a year-round basis) serving more than 10,000 people.

Table 4.2 presents the MRDLs and MRDLGs for the three (3) disinfectants. The running annual average (RAA) of samples collected at TCR sampling locations, computed quarterly, governs compliance with the MRDLs. The regulation recognizes the beneficial disinfection properties of chlorine, chloramines and chlorine dioxide. The MRDLs and MRDLGs were determined as a balance to provide adequate control for public health effects while allowing the ability to control pathogens and other microbial waterborne microbial contaminants under varying conditions. Basing compliance on a running annual average allows CWSs the flexibility to increase disinfectant residual levels for short periods, as necessary to address specific issues within the water system and still maintain compliance.

A review of chlorine residual data provided by Edina indicates their distribution system chlorine residuals are well below the 4 mg/L MRDL. Ammonia in the well water results in combined chlorine residuals prevalent in the distribution system. Total chlorine residuals typically range between 0.4 and 1.6 mg/L, but routinely are less than 0.4 mg/L at sample sites exhibiting longer water age.

Table 4.2 Stage 1 D/DBPR Maximum Residual Disinfectant Levels and Goals

Disinfectant	MRDLs (mg/L)	MRDLGs (mg/L)
Chlorine (measured as Cl_2)	4.0	4.0
Chloramines (measured as Cl_2)	4.0	4.0
Chlorine Dioxide (measured as ClO_2)	0.8	0.8

Table 4.3 identifies the MCLs for the various DBPs regulated under Stage 1. The National Cancer Institute lists some DBPs as probable human carcinogens and links some to adverse effects on the liver, kidneys, nervous system and reproductive system.

Table 4.3 Stage 1 D/DBPR MCLs

Regulated Disinfection By-Products	Stage 1 MCLs (mg/L)
Total Trihalomethanes (TTHM)	0.08
Haloacetic Acids (HAA5)	0.06
Chlorite	1.00
Bromate	0.01

Total THMs are the sum of the following four (4) trihalomethanes: chloroform, bromdichloromethane, dibromochloromethane and bromoform. The Stage 1 TTHM MCL is 80 micrograms per liter (μg/L) based on a RAA from quarterly distribution system samples. HAA5 is the sum of the following five (5) haloacetic acids: monochloracetic acid, dichloroacetic acid, trichloroacetic acid, monobromoacetic acid and dibromoacetic acid. Stage 1 established a HAA5 MCL of 60 μg/L, as an RAA of quarterly distribution system samples. Stage 1 regulates chlorite, a degradation product of chlorine dioxide, at an MCL of 1.0 mg/L. Ozonation of water containing the bromide ion form bromate. The Stage 1 D/DBPR regulates bromate at 10 μg/L.

Disinfection by-products are not a concern for the City due to the current chloramination disinfection strategy. Chlorite and bromate formation are not concerns for the City of Edina since the existing or proposed future treatment processes do not include chlorine dioxide or ozone.

4.2.1.8 *Stage 2 Disinfectants/Disinfection By-Products Rule (Stage 2 D/DBPR)*

The EPA finalized and published the Stage 2 D/DBPR on January 4, 2006. The Stage 2 D/DBPR intended to reduce potential cancer, reproductive and developmental health risks from DBPs in drinking water. Under the Stage 2 D/DBPR, systems conduct an evaluation of their distribution system, known as an Initial Distribution System Evaluation (IDSE), to identify the locations with high DBP concentrations. The systems then use these locations as the sampling sites for Stage 2 D/DBPR compliance monitoring. The system determines whether each monitoring location complies with the MCLs for two (2) groups of DBPs (TTHM and HAA5). This approach, referred to as the locational running annual average (LRAA), differs from the Stage 1 D/DBPR requirements, which determines compliance by calculating the RAAs of samples from all monitoring locations across the system.

The Stage 2 D/DBPR also requires each system to determine if they have exceeded an operational evaluation level using their compliance monitoring results. The operational evaluation level provides an early warning of possible future MCL violations, which allows the system to take proactive steps to remain in compliance. A system that exceeds an operational evaluation level is required to review their operational practices and submit a report to the Primacy Agency that identifies actions to mitigate future high DBP levels, particularly those that may jeopardize their compliance with the DBP MCLs.

The PWS compliance deadline varies based on the population served. Wholesale and consecutive systems of any size must comply with the requirements of the Stage 2 D/DBPR on the same schedule as required for the largest system in the combined distribution system (defined as the interconnected distribution system consisting of wholesale systems and consecutive systems that receive finished water).

Based on the population of Edina, the City was required to begin collecting samples at the Stage 2 D/DBPR sites by October 1, 2013 and begin complying with rule requirements by July 2014.

4.2.1.9 *Ground Water Rule*

Historically, groundwater was free of microbial contamination, but recent research indicates that some groundwater is a source of waterborne disease. Gastrointestinal symptoms such as diarrhea, vomiting, etc. characterize most cases of waterborne disease. These symptoms are much more serious and can be fatal for persons in sensitive subpopulations such as young children, the elderly, and persons with compromised immune systems. In addition, research links long-term health effects such as adult onset diabetes and myocarditis (inflammation of the middle muscular layer of the heart wall) with some viral pathogens found in groundwater.

The 1996 amendments to the SDWA required EPA to develop regulations that require disinfection of groundwater systems "as necessary" to protect the public health. The Ground Water Rule (GWR) establishes multiple barriers to protect against bacteria and viruses in drinking water from groundwater sources and will establish a targeted strategy to identify groundwater systems at high risk for fecal contamination. The EPA issues the GWR as a final regulation in 2006. This rule applies to public groundwater systems (systems that have at least 15 service connections or regularly serve at least 25 individuals daily at least 60 days out of the year). Implementation of this rule began in January 2010. The requirements of this rule include:

- System sanitary surveys conducted by the State which are intended to identify significant deficiencies;
- Hydrogeologic sensitivity assessments for non-disinfected systems;
- Source water microbial monitoring by systems that do not disinfect and draw from hydrogeologically sensitive aquifers or have detected fecal indicators within the system's distribution system;
- Corrective action by any system with significant deficiencies or positive microbial samples indicating fecal contamination; and
- Compliance monitoring for systems that disinfect to ensure that they reliably achieve 4-log (99.99 percent) inactivation or removal of viruses.

A positive total coliform result from the TCR routine sampling triggers source water monitoring. Source water monitoring requires the system to collect a sample from the well(s) for further microbial analysis. If the sample is positive, then the system must take corrective action as directed by the state. Edina's best action to maintain compliance with the Ground Water Rule is to maintain chlorine residuals in the distribution system sufficient to prevent positive coliform results in their TCR samples.

4.2.2 Secondary Drinking Water Standards

The EPA established secondary drinking water regulations for contaminants that may adversely affect the finished water appearance, taste and odor; promote adverse digestive effects; discolor human skin and teeth; or have economic impacts (hard or corrosive water on plumbing fixtures and equipment). There are three (3) general categories of established secondary maximum contamination levels (SMCLs): aesthetic objectives, cosmetic objectives and technical effects. The USEPA maintains that the SMCLs represent reasonable goals for non-health threatening contaminants. States may establish higher or lower levels as appropriate for the local conditions. SMCLs are not federally enforceable, but individual Primacy Agencies can adopted them as enforceable standards. Table 4.4 provides a list of secondary contaminants and the associated SMCLs. No SMCLs are enforceable in Minnesota at this time.

Table 4.4 Secondary Maximum Contaminant Levels

Secondary Contaminant	Secondary MCL
Aluminum	0.05 to 0.2 mg/L
Chloride	250 mg/L
Color	15 color units
Copper	1.0 mg/L
Corrosivity	Non-corrosive
Fluoride	2.0 mg/L
Foaming Agents	0.5 mg/L
Iron	0.3 mg/L
Manganese	0.05 mg/L
Odor	3 TON (threshold odor number)
pH	6.5 to 8.5
Silver	0.1 mg/L
Sulfate	250 mg/L
Total Dissolved Solids (TDS)	500 mg/L
Zinc	5 mg/L

4.2.2.1 *Aesthetic Objectives*

Aesthetic objectives are water quality objectives that a water supply system strives to meet, although they do not have adverse effects on public health. These objectives include controlling color, taste, odor and foaming.

4.2.2.1.1 Color

In addition to undesirable aesthetics, color in potable water may also stain clothes and plumbing fixtures. A colorimeter measures color on a graded from zero to 70, with zero being perfectly clear water. The test is somewhat subjective, requiring a visual comparison of the color of the water sample to a color wheel. The SMCL for color is 15 color units. Color may be indicative of aluminum, iron, manganese, dissolved organic material, inadequate treatment, high disinfectant demand or the formation of DBPs.

Naturally occurring iron and manganese in the Edina water supply is largely responsible for the color in the finished water. Soluble iron and manganese oxidize when exposed to air (oxygen) and result in noticeable color and staining of wetted surfaces, fixtures, and laundry.

4.2.2.1.2 Foaming

Foaming is not typically a problem with ground water systems. Detergents or similar substances in the water usually cause foaming when the water becomes aerated. The EPA has established an SMCL for foaming agents of 0.5 mg/L. An oily, fishy or perfume-like taste is often associated with foaming.

4.2.2.1.3 Iron and Manganese

Water systems recognize the presence of iron in water by its rusty color, metallic taste and reddish or orange staining effects. Black or brown color, bitter metallic taste and black staining effects indicate manganese presence. The SMCLs for iron and manganese are 0.3 mg/L and 0.05 mg/L, respectively.

The majority of Edina's wells have historically reported iron and manganese concentrations above the SMCLs. In the most recent data provided by the City, iron concentrations ranged from 0.34 mg/L to 1.04 mg/L, indicating that all currently active wells exceed the SMCL for iron. Manganese concentrations ranged from 0.007 mg/L to 0.375 mg/L, with seven (7) of the seventeen (17) active wells reporting manganese level above the SMCL. The four (4) existing WTPs include chemical oxidation followed by filtration to remove iron and manganese from the raw water. The City monitors the iron removal through the existing pressure filters, but not manganese. WTP No. 5 will remove iron and manganese as a primary treatment objective for the proposed facility.

4.2.2.1.4 Taste and Odor

Public acceptance of the drinking water typically measures taste and odor rather than by scientific methods, with unacceptable taste and odor usually manifested as public complaints. Most organic and some inorganic compounds contribute to the taste and odor of water. Water systems perform odor tests to describe and quantify (subjectively) odor intensity. The threshold odor number (TON) is the standard unit measurement of odor intensity. Calculate the TON by determining the dilution ratio required to keep odor detectable in the water sample with odor-free water added.

The SMCL for odor in drinking water is 3 TON. Hydrogen sulfide is a potential source of odor in the Edina water supply. However, chlorination or chloramination oxidizes hydrogen sulfide, mitigating this odor-causing substance.

4.2.2.1.5 Sulfate

Sulfate is not toxic, carcinogenic or chronically harmful to humans in reasonable concentrations. At concentrations above 250 mg/L, sulfates give a salty taste to the water. The current SMCL for sulfate is 250 mg/L, based on taste and odor effects. The federal government considered a primary drinking water standard for sulfates in the past. The EPA proposed an MCLG of 500 mg/L for sulfate in December 1994. Resource limitations, however, forced the EPA to defer action on the proposed rule.

The existing WTP No. 5 source wells, Well No. 5 and No. 19, have reported sulfate concentrations ranging from 44.1 mg/L to 46.2 mg/L. Sulfate concentrations are not a concern for the proposed facility.

4.2.2.2 *Cosmetic Objectives*

Cosmetic objectives address effects that do not damage the body, but typically produce undesirable visual effects, such as skin and tooth discoloration. These objectives include controlling silver concentrations and controlling the fluoride residual in the distribution system.

The ingestion of silver greater than the non-enforceable secondary maximum contaminate level (SMCL) of 0.10 mg/L relates to skin discoloration.

In August 2015, the US Department of Health and Human Services released a new optimum fluoride concentration of 0.7 mg/L. Previous recommendations were for a range of concentration of fluoride between 0.7 and 1.2 mg/L to reduce cavity formation without producing significant fluorosis (enamel mottling) of the teeth. The EPA SMCL for fluoride is 2.0 mg/L and the regulated MCL is 4.0 mg/L. Above 2.0 mg/L, fluorosis becomes more prominent. Minnesota State Statutes indicate a required fluoride concentration between 0.9 and 1.2 mg/L.

The City of Edina adds fluoride by dosing fluorosilicic acid in their existing WTP's and within individual well houses if the well directly enters the distribution system. Since release of the lower optimum concentration recommendation, the City of Edina has adjusted their fluoride feed to maintain a fluoride residual ranging from 0.7 to 0.9 mg/L. The City's MDH official approved this reduction below the Minnesota State Statute. MDH staff suggested in the past that the optimum concentration for fluoride in drinking water could be reduced 0.5 to 0.9 mg/L. The availability of improved dental care and dental products throughout the country justifies the reduced fluoride concentration.

4.2.2.3 *Technical Effects*

Adverse technical effects can cause damage to downstream water equipment processes and can sometimes reduce the effectiveness of treatment for other contaminants. In addition, technical effects can cause damage in the distribution system components and fixtures in homes. These adverse technical effects include corrosivity and scaling.

By-products formed by corrosion of piping and plumbing have health, aesthetic and economic implications. The SMCL for corrosivity is non-corrosive water. Water pH and the distribution of carbonate species (carbonic acid, bicarbonate and carbonate) directly affect corrosion of metal components. Lower pH water tends to be more corrosive, so pH is evaluated a surrogate indicator of corrosivity.

The pH of the raw water for the Edina water supply typically ranges between 7.6 and 7.9. Finished water pH typically ranges from 7.3 to 8.0 leaving the existing WTPs. The pH is consistently within the established SMCL pH range of 6.5 to 8.5.

4.2.3 Other Water Quality Standards

4.2.3.1 *Hardness*

Water suppliers classify the hardness of a water as soft (below 60 mg/L as $CaCO_3$), medium hard (60 to 120 mg/L as $CaCO_3$), hard (120 to 180 mg/L as $CaCO_3$), very hard (180 to 350 mg/L as $CaCO_3$) and brackish (above 350 mg/L as $CaCO_3$).[5] Although higher values of hardness are not dangerous, public acceptance typically requires a water supply below 150 mg/L as $CaCO_3$. Hard water also tends to stain bathroom fixtures and leave scale in water heaters. Agencies recommend that suppliers of potable water maintain total hardness levels below 120 mg/L as $CaCO_3$, when economically feasible.

The hardness of the Edina water supply is typically in the range of 250 mg/L to 360 mg/L. Although the water supply falls into the categories of "very hard" and "brackish", it is not problematic. Removal of hardness ("softening") can be an expensive treatment process. Home water softeners allow system customers to address hardness on an individual basis, as they feel appropriate. The Project Team does not recommend implementation of water "softening" treatment for the City of Edina at this time.

4.2.3.2 *Sodium*

Although not a primary drinking water standard, the World Health Organization (WHO) recommends a maximum concentration standard of 200 mg/L for sodium. The WHO established this guideline for people on a restricted sodium diet. In 2014, the City of Edina reported sodium concentrations of 12.4 mg/L for Well No. 5 and 20.5 mg/L for Well No. 18. Both values are well below the WHO recommended concentration for sodium.

4.2.4 References

[1] Great Lakes Upper Mississippi River Board of State Public Health & Environmental Managers, Recommended Standards for Water Works, Health Educational Services, Albany, NY, 2012 Edition.

[2] Leland, David E., "Implementation Status of Oregon's Safe Drinking Water Act", Journal of the American Water Works Association, February 1993.

[3] Pontius, Frederick W., and Warburton, Albert E., "Inside S. 1316", Journal of the American Water Works Association, March 1996.

[4] Pontius, Frederick W., SDWA Advisor, 1997.

[5] Lindeburg, PE, Michael R., Civil Engineering Reference Manual for the PE Exam, Eighth Edition, 2001, p. 25-3.

4.3 Treatment Target Goals

Beyond continued compliance with all primary drinking water regulations, the City of Edina, together with AE2S, established additional treatment target goals for the future WTP No. 5. The treatment target goals implement treatment for iron and manganese (color) removal, promote compliance with D/DBP regulations, and enhance the stability of the residual disinfectant in the finished water supply.

The Project Team determined the following treatment target goals to be primary goals for WTP No. 5. In addition to identifying the treatment target goals, the Project Team also developed recommended measurement criteria for each goal.

4.3.1 Iron and Manganese Removal

Mitigation of the aesthetic effects of iron and manganese from the finished water supply is one of the primary objectives of additional water treatment.

Recommended Measurement Criteria:

- Consistently achieve iron and manganese concentrations less than half of the established SMCL;

	SMCL Regulation	**Treatment Goal**
	(mg/L)	(mg/L)
Iron	0.30	**0.15**
Manganese	0.05	**0.025**

4.3.2 Radium Removal

Mitigation of adverse health effects associated with exposure of water system customers to radionuclides is another primary objective for WTP No. 5.

- Consistently achieve combined radium (Radium-226 and Radium-228) and gross alpha emitters concentrations less than half of the established MCL;

	MCL Regulation	**Treatment Goal**
	(pCi/L)	(mg/L)
Combined Radium	5	**2.5**
Gross Alpha Emitters	15	**7.5**

4.3.3 Finished Water Stability - Disinfection

To provide a disinfection strategy consistent with the other WTPs, WTP No. 5 will require the addition of ammonia and chlorine to create chloramines because the raw water ammonia of Well No. 5 and No. 18 is lower than ammonia concentrations present in the rest of Edina's system. Maintaining this disinfection strategy and ensuring a biologically stable distribution system water quality is another primary objective for the proposed facility.

Recommended Measurement Criteria:

- Consistently provide a total chlorine residual of 1.5 mg/L to 2.0 mg/L in the finished water leaving the WTP;
- Consistently meet the established chloramine MRDL of 4.0 mg/L;
- Consistently provide stable total chlorine residuals in the City's distribution system; and
- No nitrification in the City's distribution system.

4.3.4 Radon

Although radon is not a regulated contaminant, high radon levels in the City's existing WTPs prompted conversation to mitigate radon in the proposed facility. The facility will include enhanced ventilation and radon monitors to ensure the safety of the operational staff.

Recommended Measurement Criteria:

- Consistently monitor the air quality of the facility and alert the proper City staff if radon levels are above 2.0 pCi/L.

In the event that the radon is above the recommended criteria, the City can install additional radon mitigation measures.

CHAPTER 5 TREATMENT PROCESS TECHNOLOGY ALTERNATIVES

Based on the review of raw water quality and desired treated water quality, the City will accomplish the following treatment objectives in the water treatment process:

- Iron and manganese removal
- Hydrogen sulfide removal
- Radium removal
- Radon removal (optional)
- Ammonia removal and reaction with chlorine to form chloramines
- Fluoridation
- Disinfection and maintaining a disinfectant residual in the distribution system

AE2S evaluated several alternative technologies to accomplish these treatment objectives for the proposed Water Treatment Plant No. 5.

5.1 Pre-Oxidation Processes

Removal of dissolved iron and manganese is primarily achieved by oxidizing the soluble, reduced forms (Fe^{+2}, Mn^{+2}) to the oxidized forms (Fe^{+3}, Mn^{+4}). The oxidized ions precipitate and form particles that the filters then remove. Water treatment facilities commonly provide thirty (30) minutes of reaction time in a detention tank after oxidation and prior to filtration, depending on the relative concentrations of iron and manganese and the type of media used for filtration.

Oxidation processes occur by reaction with a chemical oxidant dosed to the water, or on the surface of an oxidizing filter media, such as manganese greensand or pyrolusite. Pre-oxidation processes utilize a chemical to oxidize the iron and manganese prior to filtration. Candidate chemical oxidants include oxygen, chlorine, and permanganate. Oxygen is typically added to water through an aeration process, whereas chlorine and permanganate are typical dosed to the water via a chemical feed system.

5.1.1 Physical Oxidation Processes

In the context of ground water sources, aeration is typically used to remove dissolved gases (i.e. hydrogen sulfide and carbon dioxide), to remove radon and VOCs, and to introduce oxygen to assist in iron removal. Aeration can release hydrogen sulfide from water, eliminating the associated rotten egg odor. Aeration also releases carbon dioxide from the water, raising its pH, thereby making the water less corrosive. Aeration also enables the release of radon and volatile organic contaminants from the water. Aeration readily oxidizes iron, facilitating its removal in a subsequent filtration process. Unlike iron, aeration does not readily oxidize manganese.

The cost effectiveness of aeration depends on the relative concentrations of contaminants removed from the water. For example, if the water contains high concentrations of radon, hydrogen sulfide and iron, aeration will be very cost effective compared to the use of chlorine to remove these substances.

Several methods of contacting air with water to strip dissolved gases and oxygenate water are available and include: 1) natural draft, 2) forced or induced draft, and 3) pressure aeration.

5.1.1.1 *Method No. 1 – Natural Draft Aeration*

Natural draft aeration happens as the result of hydraulic structures in a water treatment facility. Turbulence created as water cascades over weirs and effluent launders introduces oxygen into the water and releases dissolved gases. Although aeration resulting from this process is less efficient than other aeration alternatives for iron oxidation, it still can contribute to water quality changes in a treatment facility.

Natural draft aeration can negatively affect the treatment facility. For example, if the facility doses chlorine to the water prior to a weir or effluent launder, the cascading action of water often times leads to the release of chlorine into the atmosphere. Likewise, the weir or effluent launder may release hydrogen sulfide through the cascading event. The release of hydrogen sulfide and/or chlorine vapors can cause significant odor and major equipment damage inside a water treatment facility if not closely managed. The design of Edina WTP No. 5 will consider and control the impacts of natural draft aeration.

5.1.1.2 *Method No. 2 - Forced/Induced Draft Aeration*

Aeration towers with stacked trays/tubes use the forced or induced draft method of aeration. A forced/induced aeration system generally consists of a blower with a weather-protected motor to provide adequate counter current flow of air through the enclosed aerator column. The typical hydraulic loading rate for a forced or induced draft aeration system varies from one (1) gallon per minute (gpm) to five (5) gpm per square foot of total tray area, with provisions for uniform distribution over the top tray. An ideal aeration system consists of a minimum of two units, with each unit designed to meet half of the peak day water demand. However, a firm aeration capacity at peak day water demand could be necessary if the aeration process is necessary for radon removal and compliance.

Air introduced to the system should be free of fumes, dust, and dirt, achieved via strategic placement of a louvered air intake system. The air intake design should also provide protection from insects, birds, and debris by use of a screen. Other design standards for a forced or induced draft system include five (5) or more trays at a spacing of at least six (6)-inches and an adequately designed air exhaust to vent the air to the outside atmosphere.

The forced or induced draft aeration system is a potential alternative for the proposed Edina treatment facility. Forced/induced draft aeration can oxidize iron and release dissolved gases such as hydrogen sulfide, radon, and carbon dioxide. The aerator would vent the exhaust to the atmosphere to disperse the dissolve gases removed from the water. Since iron and hydrogen sulfide cause a chlorine demand, the removal of these substances prior to chemical injection can lower the required chlorine dosage. Additionally, removal of carbon dioxide will increase the pH, improving the oxidation rate of manganese.

5.1.1.3 *Method No. 3 - In-Line Pressure Aeration*

In-line pressure aerators force air from a pressurized air source into the water through a fine bubble diffuser placed inside a pipe spool. Controlling the flow rate of air accomplishes sufficient transfer of air into the water to increase the dissolved oxygen concentration. The dissolved oxygen oxidizes iron. The benefit of pressure aeration is that it is within a pipe, and requires less head as compared to a forced/induced draft aerator, However, the pressure aerator is an enclosed system, so dissolved gases, such as hydrogen sulfide or radon, are not released from the water. Gas release occurs within the internal environment of the water treatment plant in subsequent process units, such as the filtration area or the clearwell. Adequate ventilation of these process areas would be required to provide a safe working environment for operators. Because of its characteristics, in-line pressure aeration is less desirable than forced/induced draft aeration to accomplish the treatment objectives at the Edina water treatment facility.

5.1.2 Chemical Oxidation Processes

Iron and manganese oxidation also occurs with chemical oxidants. The chemical alternatives evaluated for Edina focus specifically on those approved for potable water use by the EPA and the National Sanitation Foundation (NSF). The Project Team evaluated and compared these chemicals to the treatment goals as well as the ability to meet disinfection requirements, effectively. The chemicals evaluated include chlorine, potassium permanganate, and sodium permanganate.

5.1.2.1 *Chlorine*

Edina currently uses chlorine as a disinfectant and the City purchases it in a liquid form under pressure in a cylinder. When pressure releases from the cylinder, the chlorine vaporizes to a gas form, which then mixes with water to form a hypochlorous acid/hypochlorite ion solution. The chlorine feed system then doses the solution to the water. This chlorine solution oxidizes iron and hydrogen sulfide very quickly – the reaction reduces chlorine to chloride, which does not contribute to a disinfecting chlorine residual. Chlorine reacts less quickly with manganese. In most instances, the reaction between chlorine and soluble manganese is too slow to be effective for manganese oxidation in a water treatment facility. In fact, if the treatment facility does not remove the manganese prior to entering the distribution system, oxidation of the manganese will slowly occur in the distribution system, discoloring the water.

Chlorine also reacts with ammonia to form combined chlorine. Combined chlorine (a common form is monochloramine) is less powerful than the hypochlorous acid/hypochlorite ion form called free chlorine. At high dosages of chlorine relative to ammonia, the chlorine oxidizes the nitrogen in ammonia, and removes the ammonia from the water. If a system is feeding chlorine to water at a dose higher than that required too remove the ammonia, a free residual (chlorine not combined with ammonia) will form. Operating at this dosage range is termed breakpoint chlorination. Breakpoint chlorination is one option to remove ammonia from water.

Both free chlorine and combined chlorine (monochloramine) are disinfectants. For most organisms, free chlorine is a more potent disinfectant than monochloramine, although the monochloramine residual typically is more persistent in the distribution system. Residuals of both disinfectants can contribute to corrosion of metals.

Given its capabilities for meeting several, but not all treatment objectives, chlorine is not a viable standalone oxidant for use at WTP No. 5. Table 5.1 summarizes its capabilities relative to other oxidants.

Table 5.1 Chlorine Advantages and Disadvantages

Advantages	Disadvantages
• Breakpoint chlorination removes ammonia • Oxidizes hydrogen sulfide • Oxidizes iron • Is a disinfectant	• Does not remove radon • Slow to oxidize manganese, likely oxidizing manganese in the distribution system causing colored water • Free chlorine can react with organics to form disinfection by-products

5.1.2.2 *Potassium Permanganate*

Use of potassium permanganate ($KMnO_4$) for manganese oxidation is frequent due to the strong oxidizing potential of the permanganate ion (MnO_4^-). Its oxidizing power also enables it to oxidize iron and hydrogen sulfide, and is an effective chemical for treating taste, odor and color from organic matter, especially in surface water sources. It is typically obtained in a granular form and dissolved in water in a batch tank to prepare a solution dosed using a chemical feed pump. The dry chemical requires handling appropriate personal protective equipment to avoid staining of clothing and skin.

When the water source contains high concentrations of iron relative to manganese, the oxidant demand for iron is typically satisfied with aeration or chlorine prior to permanganate addition, since potassium permanganate is more expensive than other oxidants. If aeration or chlorine is not applied prior to the permanganate feed point, the permanganate will oxidize the iron, manganese and hydrogen sulfide present in water. Un-used permanganate will remain in the water and may result in a pink residual. As it oxidizes other substances, permanganate reduces and precipitates as manganese dioxide, a brown precipitate, which filters remove that follow the manganese oxidation process.

Unlike free chlorine, potassium permanganate does not react with organic matter to form disinfection by-products, nor is permanganate an effective disinfectant. Table 5.2 summarizes the advantages and disadvantages of $KMnO_4$.

Table 5.2 Potassium Permanganate Advantages and Disadvantages

Advantages	Disadvantages
• Does not produce regulated DBPs • Oxidizes iron, manganese and hydrogen sulfide effectively • Aids with radium removal when high raw water manganese concentrations present	• Not an effective disinfectant • Overuse can turn water pink • Obtained in dry form and requires dissolving in a batch tank prior to dosing to the water • Does not remove radon

5.1.2.3 *Sodium Permanganate*

Sodium permanganate ($NaMnO_4$) is an alternate form of permanganate available as a 20% solution. Its behavior relative to reactions with constituents in water is identical to that of potassium permanganate. It has recently gained traction as a substitute for potassium permanganate, especially for small to medium sized water treatment plants, simply due to convenience in handling. Sodium permanganate comes in bulk liquid form so operators do not need to prepare batches of liquid chemical by dissolving a dry chemical in water. Proportioning pumps identical to those used for potassium permanganate dose the sodium permanganate solution.

Table 5.3 lists the advantages and disadvantages of $NaMnO_4$.

Table 5.3 Sodium Permanganate Advantages and Disadvantages

Advantages	Disadvantages
• Does not produce regulated DBPs • Oxidizes iron, manganese and hydrogen sulfide • Convenient to handle – obtained in liquid form • Aids with radium removal when high raw water manganese concentrations present	• Not an effective disinfectant • Overuse can turn water pink • Does not remove radon

5.2 Filtration Processes

Water treatment facilities typically use filtration as a polishing step for the removal of suspended solids and particles from water. For ground water sources, oxidation of iron and manganese, coagulation, and lime softening often precedes filtration. Excluding membrane filtration technology, there are four general classes of filters including rapid rate gravity filters, rapid rate pressure filters, diatomaceous earth filters, and slow sand filters. Based on industry trends, treatment facility footprint considerations, and operator convenience, the Project Team deemed gravity filters and pressure filters most appropriate in the treatment concepts developed for this report.

5.2.1 Gravity Filtration

The use of a rapid rate gravity filter shall generally require pretreatment according to Ten States Standards Section 4.3.1.1, except that Section 4.8.1.2 allows for iron and manganese filtration after detention without sedimentation. Consider sedimentation if iron and manganese concentrations are so high as to cause an overload of iron and manganese solids on the filter. Determine the rate of filtration based on the raw water quality, the level of pretreatment, filter media, water quality control parameters, and competency of the operating personnel. The recommended maximum filter loading rate for a manganese dioxide coated rapid rate gravity filter under normal operating conditions and acceptable pretreatment is 3.0 gpm/ft^2 according to Ten States Standards Section 4.8.3.d.

According to Ten States Standards, facilities must provide a minimum of two (2) filter units. When providing only two (2) units, each filter shall be capable of handling the plant design capacity at normal and projected maximum daily demands at the approved filtration rate. When providing multiple filters, the remaining filters shall be capable of handling the plant design capacity at projected maximum daily demands at the approved filtration rate when the largest filter is off line.

Design the filter structure to include the following:

1. A minimum filter box depth of 8½ feet;
2. A minimum water depth of three (3) feet over the surface of the filter media;
3. A trapped effluent preventing backflow of air and airlocking of the media;
4. An overflow to prevent flooding;
5. Cleanouts; and,
6. A washwater drain having a capacity capable of handling the maximum backwash flow.

Also, consider all applicable safety precautions. The bottoms of the washwater collection troughs shall be above the expanded filter media level during backwashing, and the top level of each trough shall be at the same common elevation. Provide a minimum of two (2) inches of freeboard in the washwater troughs at the maximum backwash rate. The washwater troughs shall be equally spaced throughout the filter area, and the troughs shall be spaced to provide a maximum horizontal travel distance for the backwashed solids of three (3) feet.

The filter media shall be clean silica sand or other natural or synthetic media.

The media shall possess the following characteristics:

1. A total depth of not less than 24 inches and generally not more than 30 inches;
2. A uniformity coefficient of the smallest size medium no greater than 1.65; and
3. A minimum of 12 inches of media with an effective size no greater than 0.45 mm to 0.55 mm and with a specific gravity greater than other filtering material within the filter.

Types of filter media include anthracite, sand, granular activated carbon, gravel, or other acceptable media.

Ten States Standards Section 4.3.1.7 also does not recommend porous plate bottoms (underdrains) where they may clog by iron and manganese. Gravel support should comply with Ten States Standards Section 4.3.1.6.e.2 gradation requirements.

Provide the following appurtenances with every filter: 1) influent and effluent sampling taps, 2) a head loss gauge, and 3) a flow meter. Make provisions to allow sampling and head loss measurement at several filter interior locations via wall sleeves in the filter box.

Normal backwashing of a rapid gravity filter occurs at a minimum rate of 15 gpm/ft^2. Ten States Standards recommends designing systems to be capable of a rate of 20 gpm/ft^2 or the rate required to achieve 50 percent expansion of the filter bed. However, for greensand media and manganese-coated media, Ten States Standards requires normal wash rates of 8 to 10 gpm/ft^2 and 15 to 20 gpm/ft^2, respectively. The Ten States Standards require air washing capability of 3 to 5 cfm/ft^2 suitable for iron and manganese filtration plants and meeting the requirements of Section 4.3.1.9. When backwashing simultaneously with air wash, Ten States Standards state that wash water flows should not exceed 8 gpm/ft^2 unless operating experience demonstrates a need for higher flows and media loss is not problematic.

A rate of flow indicator, preferably fitted with a totalizer, shall be located in a place where the operator can easily read the flow along the main washwater line. Ten States Standards requires redundancy of the backwash pumps, unless an alternate source is available. The backwash shall last at least 15 minutes per filter at the design backwash rate.

Section 9.5 of Ten States Standards outlines the design requirements of filter backwash waste from iron and manganese filtration plants. It allows sand filter beds (Section 9.5.1), lagoons (Section 9.5.2), and sanitary sewer discharge (Section 9.5.3). Recycle of supernatant or filtrate from “red water” waste treatment facilities is not allowed except as approved by the reviewing authority.

Figure 5.1 illustrates the general plan view arrangement of a gravity filter.

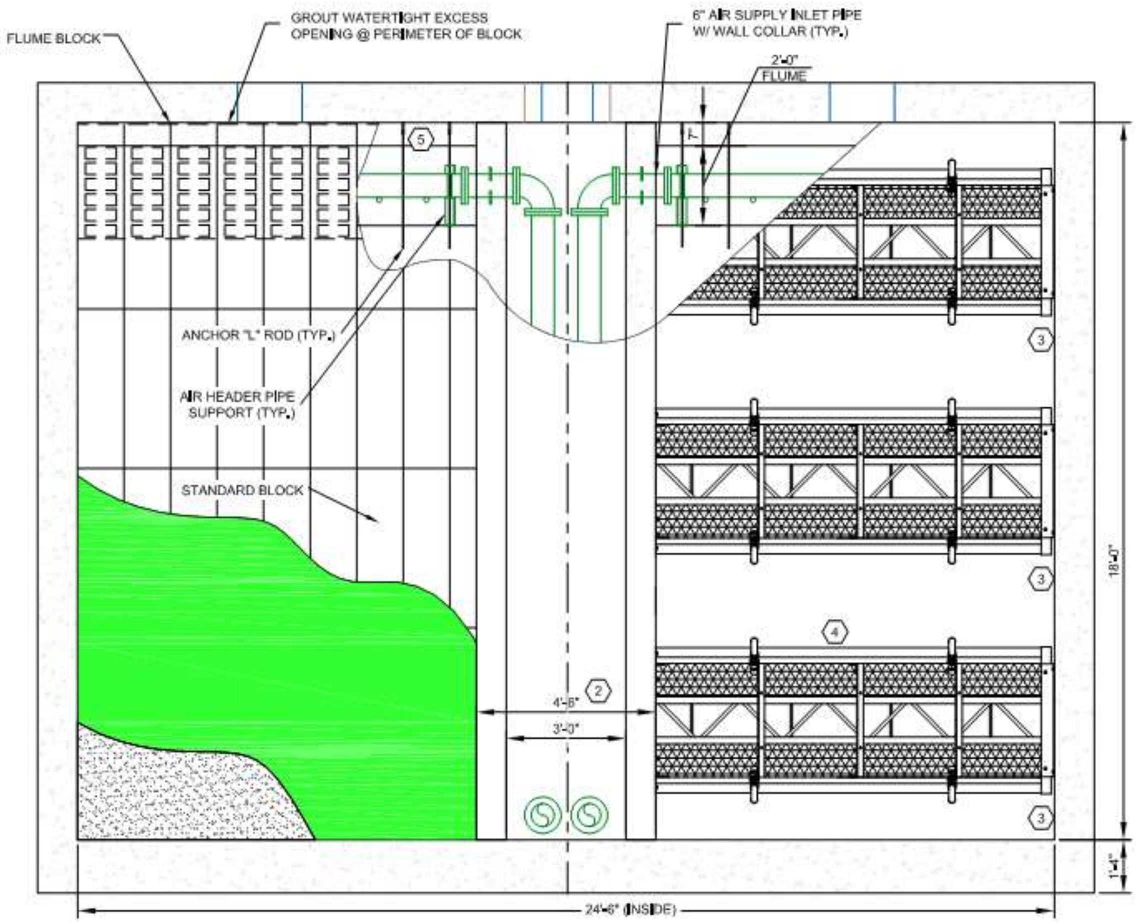

Figure 5.1 Typical Gravity Filter - Plan View

5.2.2 Pressure Filtration

Ten States Standards recommends that the rate of filtration within pressure filters not exceed 4.0 gpm/ft^2; and often iron and manganese WTPs reduce this value to 2.0 to 2.2 gpm/ft^2 to maintain consistent finished water quality. Additionally, Ten States Standards recommends the design of pressure filters include the following components:

1. Loss of head gauges on the inlet and outlet pipes for each battery of filters;
2. A flow meter for each filtering unit;
3. A minimum side wall height of five feet;

4. The top of the backwash water collection troughs to be at least 18 inches above the surface of the media;
5. The underdrain system to efficiently collect the filtered water and to uniformly distribute the backwash water at a rate not less than 15 gpm/ft^2 of filter area;
6. An air release valve on the highest point of each filter;
7. An accessible manway to facilitate inspection and repairs of at least 24 inches in diameter; and
8. A means to observe the wastewater during the backwashing process.

The minimum criteria relative to structural details, hydraulics, filter media, etc., provided in the conventional rapid rate gravity filters also applies to pressure filters, where appropriate. Figure 5.2 and Figure 5.3 below illustrate the general cross section and side elevation arrangement of a pressure filter.

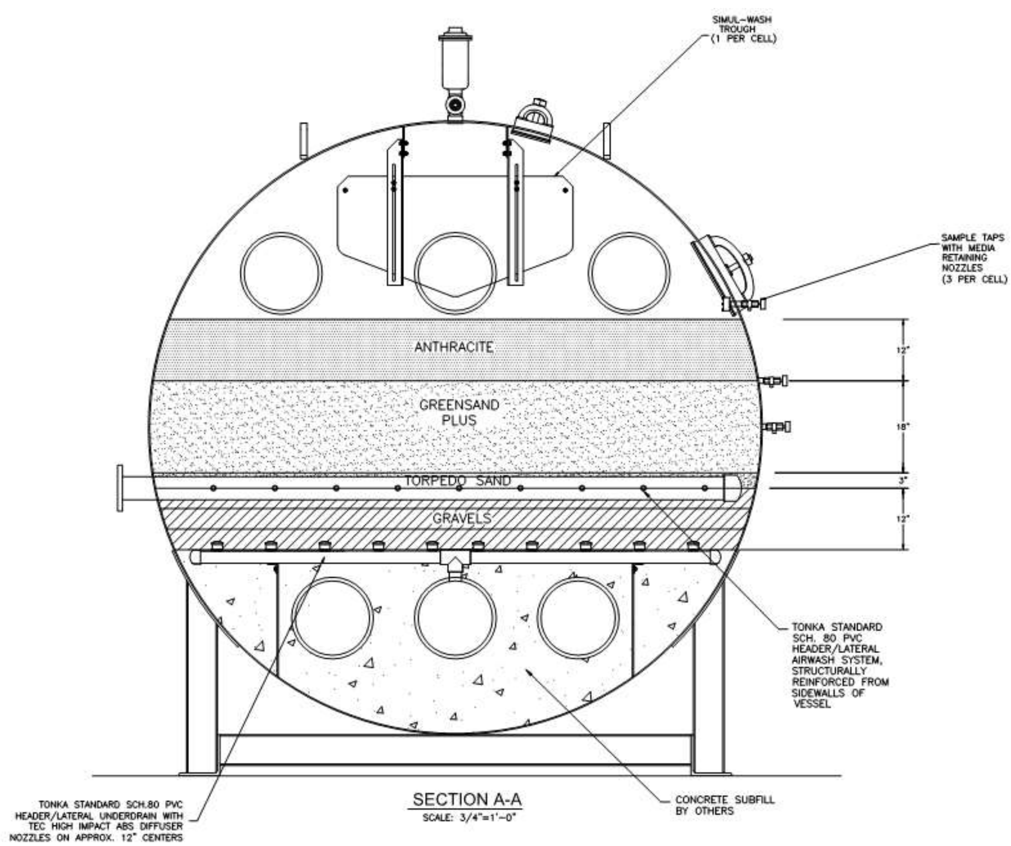

Figure 5.2 Pressure Filter Cross Section

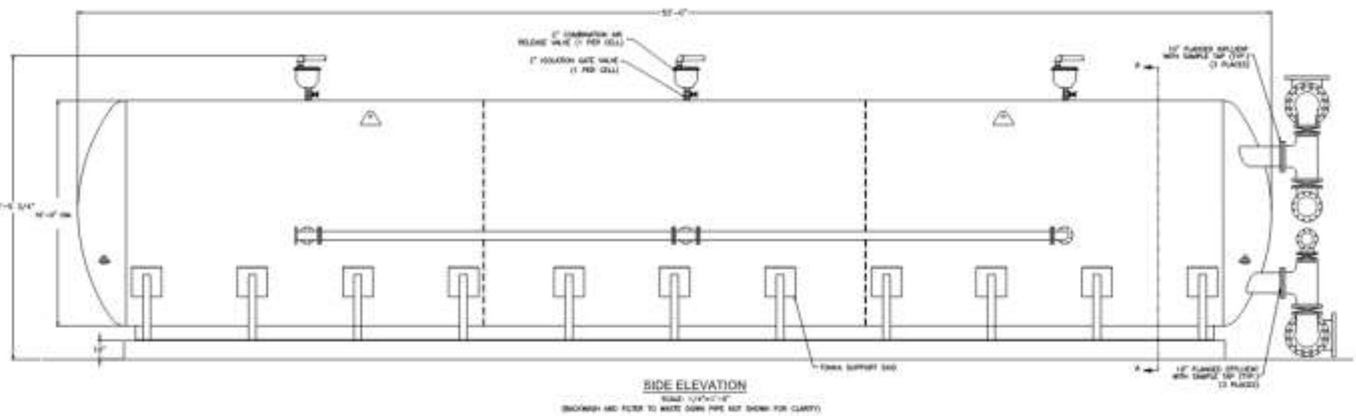

Figure 5.3 Typical Pressure Filter Side Elevation

5.2.3 Media Selection and Filtration Rates

Filter media is generally clean silica sand or other natural or synthetic media. Types of filter media include anthracite, manganese greensand, garnet, granular activated carbon, gravel, or other acceptable materials. Design standards for media include a total depth of not less than 24 inches and generally not more than 30 inches, a uniformity coefficient no greater than 1.65, a minimum of 12 inches of media with an effective size no greater than 0.45 millimeters (mm) to 0.55 mm, and a specific gravity greater than other filtering material within the filter. Manganese greensand is an alternate media for treating water containing iron and manganese. Manganese greensand has an effective size of 0.30 mm to 0.35 mm, a uniformity coefficient of less than 1.60, and a specific gravity of approximately 2.4. For iron and manganese removal by the lime softening process, dual media filters (i.e. sand and anthracite) are acceptable.

5.2.3.1 *Sand/Anthracite Media*

The most common filter media used in pressure or gravity filters is a dual sand/anthracite media. Typical dual media filters consists of 6-12 inches of silica sand overlain by 18-30 inches of anthracite media. The larger-sized anthracite settles on top of the smaller-sized sand following backwash. The anthracite traps larger particles and the sand traps smaller particles, enabling filtration throughout the entire filter bed. As a result, filter attain longer run times with lower rates of head loss accumulation and higher allowable loading rates. The major advantages of dual-media filtration are higher rates and longer runs. Anthracite/sand beds have operated at normal rates of approximately 4 gpm/ft^2 and peak rates as high as 8 gpm/ft^2 without loss of effluent quality. These two layers do mix slightly depending on the size, shape, and specific gravity of the media. Slightly mixed media beds have proven to perform better than distinctly layered media beds.

Dual media filters exhibit excellent turbidity/particle removal capability, routinely achieving filter effluent turbidity less than 0.1 NTU throughout a filter run. Water treatment facilities commonly use dual media to remove oxidized iron and manganese following oxidation and detention processes.

Table 5.4 summarizes the advantages and disadvantages of dual media compared to other media.

Table 5.4 Sand/Anthracite (Dual Media) Advantages and Disadvantages

Advantages	Disadvantages
• Higher loading rates • Longer filter runs • Lower head loss accumulation • Excellent turbidity removal	• Requires pre-oxidation • Higher backwash flow rates

5.2.3.2 *Manganese Greensand Media*

Manganese coated filter media is a filter media coated with a layer of oxidized manganese. Several types of manganese coated media exist, including manganese coated sand and greensand (glauconite). The media oxidizes the iron and manganese in the water passing through and the oxidized iron and manganese precipitates. Either the media catches the precipitate or it adsorbs to the media. The oxidizing capability of the media diminishes over time, and must be regenerated with another oxidant, typically potassium or sodium permanganate. Chlorine regenerates and maintains the oxidizing nature of the media in certain applications when the filter maintains a free chlorine residual.

HMO used for radium removal is the same chemical as the coating on manganese greensand. This creates the potential for adsorption of radium to the filter media, and as a result, may create a radioactive filter media. The presence of radium in the raw water feeding the proposed facility in Edina makes manganese greensand media an undesirable filter media.

Pretreatment chemicals and required pretreatment process prior to manganese greensand filters depend on the raw water quality. Waters containing high iron and manganese concentrations may require pre-oxidation of iron and manganese to reduce the oxidation demand on the greensand media. Placing a coarser anthracite layer above the greensand media in the filter helps to remove iron/manganese precipitates, lengthen the filter runs, and reduce the regeneration requirements of the greensand. In many cases, systems effectively treat water containing low iron concentrations and low to medium manganese concentrations using continuously regenerated greensand filters without prior oxidation chemical requirements.

Ten States Standards Section 4.8.3 provides several guidelines for design of manganese greensand filters. Depending on the expected oxidant demand, consider several points of permanganate application, with one point directly prior to filtration to achieve regeneration. Apply other oxidants such as aeration and chlorine ahead of $KMnO_4$ to reduce the demand for and subsequent cost of $KMnO_4$. Depending on the raw water quality, provide an anthracite media cap of at least 6 inches over the top of manganese dioxide coated media. The anthracite should have an effective size of 0.8 to 1.2 mm and a uniformity coefficient less than or equal to 1.85. A typical loading rate on manganese greensand filters is 3 gpm/ft^2.

Table 5.5 summarizes the advantages and disadvantages of manganese greensand filter media.

Table 5.5 Manganese Greensand Advantages and Disadvantages

Advantages	Disadvantages
• Effectively used without pre-oxidation with low iron and medium manganese concentration raw water • Can be regenerated with chlorine in low oxidant demand water	• Pre-oxidation of high iron/manganese required • Small grain size can accumulate high head loss • Becomes radioactive over time with raw water radium presence

5.2.3.3 *Engineered Media (Pyrolusite)*

Pyrolusite is the common name for naturally occurring manganese dioxide and is available in the United States, United Kingdom, South America, and Australia. It is a mined ore consisting of 40 to 85 percent manganese dioxide by weight. Since the individual pieces of Pyrolusite are made of MnO_2, the development of a manganese dioxide coating is not necessary as required by manganese greensand media. The various configurations of pyrolusite also provide extensive surface sites available for oxidation of soluble iron and manganese.

Pyrolusite filters are a blend of pyrolusite and sand, typically between 10-50 percent by volume, to combine a filtering media with the oxidizing properties of pyrolusite. No chemical regeneration is required, but facilities typically feed chlorine prior to filtration to assist in continuous regeneration.

Backwash is critical for proper operation of pyrolusite filters. Because of the high specific gravity of this filter media, additional backwash flow is necessary to fluidize the media bed. Attrition during backwash can be a benefit as it exposes more surface sites for oxidation of soluble iron and manganese.

The primary benefit of Pyrolusite is the ability to achieve extremely high filtration rates in comparison to traditional media filters or manganese greensand. Typical filtration rates range from 3-5 gpm/ft^2 where pyrolusite offers a filtration rate between 10 and 15 gpm/ft^2.

Table 5.6 summarizes the advantages and disadvantages of pyrolusite filter media.

Table 5.6 Pyrolusite Advantages and Disadvantages

Advantages	Disadvantages
• Smaller filter footprints	• More costly • Higher backwash flow required.

5.3 Disinfection

Two common methods of disinfection for municipal water treatment plants include chloramination and breakpoint chlorination. Ammonia in the water reacts with chlorine to form chloramines. The reactions create monochloramine (NH_2Cl), dichloramine ($NHCl_2$) and trichloramine (NCl_3) as shown in the following equations.

Monochloramine: $NH_3 + HOCl \leftrightarrow NH_2Cl + H_2O$

Dichloramine: $NH_2Cl + HOCl \leftrightarrow NHCl_2 + H_2O$

Trichloramine or nitrogen trichloride: $NHCl_2 + HOCl \leftrightarrow NCl_3 + H_2O$

The concentration of each type of chloramine depends on the concentration of chlorine added relative to the ammonia present in the water. As the chlorine reactions occur, the free ammonia residual decreases. Figure 5.4 illustrates the changes in chlorine residual and ammonia concentrations with addition of more chlorine to water that contains ammonia. The horizontal axis is the ratio of chlorine to ammonia (Cl_2 to NH_3 as N), and the vertical axis is the chlorine residual. The type of residual chlorine formed changes with increased ratios of chlorine to ammonia. Monochloramine forms in increasing concentrations as the chlorine to ammonia ratio increases up to 5:1. In this range, free ammonia concentrations decrease as it reacts with chlorine. In the range of ratios between 5:1 and 7.6:1, dichloramines form, and the chlorine residual drops as nitrogen in the ammonia oxidizes to nitrogen gas. Beyond the chlorine to ammonia ratio of approximately 7.6:1, the breakpoint, free chlorine exists.

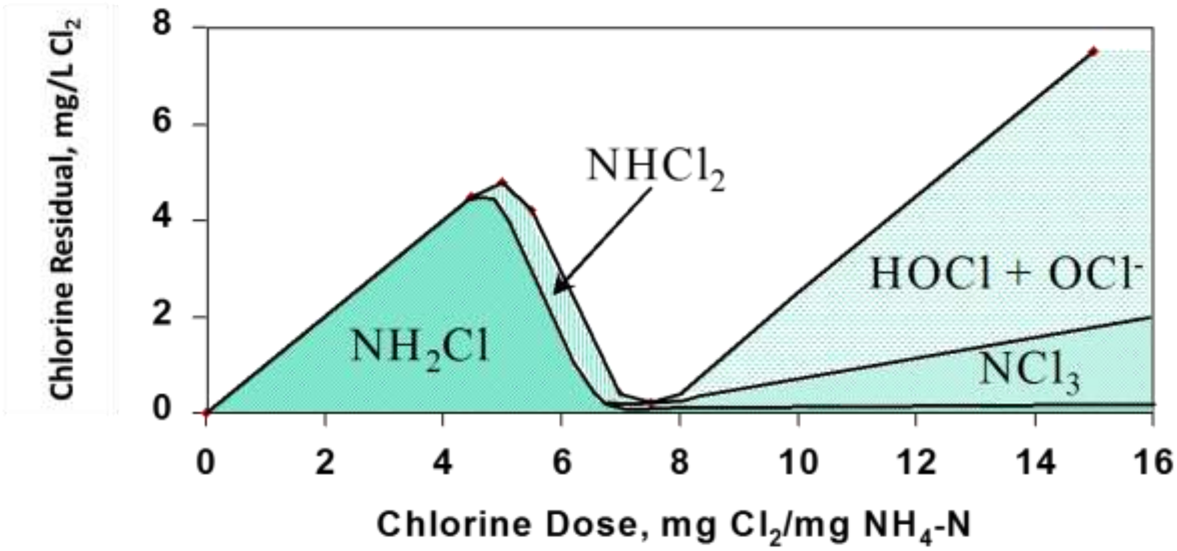

Figure 5.4 Breakpoint Chlorination Curve Explained

Both free chlorine and combined chlorine (monochloramine) are disinfectants. For most organisms, free chlorine is a more potent disinfectant than monochloramine, although the monochloramine residual typically is more persistent in the distribution system. Residuals of both disinfectants can contribute to corrosion of metals.

5.3.1 Existing Disinfection Strategy

For chlorination, milligrams per liter (mg/l) or pounds per day (PPD) of Free Available Chlorine (FAC) typically represents the disinfection demand. Identifying the FAC of the applicable process water creates a universal unit of measurement used to estimate the required dosages of various chlorine disinfection chemicals.

In the summer of 2016, the City adjusted their disinfection strategy from breakpoint chlorination to chloramination, which reduced their chlorine consumption approximately in half. Based on operational data from 2016 and current chlorine doses, the average demand for production of water throughout the Edina system is approximately 120 PPD FAC. This FAC demand aligns with average water production rates for 2016 of approximately 6 million gallons per day (MGD).

As part of the facility planning process, the Project Team projected an average day water demand of 8.63 MGD, and a peak water demand of 25.89 MGD, aligning with a 2040 planning horizon. If the City maintains the current disinfection strategy and an average total chlorine residual of 2.0 mg/L across the system, future chlorine demands increase to 140 PPD for average day water demand and 425 PPD for peak water demand.

5.3.2 Chlorine Alternatives

The Project Team evaluated three options for chlorine addition at the future WTP No. 5. The chlorination processes selected for evaluation included:

1. The City's current chlorination process, gaseous chlorination;
2. Bulk delivery of sodium hypochlorite; and
3. Onsite generation of sodium hypochlorite.

Evaluation included considerations of initial capital including building footprint, operation and maintenance (O&M) costs, and the advantages and disadvantages of implementation. The purpose of providing such information is in an effort to develop a preliminary recommendation related to a preferred chlorination process for the proposed facility.

5.3.2.1 *Gas Chlorination*

A gas chlorination disinfection system is generally comprised of bulk chlorine delivery and storage, a chemical feed and injection system, a chlorine gas scrubber or automatic shutoff valve system, leak detection and alarm systems, and chlorine dose and residual monitoring devices. For municipal water treatment purposes, transport of chlorine is typically by truck as a 100 percent (%) FAC liquefied compressed gas in either 1-ton or 150-lb steel cylinders. The City of Edina currently receives gaseous chlorine in 150-lb cylinders at all existing WTPs and well houses requiring their own chlorine feed systems

Delivery of 1-ton chlorine gas cylinders onsite requires a hoist and handling system for loading and unloading of the cylinders. Chlorine storage and feed rooms are typically equipped with leak detection equipment and a gas scrubber (leak adsorption system), and require proper ventilation and operator safety considerations.

Chlorine gas feed equipment includes:

1. Chlorinators, which regulate the gas feed rate;
2. Gas ejectors, which accomplish the chlorine-to-water mass transfer through a venturi-type device; and,
3. A contact facility, typically in the finished water storage chamber or clearwell.

Residual chlorine levels typically control gas chlorination systems and associated chlorine dosages, monitored with on-line chlorine analyzers.

Table 5.7 describes the advantages and disadvantages associated with implementation of a gas chlorination system at the proposed WTP No. 5.

Table 5.7 Gas Chlorine Advantages and Disadvantages

Advantages	Disadvantages
• Gas chlorination is a process that Edina WTP staff are familiar with operating. • The chemical consists of 100% FAC, which makes calculating feed rates much easier. • There is no degradation of the disinfection chemical.	• Storage of liquefied chlorine gas requires the development, implementation, and upkeep of an EPA Risk Management Plan, and OSHA Process Safety Management Plan. • Toxicity – operational safety hazard to staff, as well as risk to businesses, residences, or other institutions located adjacent to or near the WTP. • Corrosive to equipment and requires frequent operator O&M. • High capital cost associated with gas scrubber, if required.

5.3.2.2 *Bulk Delivery of Sodium Hypochlorite*

This system involves bulk delivery of liquid sodium hypochlorite (NaOCl), typically 12.5% by weight, into a bulk storage tank at the WTP. Depending on the size of the system, one (1) or more transfer pumps are required to convey the NaOCl solution from the bulk storage tank to a day tank. Peristaltic or diaphragm pumps dose NaOCl at desired disinfectant injection points. Bulk NaOCl systems also require miscellaneous tank level sensors, vents, gauges, and piping and appurtenances. Table 5.8 summarizes the characteristics of sodium hypochlorite systems.

Table 5.8 Sodium Hypochlorite Advantages and Disadvantages

Advantages	Disadvantages
• Straightforward delivery, storage, and feed process. • Improved staff and public safety compared to chlorine gas.	• Liquid NaOCl can degrade over time; consider strategic bulk delivery scheduling.

Advantages	Disadvantages
• Simplifies regulatory requirements of EPA's Risk Management Plan, and eliminates and/or reduces the OSHA Process Safety Management requirements. • Low capital cost.	• New process to Edina WTP staff with new O&M procedures, etc. • Off-gassing is a common problem with liquid 12.5% NaOCl systems, resulting in concentration decay with increased storage times. • High O&M cost associated with chemical.

5.3.2.3 *Onsite Generation of Sodium Hypochlorite*

Onsite generation of NaOCl involves the use of an electrolytic cell to induce an electrical current in a sodium chloride brine solution to produce a 0.8% NaOCl solution. Several components of an onsite generation system include bulk storage of salt, brine tank, water softener, water heater, NaOCl generation unit(s), NaOCl storage tanks, chemical metering pump feed equipment, and all associated booster and transfer pump systems. Onsite generation systems also require miscellaneous tank level sensors, gauges, piping and appurtenances, and a robust venting system.

The basic chemical and power requirements for production of 0.8% NaOCl with an onsite generation system are as follows:

1 lb of FAC =	Salt (NaCl)	~3 lbs
	+ Softened Water	~15 gallons
	+ Power	~2 kilowatt-hours (kWh)

Table 5.9 describes the advantages and disadvantages of on-site chlorine generation.

Table 5.9 Onsite Generation Advantages and Disadvantages

Advantages	Disadvantages
• No EPA Risk Management Plan or OSHA Process Safety Management requirements. • Increased staff and public safety compared to other alternatives.	• New process to Edina WTP staff with new O&M procedures, etc. • Relatively complex system with multiple processes, tanks, and operational requirements.

Advantages	Disadvantages
• Fresh NaOCl continuously produced results in reduced degradation. • Relatively low O&M expense associated with chemical acquisition.	• Potentially increased operator involvement compared to other systems. • High capital cost. • Greater space requirements than other systems. • Relatively high fixed O&M expenses.

5.3.3 Ammonia Alternatives

In the City's existing WTP's, raw water ammonia concentrations form enough chloramines to maintain a consistent 2.0 mg/L chloramine residual leaving the facility. For Wells No. 5 and No. 18 serving future WTP No. 5, the raw water ammonia is not high enough to provide this level of chloramination. Preliminary jar testing and pilot study results showed a maximum monochloramine residual of 1.1 mg/L, indicating that supplemental ammonia is necessary to achieve the recommended chloramine residual leaving the facility.

The Project Team evaluated three options for ammonia addition at the future WTP No. 5. The ammonia processes selected for evaluation included:

1. Anhydrous ammonia (compressed or liquefied gas);
2. Aqua ammonia (aqueous); and,
3. Ammonium sulfate (dry/solid or liquid).

Evaluation included considerations of initial capital including building footprint, operation and maintenance (O&M) costs, and the advantages and disadvantages of implementation. The purpose of providing such information is in an effort to develop a preliminary recommendation related to a preferred chlorination process for the proposed facility.

5.3.3.1 *Anhydrous Ammonia*

Anhydrous ammonia is typically stored in 150-lb compressed gas cylinders, and is the most commonly used chemical form of ammonia. Application of anhydrous ammonia requires a vacuum injection system, much like a gaseous chlorine injection system. While anhydrous ammonia is a relatively typical selection for chloramination, storage and handling of a flammable and toxic compressed gas is not preferred if other viable options exist. Use of anhydrous ammonia at the WTP site would likely require maintenance of more extensive safety measures and risk management procedures.

Gaseous ammonia injection systems differ slightly from gaseous chlorine systems because additional measures must be in place to ensure the injector does not encrust with hardness. Ammonia is a very basic chemical that will cause hardness precipitation in the injector, creating a routine maintenance item. To eliminate this concern, a reverse osmosis or ion exchange system would soften the carrier water prior to gaseous ammonia injection. Another option is a periodic acid feed system to dissolve the hardness from the injector.

Table 5.10 describes the advantages and disadvantages of anhydrous ammonia.

Table 5.10 Anhydrous Ammonia Advantages and Disadvantages

Advantages	Disadvantages
• Edina operates a gaseous injection system in existing facilities. • The chemical consists of 100% ammonia, which makes calculating feed rates much easier. • There is no degradation of the disinfection chemical.	• New process to Edina WTP staff with new O&M procedures, etc. • High capital cost • Storage of anhydrous ammonia gas requires the development, implementation, and upkeep of an EPA Risk Management Plan, and OSHA Process Safety Management Plan. • Toxicity creates safety concern for staff and public • Corrosive to equipment and requires frequent operator O&M. • Scaling of injector creates need for a carrier water softening system or periodic acid wash system, which increases operator O&M and complicates the system.

5.3.3.2 *Aqua Ammonia*

Aqua ammonia (ammonium hydroxide) is a relatively unstable solution, is highly corrosive, and accompanied by safety issues. A common issue is loss of the chemical strength to volatilization if storage temperatures are not low enough. Facilities can feed aqua ammonia in solution with metered chemical feed pumps, but selection as the preferred ammonia chemical form for water treatment purposes due to its unstable nature and associated storage and dosing difficulties is undesirable. Table 5.11 describes the advantages and disadvantages of aqua ammonia.

Table 5.11 Aqua Ammonia Advantages and Disadvantages

Advantages	Disadvantages
• Easier to feed with a simple chemical feed system. • Relatively low O&M expense associated with chemical acquisition.	• New process to Edina WTP staff with new O&M procedures, etc. • Off-gassing is a common problem with aqua ammonia systems, resulting in concentration decay with increased storage times. • Highly corrosive which makes it an operational safety hazard to staff. • Corrosive to equipment when not stored properly.

5.3.3.3 *Ammonium Sulfate*

Ammonium sulfate is available in dry (solid) or liquid forms. Dry ammonium sulfate is typically available in 50-lb bags, and the associated batch and feed system requires a dry chemical hopper, auto feeder, solution tanks with mixers, and a control panel for operation of the system. A dry system requires more extensive operator man hours for loading and unloading of ammonium sulfate bags and loading the dry chemical into the batch and feed system. Dry chemical feed systems require regular maintenance to clean out equipment clogs, and can be quite labor intensive compared to liquid chemical feed systems.

Table 5.12 describes the advantages and disadvantages of dry ammonium sulfate.

Table 5.12 Dry Ammonium Sulfate Advantages and Disadvantages

Advantages	Disadvantages
• No EPA Risk Management Plan or OSHA Process Safety Management requirements. • Fresh continuously produced ammonium sulfate solution results in reduced degradation.	• New process to Edina WTP staff with new O&M procedures, etc. • Relatively complex system with multiple processes, tanks, and operational requirements. • Obtained in dry form and requires dissolving in a batch tank prior to dosing to the water

Advantages	Disadvantages
• Relatively low O&M expense associated with chemical acquisition.	• Greatly increased operator involvement compared to other systems. • Higher capital cost. • Greater space requirements than other systems.

Alternatively, ammonium sulfate is available as a liquid solution. Liquid ammonium sulfate is a very stable solution that is completely soluble and does not have the safety and handling issues often experienced with the other liquid ammonia form – aqua ammonia. Liquid ammonium sulfate feed requires only bulk solution storage and chemical metering pumps, as well as miscellaneous piping and appurtenances. Based on the simplicity of a liquid ammonium sulfate chemical feed system and the significant safety advantages over the other ammonia chemical options, the Project Team recommends liquid ammonium sulfate as the preferred ammonia system at future WTP No. 5.

Table 5.13 describes the advantages and disadvantages of liquid ammonium sulfate.

Table 5.13 Liquid Ammonium Sulfate Advantages and Disadvantages

Advantages	Disadvantages
• No EPA Risk Management Plan or OSHA Process Safety Management requirements. • Stable solution at supplied concentrations to reduce safety and handling issues • Relatively simple chemical feed system.	• New process to Edina WTP staff with new O&M procedures, etc. • Obtained in dry form and requires dissolving in a batch tank prior to dosing to the water • High cost associated with chemical acquisition.

5.4 Corrosion Control

Various technologies are available for corrosion control, but this section only discusses those relevant to the City of Edina. The City's current method is dosing a 50/50 blend of orthophosphate and polyphosphate, which is in the form of a pre-formed chemical called Carus™ 8500. Phosphate blend addition is a common form of corrosion control.

5.4.1 Orthophosphate/Polyphosphate Blend (OCCT Evaluation Technical Recommendations Source)

Water systems use an orthophosphate/polyphosphate blend to inhibit corrosion of iron pipe and other metals in the distribution system and sequester iron and manganese. While this method is common, there are still limitations to its application. This includes instances where aluminum exists in the distribution system and where impacts on downstream wastewater treatment plants may occur.

Aluminum may interfere with orthophosphate by forming aluminum phosphate precipitates that reduce the amount of orthophosphate available for lead and copper control and cause build-up in the water main that will result in smaller pipe diameters and increased head loss.

Wastewater impacts include concern about increasing the phosphorus loading to the treatment plant. This is an issue when wastewater treatment plants have stringent limitations on the amount of phosphorus that they can discharge to receiving waters.

In November 2016, the City had a Copper Rule exceedance that they traced to internal plumbing in houses constructed during a specific time-period in the early 1980's. Aside from this instance, the phosphate blend method has been an effective strategy to date for the City of Edina, so the Project Team does not recommend additional corrosion control methods at this time. The City may consider optimization of the orthophosphate / polyphosphate blend ratios to provide the best corrosion control method.

5.5 Radium Removal

Many treatment technologies exist that the EPA considers Best Available Technologies (BAT) for removal of radium. These include, but are not limited to, ion exchange, reverse osmosis, lime softening, greensand filtration, and addition of preformed hydrous manganese oxide (HMO) followed by filtration. HMO addition is the current method used throughout Edina for wells containing high levels of combined radium or gross alpha.

The 2007 feasibility study for WTPs No. 5 and No. 6 described the advantages and disadvantages of the radium removal alternatives listed herein. This study did not recommend lime softening due to the high capital and annual operation and maintenance costs, along with the complex nature of the lime softening process. Ion exchange requires that influent combined iron and manganese concentrations of less than 0.3 mg/L. Wells No. 5 and No. 18 both contain elevated levels of iron and manganese, requiring iron and manganese removal upstream of the ion exchange system. While iron and manganese removal is a treatment target goal for this proposed facility, the capital and O&M costs of operating an ion exchange system in addition to an iron and manganese removal system outweigh the costs of a similar system that includes a more affordable radium removal technology. This same disadvantage exists for a reverse osmosis system.

Installation of manganese greensand media with permanganate and preformed HMO addition is common. This treatment train provides a pre-oxidation and filtration system that removes iron, manganese, and radium simultaneously is common. Permanganate oxidizes the iron and manganese and provides continuous regeneration of the filter media and radium adheres to the HMO particles. Additionally, the manganese greensand media aids in radium removal by adsorption of the radium particles on to the media coating. In recent years, facilities with this type of treatment system have had problems with the disposal of the filter media once it reaches its usable lifespan. Radium accumulation occurs over time and results in a media classified as radioactive waste. Based on this knowledge, the Project Team does not recommend manganese greensand media use for the proposed facility.

5.6 Backwash Recovery / Recycle Processes

During the treatment process, filters regularly undergo backwashes for removal of built up particulates. This backwash water is then routed to either the sanitary sewer or some sort of backwash reclamation facility. As water resources in the area become more and more scarce, the use and re-use of water will become a more important topic for large water producers and individual water consumers. While a financial investment is required for the re-use of backwash water, providing good stewardship of the state's resources is a primary concern for the City of Edina. Facilities achieve backwash reclamation through traditional settling basins or through a treatment and recycle process utilizing plate settlers. The sections that follow detail these processes.

5.6.1 Traditional Backwash Reclamation Basin(s)

The use of a traditional backwash reclaim basin is the standard method for the recovery of process water. Typical guidelines for a backwash reclaim basin require a detention volume equal to the amount of water required to achieve a 15 minute backwash at 20 gpm per square foot for each filter. It is also important to note that the reclaim rate cannot exceed 10 percent of the maximum flow rate of the WTP. For the City of Edina, the reclaim rate for WTP No. 5 would be approximately 300 gpm. A backwash reclaim basin system includes two separate pump systems, one for recirculation of the clarified backwash water and a second for removal of the settled sludge from the bottom of the tank and pump it to the sanitary sewer.

Table 5.14 summarizes the characteristics of traditional backwash reclaim.

Table 5.14 Traditional Backwash Reclaim Basin Advantages and Disadvantages

Advantages	Disadvantages
• Simple to construct and maintain • Limited long term maintenance • Able to place under other treatment processes	• Longer settling times required • Large tank volume requires more excavation and concrete • Less efficient water recovery • Less flexible than other options • Confined space entry for maintenance

5.6.2 Above Grade Plate Settler

A second option for backwash recovery utilizes an above ground plate settler unit. This system decreases the footprint of the backwash water reclamation facility while providing increased sludge storage and improved recovery efficiency. The inclined plate technology works by having a flocculated solid/liquid stream enter a tank and flow upward between a pack of inclined plates. The solids fall to the plate surface, where they tumble by gravity down to a sludge collection hopper. The clarified effluent flows through orifice holes and exits the top of the settler. This technology is extremely efficient and provides backwash recovery to facilities that may not have the space to employ traditional techniques.

Figure 5.5 Above Grade Plate Settler

Figure 5.5 depicts a typical above grade plate settler and Table 5.15 summarizes the advantages and disadvantages of the above ground plate settler.

Table 5.15 Above Grade Plate Settler Advantages and Disadvantages

Advantages	Disadvantages
• Smaller footprint • Increased backwash efficiency • Easy access to all parts • More sludge storage • More flexible	• Continued touchup required on carbon steel. • More vertical building height required • Shorter service life

5.6.3 Below Grade Plate Settler

The third option for backwash recovery is similar to the above ground plate settler but installed inside a below ground concrete tank. Tank construction occurs with a sludge collection system below the inclined plate settler seen in Figure 5.6. A chemical addition and flocculation system precedes the plate settler system to coagulate the backwash solids in improve settling. The technology utilized in this construction is the same as the above ground type but it utilizes stainless steel plates and a concrete tank rather than a steel tank. These construction materials enable a significantly longer lifespan and lower annual maintenance costs. Characteristics of the below grade plate settler option are summarized in Table 5.12.

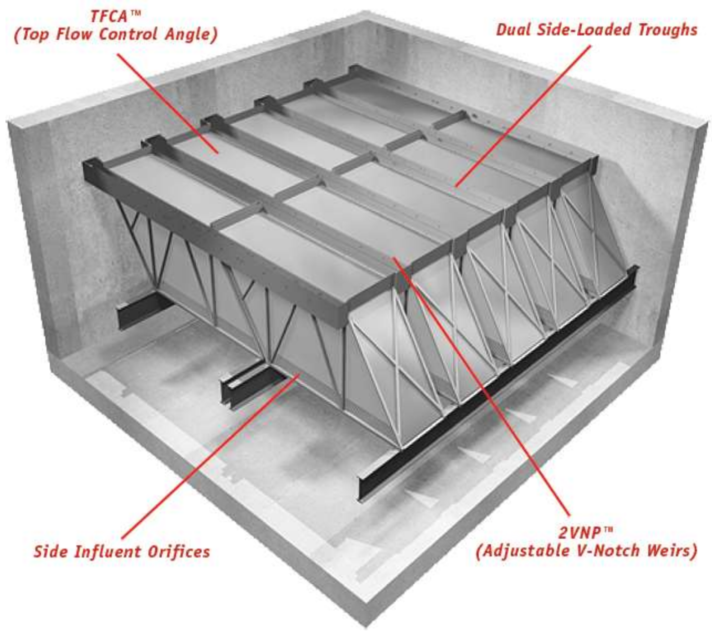

Figure 5.6 Below Grade Plate Settler

Advantages	Disadvantages
• Simple design • Low maintenance • Limited long term maintenance • Long service life • Stainless steel parts	• Confined space entry • More space needed • Less Sludge Storage

CHAPTER 6 PILOT STUDY EXAMINATION

AE2S conducted bench scale testing and a pilot study examination to determine the recommended treatment technologies to meet the treatment goals for WTP No. 5. The following sections detail the methods, results, and conclusions drawn from the analyses.

6.1 Preliminary Bench Scale Testing

AE2S conducted experiments to create breakpoint chlorination curves for the City's raw water supply Well No. 5 on May 31, 2017 and for Well No. 18 on June 1, 2017. Additionally, the design team completed preliminary oxidant demand jar testing of Well No. 5 on June 18, 2017. The goal was to observe the oxidant demand of the source water and gather water quality information helpful for conducting the pilot study. The preliminary jar testing also analyzed varying detention times and observed the use of permanganate and HMO for radium removal.

6.1.1 General Raw Water Characteristics

Raw water characterization of the investigated wells included analysis of iron, manganese, ammonia, and confirmation of hydrogen sulfide presence by a rotten egg odor. Table 6.1 provides a summary of the data collected during sampling events for Well No. 5 and No. 18. Iron and manganese concentrations in both wells exceed the Secondary Maximum Contaminant Levels (SMCLs) of 0.3 and 0.05, respectively, and are 0.1 to 0.2 mg/L higher in Well No. 5 compared to Well No. 18, whereas the ammonia concentration is slightly higher in Well No. 18.

Table 6.1 Well No. 5 and No. 18 Raw Water Characteristics

	Well No. 5		Well No. 18
Sample Date	5/31/2017	6/18/2017	6/1/2017
Iron, mg/L	0.44	0.57	0.36
Manganese, mg/L	0.409	0.338	0.257
Ammonia, mg/L as N	0.11	0.13	0.17
Hydrogen Sulfide Presence?	Yes	No	Yes

The raw water ammonia will exert an additional chlorine demand beyond that required for oxidation of iron, hydrogen sulfide, and manganese. Ammonia reacts with chlorine to form monochloramine, exerting a chlorine demand. The low ammonia concentrations (0.1 to 0.2 mg/L) present in both wells may require the addition of supplemental ammonia to provide a satisfactory monochloramine residual in the distribution system that matches monochloramine residuals from other Edina water treatment facilities and inhibits microbiological growth.

Oxidation of manganese with chlorine is slow, and with the concentration of manganese present in these wells, chlorine alone will likely not oxidize manganese to a concentration below the secondary maximum contaminant level (SMCL) of 0.05 mg/L within the residence time available in a water treatment plant. Permanganate oxidizes manganese at a much faster rate, making it a more feasible option for the future WTP.

AE2S noticed a hydrogen sulfide odor while collecting the water samples at both wells during the breakpoint chlorination sampling. Hydrogen sulfide will consume oxidant chemicals and could cause odor and corrosion issues in the WTP facility if not considered in the WTP design.

The pilot study will analyze the aeration process to determine whether aeration will oxidize the hydrogen sulfide and iron prior to chemical oxidation.

6.1.2 Breakpoint Chlorination Curve Test

AE2S created customized breakpoint chlorination curves for Wells No. 5 and No. 18. For this test, we filled jar test jars with 1L of water and dosed with chlorine over a range of 0.3 to 2.7 mg/L. After dosing each jar, a mixing apparatus gently stirred the water for approximately 30 minutes to allow reactions to take place. After the 30 minute reaction time, we collected samples from each jar, and analyzed free chlorine, total chlorine, free ammonia and monochloramine. The results are representative of the water quality from each well on that day. The curve could be slightly different for other well combinations or different days.

6.1.2.1 *Well No. 5 Results*

Table 6.2 summarizes the data collected from the jar test experiment and Figure 6.1 depicts the water quality trends for Well No. 5. As indicated by the concentration trends, Well No. 5 should achieve peak chloramination with a dosage of approximately 0.9 mg/l of chlorine and breakpoint chlorination with a chlorine dose of approximately 1.6 mg/L.

Table 6.2 Well No. 5 Breakpoint Chlorination Results

Jar	Cl_2 Dose (mg/L)	Total Cl_2 (mg/L)	Free Cl_2 (mg/L)	Free NH_3* (mg/L as N)	Mono-chloramine (mg/L)
0	0.0	0.00	0.00	0.11	0.02
1	0.3	0.21	0.00	0.09	0.24
2	0.6	0.42	0.00	0.05	0.42
3	0.9	0.64	0.00	0.00	0.67
4	1.1	0.57	0.00	0.00	0.40
5	1.4	0.23	0.18	0.00	0.16
6	1.7	0.41	0.38	0.01	0.07
7	2.2	0.93	0.86	0.00	0.10
8	2.7	1.34	1.18	0.00	0.09

Theoretical (stoichiometric) ratios (mg/L chlorine per mg/L ammonia (as N) to reach peak chloramination are 5:1 chlorine to ammonia, and the oxidant demands of iron and manganese are 0.63 mg/L chlorine per 1 mg/L iron, and 1.3 mg/L chlorine to 1 mg/L manganese. These ratios assume the reactions reach equilibrium. Based on the raw water on the day of testing, the stoichiometric peak chloramination chlorine dose is 1.4 mg/L. This is 0.5 mg/L higher than the experimental results. The difference between theoretical chlorine demand and the experimental demand indicate that the reactions might not have reached equilibrium, and that chlorine was not a strong enough oxidant to oxidize all the high manganese concentrations present in the raw water. Another explanation may be that involuntary aeration caused oxidation of iron during sampling or transportation of the sample for testing, which would reduce the chlorine demand of iron. The use of aeration to oxidize iron is part of the upcoming pilot study protocol.

Breakpoint chlorination (indicated by the valley in the total chlorine residual curve) theoretically requires a 7.6:1 ratio of chlorine to ammonia. The 0.11 mg/L ammonia concentration of Well No. 5, would require 0.8 mg/L of chlorine to reach breakpoint. Well No 5 required 1.6 mg/L of chlorine to reach breakpoint. This additional 0.8 mg/L chlorine demand above the ammonia demand is due to other constituents such as iron, a portion of manganese, or unquantified parameters such as organics or hydrogen sulfide.

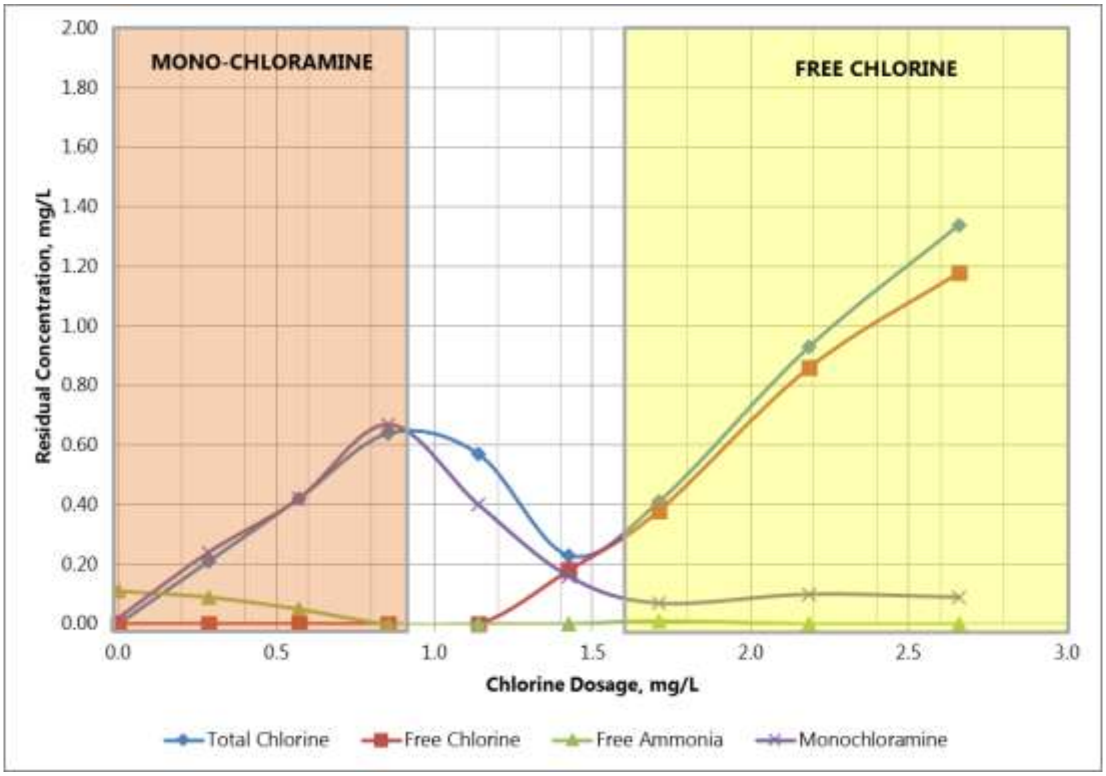

Figure 6.1 Well No. 5 Breakpoint Chlorination Curve

6.1.3 Well No. 18 Results

Table 6.3 summarizes the experimental data and Figure 6.2 depicts the concentration trends for Well No. 18. As shown by the chlorine residuals plotted in Figure 6.2, Well No. 18 should achieve peak chloramination with a chlorine dose of approximately 1.2 mg/l of chlorine and breakpoint chlorination with a chlorine dose of approximately 1.8 mg/L.

Stoichiometric calculations indicate a peak chloramination dose of 1.4 mg/L, which is slightly higher than the experimental results for reasons similar to those identified for Well No. 5. For breakpoint chlorination, Well No. 18 requires 1.3 mg/L to consume the raw water ammonia. Other constituents in the water contribute to the remaining chlorine demand.

Table 6.3 Well No. 18 Breakpoint Chlorination Results

Jar	Cl_2 Dose (mg/L)	Total Cl_2 (mg/L)	Free Cl_2 (mg/L)	Free NH_3* (mg/L as N)	Mono-chloramine (mg/L)
0	0.0	0.00	0.00	0.17	0.01
1	0.3	0.19	0.00	0.15	0.16
2	0.6	0.51	0.00	0.11	0.40
3	0.9	0.70	0.00	0.05	0.59
4	1.2	0.92	0.00	0.01	0.73
5	1.5	0.82	0.13	0.00	0.50
6	1.8	0.40	0.23	0.00	0.14
7	2.3	0.65	0.59	0.00	0.08
8	2.8	1.02	0.86	0.02	0.02

6.1.4 Potassium Permanganate Demand Tests

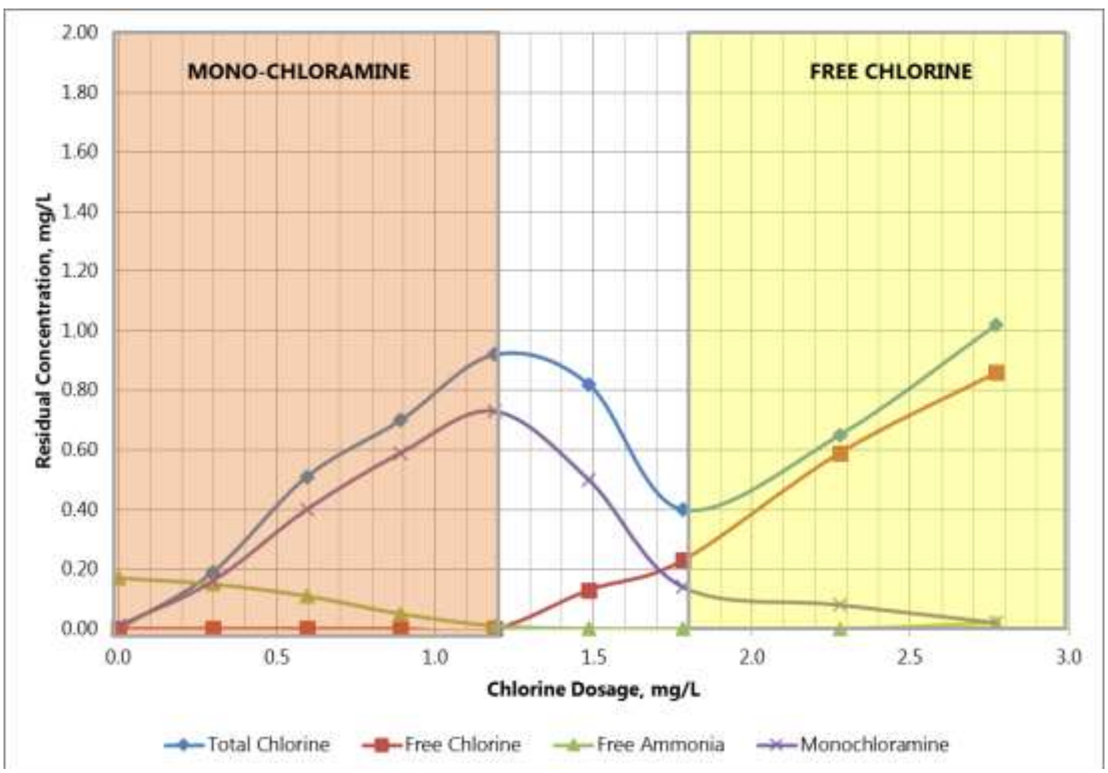

Figure 6.2 Well No. 18 Breakpoint Chlorination Curve

6.1.4.1 *Optimum Permanganate Dose Determination*

AE2S completed oxidant demand tests with potassium permanganate, which is a strong oxidizing agent. The first analysis varied permanganate doses to determine the optimum dosage to achieve manganese oxidation. We added a range of permanganate dosages to separate jar test jars, and mixed the jars for 30 minutes to enable oxidation reactions to occur. After 30 minutes of reaction time, we filtered samples from each dosage through a 0.45 μm filter to remove precipitated manganese, and analyzed the filtrate for manganese residual. As indicated by the low point of the manganese residual curve shown in Figure 6.3, the optimum dosage of permanganate was approximately 0.5 to 0.6 mg/L. At this dosage, the filtered manganese concentration was less than 0.05 mg/L. Dosing more than 0.6 mg/L resulted in excess permanganate in the water, indicating an overdose of permanganate at dosages greater than 0.6 mg/L. Although we did not measure iron residuals in the demand test, given the ease of iron oxidation with permanganate, the iron residual concentrations of the samples over the range of permanganate dosages was likely less than 0.05 mg/L.

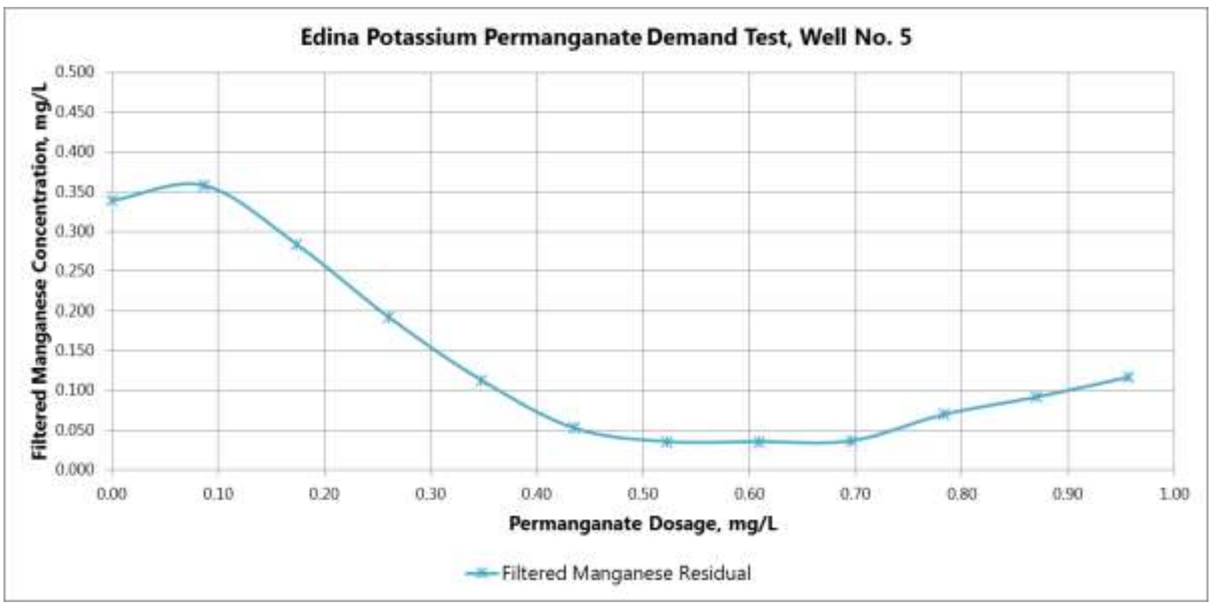

Figure 6.3 Potassium Permanganate Demand Test, Well No. 5

6.1.4.2 *Minimum Detention Time Determination*

The second permanganate experiment examined the required detention time to achieve the target manganese residual of less than 0.05 mg/L when dosing the water with 0.5 mg/L of permanganate. AE2S dosed a water sample with 0.5 mg/L of permanganate, and collected samples at five minute intervals of detention time, filtered, and analyzed for manganese. Based on the results plotted in Figure 6.4, the majority (90%) of oxidation occurred within the first five minutes of detention time. An additional 4% of removal took place between 5 minutes and 30 minutes. Note that the detention time determined is only applicable to this specific permanganate dose.

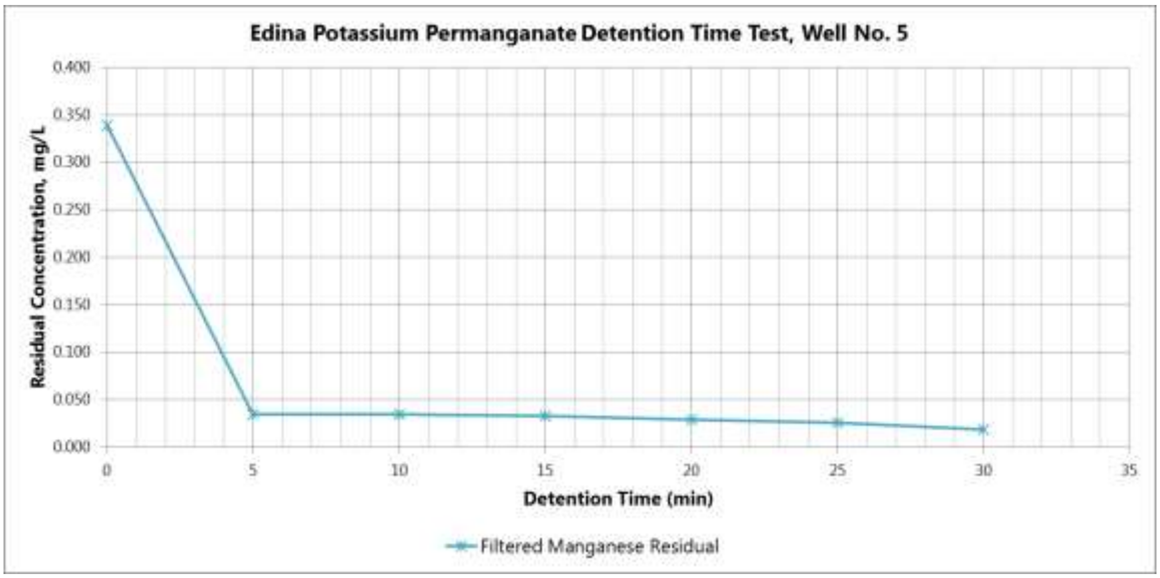

Figure 6.4 Potassium Permanganate Detention Time Test, Well No. 5

6.1.5 HMO Demand Tests

Similar to the potassium permanganate demand tests, AE2S completed a demand test to determine whether preformed hydrous manganese oxide (HMO) will remove manganese. Since HMO is manganese dioxide, similar to the coating on manganese greensand filters, HMO has potential to oxidize and adsorb manganese similar to the mechanism of removal in a greensand filter. The test involved analyzing filtered manganese residuals at varying HMO doses from 0.1 to 0.5 mg/L. Based on feed rate information provided by Edina operations staff, Edina currently adds 0.2 mg/L HMO to remove radium from the raw water at other treatment facilities. AE2S selected a dosage range encompassing the current operating dose for this experiment to examine manganese removal by HMO.

The results of the test, plotted in Figure 6.5, indicate that at the selected dosage range, HMO does remove some manganese, but does not achieve manganese residuals below the SMCL of 0.05 mg/L. Additional analysis is necessary to determine the required HMO dose to remove manganese to below the SMCL of 0.05 mg/L.

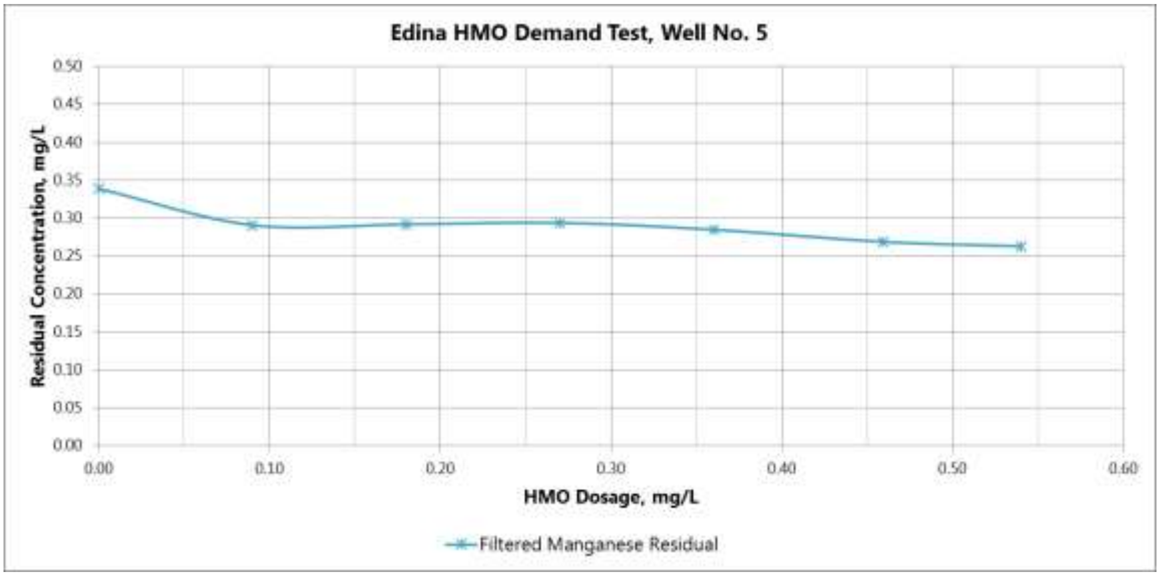

Figure 6.5 HMO Demand Test, Well No. 5

6.1.6 Radium Removal

Additionally on June 18, 2017, AE2S took radium samples and sent them to Eurofins Laboratory for analysis to determine whether permanganate alone removes radium, or if the raw water requires HMO for radium removal. The three samples taken on Well No. 5 water included a raw water sample, a sample dosed with 0.5 mg/L permanganate then filtered, and a sample dosed with 0.2 mg/L HMO then filtered.

Table 6.4 summarizes the results of the bench scale testing conducted on the raw water from Well No. 5.

Table 6.4 Bench Scale Testing Radium Removal for Well No. 5

Sample ID	Gross Alpha (pCi/L)	Radium-226 (pCi/L)	Radium-228 (pCi/L)	Combined Radium (pCi/L)
MCL (pCi/L)	15.0	-	-	5.0
Well 5 Raw Water	9.5 ± 3.1	2.2 ± 0.4	1.1 ± 0.6	3.3 ± 0.7
Well 5 w/ $KMnO_4$, filtered	3.7 ± 2.8	1.5 ± 0.3	1.1 ± 0.6	2.6 ± 0.7
Well 5 w/ HMO, filtered	8.7 ± 2.9	1.7 ± 0.3	0.94 ± 0.55	2.64 ± 0.63

Based on the data presented in Table 6.4, permanganate at a 0.5 mg/L dose reduced gross alpha by approximately 61% and Radium-226 by 32%, but provided no Radium-228 reduction. The combined radium reduction was approximately 21%. At a 0.2 mg/L HMO dose, gross alpha reduction was limited to 8%, Radium-226 to 23%, and Radium-228 to 15%. The two radium results equated to a combined radium reduction of approximately 20%. In general, permanganate was more successful at gross alpha removal and both permanganate and HMO at the analyzed concentrations removed combined radium to a similar level.

Appendix C includes a copy of the laboratory result report provided by Eurofins Laboratory.

This data is just one sample set, and AE2S recommends collection of additional data sets to confirm the conclusions drawn from initial bench scale testing. The pilot study will aim to confirm the effectiveness for permanganate alone to remove radionuclides, whether a higher HMO dose will remove more radium, and review impacts of detention time on radium removal.

6.1.7 Observations from the Experimental Results

- Wells No. 5 and No. 18 contain low levels of ammonia and concentrations of iron and manganese above the SMCLs. Both wells have a confirmed rotten odor, indicating presence of hydrogen sulfide.
- To provide adequate chloramine residual leaving the future facility, the addition of ammonia may be necessary. Increases in ammonia will increase the required chlorine demand.
- The raw water required approximately 0.5 mg/L potassium permanganate to oxidize manganese below the SMCL. Literature confirms that the reaction of permanganate with iron occurs faster than permanganate with manganese, indicating that some of the 0.5 mg/L demand may be from iron.
- Potassium permanganate oxidized manganese and resulted in a filtered manganese residual below the SMCL of 0.05 mg/L within a five (5) minute detention time. This indicates the effectiveness of this oxidant to remove manganese in the raw water.
- HMO did not oxidize manganese to below the SMCL within the range of HMO doses analyzed. When AE2S applied the current HMO dose used throughout other Edina WTPs to the raw water form Well No. 5, we observed an approximately 14% reduction in manganese residual post-filtration. The range of HMO doses applied did not reduce manganese to below the SMCL.
- Permanganate and HMO at the analyzed doses removed approximately 20% of the combined radium. Permanganate removed more Gross Alpha than HMO.

6.2 Pilot Test Protocol

The bench scale tests provided valuable input to refine the pilot plant protocol and insure measurement of appropriate water quality parameters. The primary objective of the pilot was affirmation of successful iron and manganese removal using permanganate and a sand and anthracite dual media filter. Additional treatment technologies investigated include aeration and/or chlorine for pre-oxidation, HMO for radium removal, chlorine for peak chloramination, and detention tanks for extended pre-oxidation reaction time.

AE2S conducted the pilot study protocol under the following guidelines:

1. Pump raw water from Well No. 18 to the pilot plant units.
2. Compare three process trains capable of the following:
    a. Column 1: Chlorine, permanganate, and HMO dosed to the raw water followed by a 30 minute detention time followed by filtration through Iron, Manganese, Arsenic, and Radium (IMAR™) media.
    b. Column 2: Chlorine, permanganate, and HMO dosed to the raw water followed by a 30 minute detention time followed by filtration through silica sand (0.45 – 0.55 mm, uniformity coefficient <1.6) and anthracite media (0.8 – 1.0 mm, uniformity coefficient <1.6).
    c. Column 3: Aeration of the raw water followed by dosing chlorine, permanganate, and HMO followed by a 30 minute detention time followed by filtration through silica sand (0.45 – 0.55 mm, uniformity coefficient <1.6) and anthracite media (0.8 – 1.0 mm, uniformity coefficient <1.6
3. Load the filters at 3 gpm/ft^2.
4. Provide chemical feed systems to dose chlorine, permanganate, and HMO. Confirm chemical feed rates by bulk concentration confirmation and pump drawdown analyses.
5. Analyze manganese removal based on the following items:
    a. Optimum permanganate concentration. Adjust dose as needed.
    b. Add HMO and/ or detention time and determine potential manganese removal benefits.
    c. Compare between the two sand and anthracite media gradations used
6. Obtain various sets of radionuclide samples and confirm preliminary bench scale findings related to removal of radium with permanganate or HMO.
7. Maintain a chlorine dose that provides operation of the pilot system at peak chloramination using the raw water ammonia concentration.

8. Determine the difference in chlorine dosages between Column 1 and Column 2 compared to Column 3 and examine the benefits and costs of aeration.
9. Examine filter performance over multiple filter runs.
10. Determine the expected length of filter runs based on filter head loss development and manganese breakthrough occurrence.
11. Verify field water quality tests by lab tests conducted by Pace Analytical.

The following list summarizes the objectives of each selected pilot study treatment alternatives:

- **Aeration** – determine the extent aeration will oxidize the raw water iron and hydrogen sulfide and reduce the required chemical oxidants added. Radon removal is an additional benefit.
- **Chlorine** – oxidize remaining iron post-aeration, when present, or in its entirety, when absent, upstream of permanganate addition to limit permanganate addition. Also to eliminate biological growth in the detention basin and through the filters.
- **Permanganate** – oxidize manganese and other remaining constituents.
- **HMO** – remove radium.
- **Detention** – allow adequate reaction time for the pre-oxidants and HMO prior to filtration.
- **Sand and Anthracite Filtration** – filter out the oxidized particulate. Analysis will include conventional media and IMAR media. According to Tonka Water, IMAR is a proprietary dual media formula of anthracite and sand, designed to remove iron, manganese, arsenic, and radium from groundwater supplies.

Throughout pilot study operation, AE2S adjusted the treatment train components as needed to collect additional data related to various treatment goals.

6.3 Pilot Study Investigation and Application

AE2S conducted a pilot study to treat the City's raw water supplied by Well No. 18 from July 19 through August 1, 2017. The pilot equipment ran continuously throughout the duration of the study with minor exceptions including all of July 29 and 30, when the system was offline for the weekend.

The Pilot Team used various treatment technologies to determine a treatment process train that successfully removes manganese from the raw water down to levels below the Environmental Protection Agency's (EPA) established secondary maximum contaminant level

(SMCL) for manganese of 0.05 mg/L. Additionally, treatment goals included removal of iron to levels below the SMCL of 0.3 mg/L and radium removal to concentrations below the maximum contaminant levels (MCL) of 5.0 pCi/L for combined radium and 15 pCi/L for gross alpha.

The existing raw water wells planned to serve WTP No. 5 do not have combined radium or gross alpha concentrations that exceed the MCLs. Future Well No. 21 may have high levels, so another pilot objective included further analysis on the effectiveness of preformed HMO and permanganate to remove radium. Preliminary jar testing indicated that permanganate removed combined radium to the same level as HMO, when dosed at the current concentration of HMO added at the City's other facilities that treat radium. Additionally, permanganate removed approximately 50% more gross alpha than HMO during this test. During pilot testing, the Pilot Team dosed 1.0 mg/L of HMO to determine whether a higher feed rate provides enhanced radium removal.

6.3.1 Pilot Objectives

The primary objectives of the pilot study were to determine:

1. The ability of the system to achieve effluent manganese levels of less than the 0.05 mg/L SMCL.
2. The permanganate feed rate required for optimal manganese removal.
3. The effects of aeration on the iron and manganese removal performance and the potential reduction of required chemical oxidant demands.
4. The reaction time required for optimal manganese removal.
5. The effectiveness of radium removal by permanganate and/or HMO with and without detention time.
6. Filter head loss as a function of time in service.
7. Approximate filter run length.
8. Approximate backwash settling time to aid in design of the backwash reclaim system.

6.3.2 Pilot Treatment Process Trains

The pilot study compared three process trains. During each filter run, the Pilot Team adjusted various treatment components to meet different objectives. The following lists the capabilities of each process train:

1. Chlorine, permanganate, and HMO dosed to the raw water followed by 30 minutes of detention then filtration through Iron, Manganese, Arsenic, and Radium (IMAR™) media.
2. Chlorine, permanganate, and HMO dosed to the raw water followed by 30 minutes of detention time then filtration through silica sand and anthracite media.
3. Aeration of the raw water followed by dosing chlorine, permanganate, and HMO followed by 30 minutes of detention time then filtration through silica sand and anthracite media.

Figure 6.6 depicts the pilot process diagram described above. Section 6.5.4 details each of the five filter runs completed during the pilot study.

6.3.3 Methods

6.3.3.1 *Equipment Description and Operation*

The following sections describe the equipment used to conduct the pilot study, with all equipment supplied by Tonka Water, Inc.

Aeration

The forced draft aerator included an inlet water distribution system located near the top of the chamber, random packing throughout the middle section, and a forced draft blower with an internal air flow distributor toward the bottom.

Chemical Feed

The pilot system included equipment to feed sodium hypochlorite, potassium permanganate, and HMO for each of the three process trains.

The Pilot Team verified chemical feed by bulk solution concentration checks and pump drawdown calculations to ensure accurate chemical dose control. The pilot system included Pulsafeeder Chem-Tech peristaltic metering pumps, bulk solution tanks, inline static mixers after chemical injection points, and a mixer for the HMO solution.

Detention

The two detention tanks provided time for the pre-oxidant reactions to take place prior to filtration. One tank accepted flow from Columns 1 and 2, while Column 3 had its own tank.

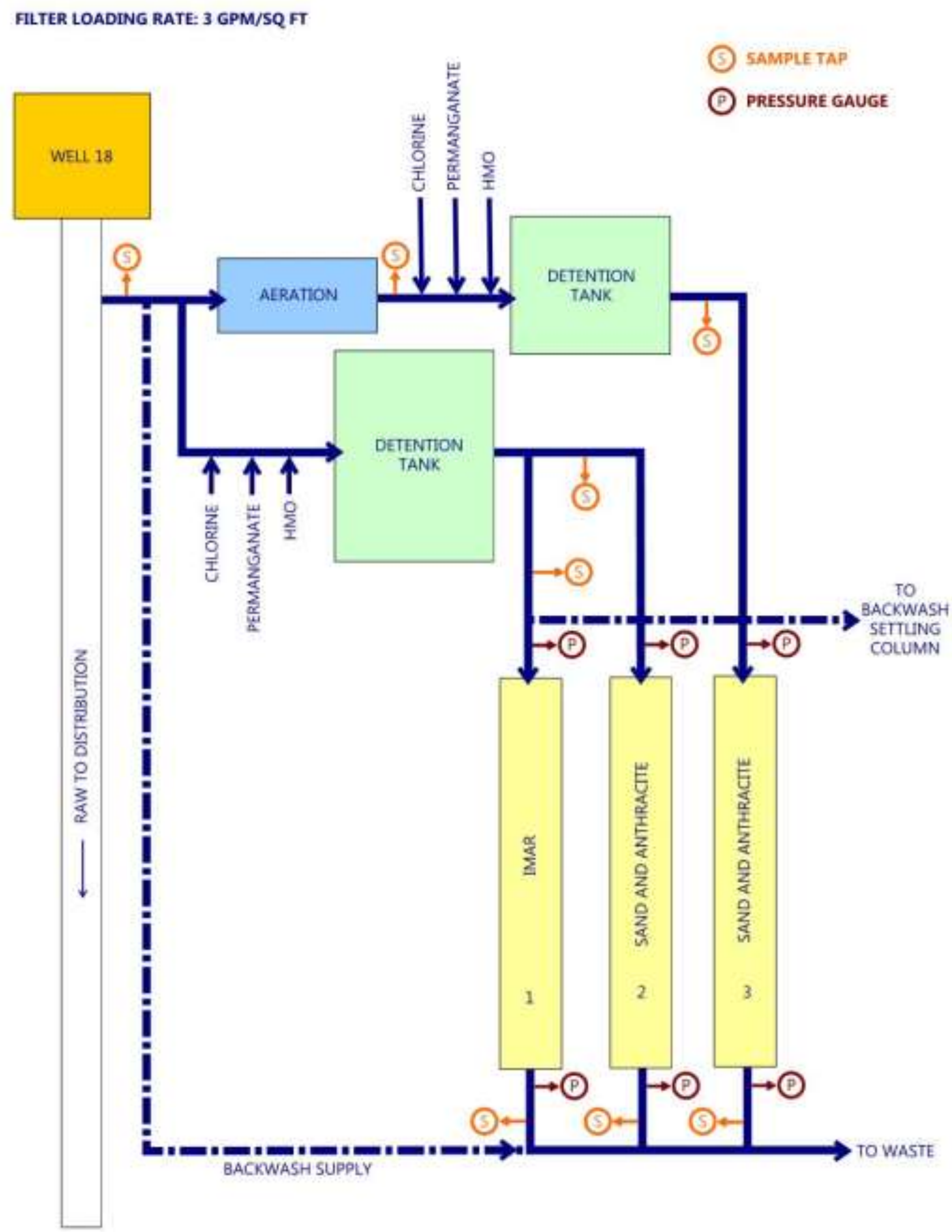

Figure 6.6 Pilot Study Process Diagram

The Pilot Team installed the flow rotameters on the influent side of the detention tank to ensure accurate metering of reaction times.

Filters

The system included three (3) filters, each eight (8) inches in diameter and approximately eight (8) feet tall. Each filter incorporates a simultaneous air/water backwash system, underdrain system, air release valve, rate control rotameters, sample taps and filter media. Each 8-inch diameter filter provided a total 0.35 ft^2 of surface area, which corresponds to a filter loading rate of 3 gpm/ft^2 when operating at 1.05 gpm. Column 1 included IMAR™ media and Columns 2 and 3 included standard dual media with 18″ silica sand (0.45 – 0.55 mm) and a 12″ cap of anthracite (0.8 – 1.0 mm). The underdrain included approximately 6″ of ¼″ by 1/8″ gravel and a 3″ layer of Torpedo Sand (0.8 – 1.2 mm).

Each filter included differential pressure gauges to monitor head loss over time. Rotameters installed on the filter effluent controlled the rate of flow through the filters. Upon startup of the system, each filter underwent a backwash cycle to remove as much of the media fines as possible and cleanse the media before beginning the first filter run. No pre-conditioning of the media prior to the pilot study took place.

Backwash rates used throughout the pilot were 3 gpm/ft^2 of water with 3 $scfm/ft^2$ of air for 10 minutes during simul-wash followed by a 2 minute air purge at a 3 gpm/ft^2 water rate. For restratification, flow increased to 10 gpm/ft^2 without air for 3 minutes for the IMAR™ media and 13-15 gpm/ft^2 without air for 3 minutes for standard sand and anthracite media.

Raw water provided by Well No. 18 entered the system at a pressure of approximately 30 to 40 psi, depending on the conditions of the well and the pressure reducing valve set point. Each flow train drew approximately 1.05 gpm. The aerator used for Column 3 maintained a small overflow of approximately 0.2 to 0.3 gpm to keep the booster pump primed. Similarly, during runs that included detention tanks, the system maintained a small overflow to adjust reaction times to 30 minutes. During detention runs, the Pilot Team adjusted chemical feed to account for this increase in flow.

Filter effluent discharged into a storage tank to use at the end of filter runs for backwashing. Excess effluent overflowed to waste.

The Pilot Team truncated filter runs based on one of the three (3) conditions:

1. Terminal head loss as defined by 200 inches of water column
2. Contaminant breakthrough – manganese greater than 0.05 mg/L or iron > 0.3 mg/L

3. Time constraints of the pilot study to meet additional objectives.

6.3.3.2 Sampling and Analysis

The Pilot Team collected raw water and filter effluent samples throughout the course of the pilot study to analyze treatment train performance. Field sampling and analysis occurred several times per day. The Team collected daily samples of raw water iron, manganese, and ammonia, filter effluent iron and manganese for laboratory analysis. Sample kits included acid preservatives for later analysis at Pace Laboratory in Minneapolis, MN. These samples verified the results of field testing.

The Pilot Team performed onsite pH, temperature, manganese, iron and ammonia measurements on the raw water. Field filter effluent analysis included iron, manganese, free ammonia, monochloramine, total chlorine, free chlorine, pH, and temperature. Additionally the Team analyzed aerator effluent and detention basin effluent iron and manganese samples to quantify performance of the pre-filter treatment technologies.

The low range manganese test used in the field has a 0.02 mg/L detection limit and a standard deviation of ±0.013 mg/L. The method used in the laboratory has a 0.0005 mg/L reporting limit, providing a more accurate result. The iron field test has a 0.02 to 3.00 mg/L detection limit, while the laboratory method has a 0.05 mg/L reporting limit. The method reporting limits are important to consider when drawing conclusions about filter performance.

6.3.4 Filter Run Descriptions

The pilot study included five (5) separate filter runs, with three (3) online process trains at a time, totaling fifteen (15) separate analyses. The following sections describe the treatment trains and chemical doses for each column and filter run.

Filter Run 1 (39 to 51 Hours)

Run 1 focused on removal of iron, manganese, and radium removal with chlorine and permanganate. The Pilot Team set a 1.2 mg/L target chlorine dose to operate the system at peak chloramination. Results of preliminary bench jar testing aided in determining this target chlorine dose. Permanganate dose varied from 0.5 to 0.7 mg/L. Detention time was limited to the seven (7) to eight (8) minutes provided in the top of each filter column. Treatment processes for each column included the following:

Column 1: Chlorine and permanganate addition followed by filtration through IMAR™ media

Column 2: Chlorine and permanganate addition followed by filtration through sand and anthracite

Column 3: Aeration followed by chlorine and permanganate addition, then filtration through sand and anthracite

Filter Run 2 (71 Hours)

Run 2 was similar to Run 1 and again focused on removal of iron, manganese, and radium removal with chlorine and permanganate. Chlorine was set at 1.2 mg/L and permanganate dose varied from 0.5 to 1.0 mg/L. Detention time was limited to that available in the top of the column. Treatment processes for each column included the following:

Column 1: Chlorine and permanganate addition followed by filtration through IMAR™ media

Column 2: Chlorine and permanganate addition followed by filtration through sand and anthracite

Column 3: Aeration followed by chlorine and permanganate addition, then filtration through sand and anthracite

Filter Run 3 (67 Hours)

Run 3 added HMO at a dose of 1.0 mg/L in addition to chlorine and permanganate to determine whether enhanced radium removal occurs with preformed HMO. Chlorine remained at 1.2 mg/L and permanganate dose varied from 0.3 to 0.5 mg/L. Detention time was limited to that available in the top of the column. Treatment processes for each column included the following:

Column 1: Chlorine, permanganate, and HMO addition followed by filtration through IMAR™ media

Column 2: Chlorine, permanganate, and HMO addition followed by filtration through sand and anthracite

Column 3: Aeration followed by chlorine, permanganate, and HMO, then filtration through sand and anthracite

Filter Run 4 (18 to 32 Hours)

To determine whether additional reaction time provides additional manganese removal, Run 4 added an additional 23 minutes of dedicated detention to provide a total 30 minute reaction period when including the 7 minutes of filter head space. Chemical set points were 1.2 mg/L for chlorine, 0.4 to 0.7 mg/L for permanganate and 1.0 mg/L for HMO. Treatment processes for each column included the following:

Column 1: Chlorine, permanganate, and HMO addition followed by 30 min. detention, then filtration through IMAR™ media

Column 2: Chlorine, permanganate, and HMO addition followed by 30 min. detention, then filtration through sand and anthracite

Column 3: Aeration followed by chlorine, permanganate, and HMO, then 30 min. of detention, and finally, filtration through sand and anthracite

Filter Run 5 (24 Hours)

Run 5 mimicked Run 4 and again provided a total 30 minutes of detention time to allow chemical reactions to take place. Chemical set points were 1.2 mg/L for chlorine, 0.2 to 0.5 mg/L for permanganate and 1.0 mg/L for HMO. This run also had portions of runtime without permanganate feed or HMO feed to monitor the systems performance without one or the other chemical prior to completing the pilot study. Treatment processes for each column included the following:

Column 1: Chlorine, permanganate, and HMO addition followed by 30 min. detention, then filtration through IMAR™ media

Column 2: Chlorine, permanganate, and HMO addition followed by 30 min. detention, then filtration through sand and anthracite

Column 3: Aeration followed by chlorine, permanganate, and HMO, then 30 min. of detention, and finally, filtration through sand and anthracite

Table 6.5 summarizes the filter run operational parameters throughout the pilot study.

Table 6.5 Pilot Study Filter Run Operational Parameters

<table>
<tr><th>Run</th><th>Column</th><th>Permanganate Dose (mg/L)</th><th>HMO Dose (mg/L)</th><th>Chlorine Dose (mg/L)</th><th>Filter Loading Rate (gpm/ft^2)</th><th>Detention Time (minutes)</th><th>Total Run Time (hours)</th></tr>
<tr><td rowspan="3">1</td><td>1</td><td rowspan="3">0.5, 0.6, 0.7</td><td rowspan="3">0</td><td rowspan="15">1.2</td><td rowspan="15">3.0</td><td rowspan="9">7-8</td><td rowspan="2">51</td></tr>
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<tr><td>3</td><td>39</td></tr>
<tr><td rowspan="3">2</td><td>1</td><td rowspan="3">0.5, 0.7, 0.8, 1.0</td><td rowspan="3">0</td><td rowspan="3">71</td></tr>
<tr><td>2</td></tr>
<tr><td>3</td></tr>
<tr><td rowspan="3">3</td><td>1</td><td rowspan="3">0.3, 0.4, 0.5</td><td rowspan="3">1.0</td><td rowspan="3">67</td></tr>
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<tr><td>3</td></tr>
<tr><td rowspan="3">4</td><td>1</td><td rowspan="3">0.4, 0.5, 0.6, 0.7</td><td rowspan="3">1.0</td><td rowspan="6">30</td><td>18</td></tr>
<tr><td>2</td><td>18</td></tr>
<tr><td>3</td><td>32</td></tr>
<tr><td rowspan="3">5</td><td>1</td><td rowspan="3">0.2, 0.3, 0.5</td><td rowspan="3">0, 1.0</td><td rowspan="3">24</td></tr>
<tr><td>2</td></tr>
<tr><td>3</td></tr>
</table>

6.3.5 Pilot Study Results

The following sections present the results and discuss the performance of the various treatment technologies analyzed throughout the pilot study. ***Appendix D*** summarizes the field data collected throughout the pilot study, ***Appendix E*** provides the chemical feed system calibration results, and ***Appendix F*** includes the pilot study laboratory data from Pace Analytical and Eurofins Laboratory.

6.3.5.1 *Raw Water Characteristics*

Pilot study raw water came from Well No. 18. Table 6.6 summarizes the average field and laboratory confirmed raw water characteristics observed throughout the pilot study.

Table 6.6 Average Raw Water Characteristics of Well No. 18 during Piloting

	pH	Temp (˚C)	Iron (mg/L)	Manganese (mg/L)	Ammonia (mg/L)
Field	7.54	12.0	0.37	0.202	0.24
Laboratory	-	-	0.390	0.168	0.26

6.3.5.2 *Aerator Performance*

The two treatment methods used during piloting for iron removal were physical oxidation by aeration and chemical pre-oxidation by chlorine and permanganate addition. Column 3 was the only process train with aeration. Table 6.7 summarizes the iron and manganese results for the aerator effluent compared to raw water concentrations. The Pilot Team filtered the aerator effluent samples through a 0.45 micron filter paper prior to analysis to measure only un-oxidized iron and manganese residual. All data presented in Table 6.7 is from field measurements. The Pilot Team did not collect any aerator effluent samples for lab analysis.

Table 6.7 Summary of Aerator Effluent Iron and Manganese Removal Results

	Iron (mg/L)				
Run	1	2	3	4	5
Raw Water	0.34 – 0.40 (0.37 avg)	0.33 – 0.37 (0.35 avg)	0.34 – 0.50 (0.38 avg)	0.35 – 0.40 (0.38 avg)	0.38
Aerator Effluent	0.05 – 0.23 (0.14 avg)	0.23	0.02 – 0.08 (0.05 avg)	No Sample	0.05
	Manganese (mg/L)				
Run	1	2	3	4	5
Raw Water	0.192 – 0.226 (0.205 avg)	0.183 – 0.192 (0.188 avg)	0.188 – 0.230 (0.205 avg)	0.190 – 0.202 (0.196 avg)	0.199
Aerator Effluent	0.217 – 0.230 (0.225 avg)	0.214	0.181 – 0.223 (0.204 avg)	No Sample	0.200

On average, the aerator removed approximately 73% of the raw water iron when looking at raw water and aerator effluent results over the duration of the entire pilot study. Some samples indicated iron removal to below the field analysis method detection limit of 0.02 mg/L. All aerator effluent samples resulted in iron residual concentrations below the 0.3 mg/L SMCL. Aeration was not successful at removing manganese from the raw water, which is typical.

6.3.5.3 *Iron Removal*

All filter effluent samples taken throughout the pilot study resulted in iron residual concentrations below the 0.3 mg/L SMCL, indicating that the pre-oxidation processes of aeration and/or chlorine and permanganate addition were effective means of removing iron from the raw water of Well No. 18 in Edina. Raw water iron averaged 0.37 mg/L in the field and 0.39 mg/L in lab samples. All three process trains reduced this value to <0.02 mg/L on average in the field and <0.05 mg/L for lab samples.

As discussed previously, using aeration ahead of chemical pre-oxidant addition for Column 3 produced similar iron removal results to Column 1 and 2 when comparing filter effluent results. The only difference in performance between the two process trains was the required chlorine dose. The Pilot Team maintained the same dose between all three columns, but differences in monochloramine resulted. Column 3 filter effluent had higher monochloramine residuals throughout the pilot study because the aerator oxidizes iron prior to chemical injection, reducing the demand iron has on chlorine. Higher chlorine residual post-aeration allowed more chloramine formation to occur between the raw water ammonia and chlorine at the 1.2 mg/L target dose.

Table 6.8 summarizes the iron results throughout each run of the pilot study and includes both field and laboratory result ranges and averages.

Table 6.8 Summary of Pilot Study Iron Removal Results

Run	Analysis Type	Iron (mg/L)			
		Raw	Column 1 Effluent	Column 2 Effluent	Column 3 Effluent
1	Field	0.34 – 0.40 (0.37 avg)	<0.02 – 0.05 (<0.02 avg)	<0.02 – 0.05 (<0.02 avg)	<0.02 – 0.05 (0.02 avg)
	Lab	0.383 – 0.401 (0.391 avg)	<0.05	<0.05	<0.05
2	Field	0.33 – 0.37 (0.35 avg)	<0.02 – 0.05 (0.03 avg)	<0.02 – 0.07 (0.04 avg)	<0.02 – 0.06 (0.03 avg)
	Lab	0.390	<0.05	<0.05	<0.05
3	Field	0.34 – 0.50 (0.38 avg)	<0.02 – 0.04 (<0.02 avg)	<0.02 – 0.03 (<0.02 avg)	<0.02 – 0.03 (<0.02 avg)
	Lab	0.380 – 0.388 (0.384 avg)	<0.05	<0.05	<0.05
4	Field	0.35 – 0.40 (0.38 avg)	<0.02	<0.02	<0.02
	Lab	-	-	-	-
5	Field	0.38	<0.02	<0.02	<0.02
	Lab	0.396	<0.05	<0.05	<0.05

When considering the filter media type and performance differences, there was no significant difference between IMAR™ and standard sand and anthracite media when looking at iron removal.

Comparing results between filter runs, the first two runs consistently had the highest iron results, but averages were still at or below detection limits. Minor removal improvement occurred during Run 3, which was similar to Runs 1 and 2, but included HMO addition. The last two filter runs added detention time and all field samples resulted in iron residuals below the field detection limit of 0.02 mg/L. This indicates that the additional reaction time provided by the detention tanks may have resulted in a slight improvement in the iron removal consistency. The Pilot Team reduced iron sampling frequency after the first three filter runs after initial results indicated that the treatment processes effectively removed iron to the treatment goals of WTP No. 5. Filter runs 4 and 5 focused on manganese and radium removal. Note that it is common that filter performance improves with each consecutive filter run due to removal of media fines with each backwash and media conditioning that occurs over time.

Figure 6.7 (a) through (e) depict the iron results throughout each of the filter runs. The legend at the bottom of the page corresponds to all five (5) graphs.

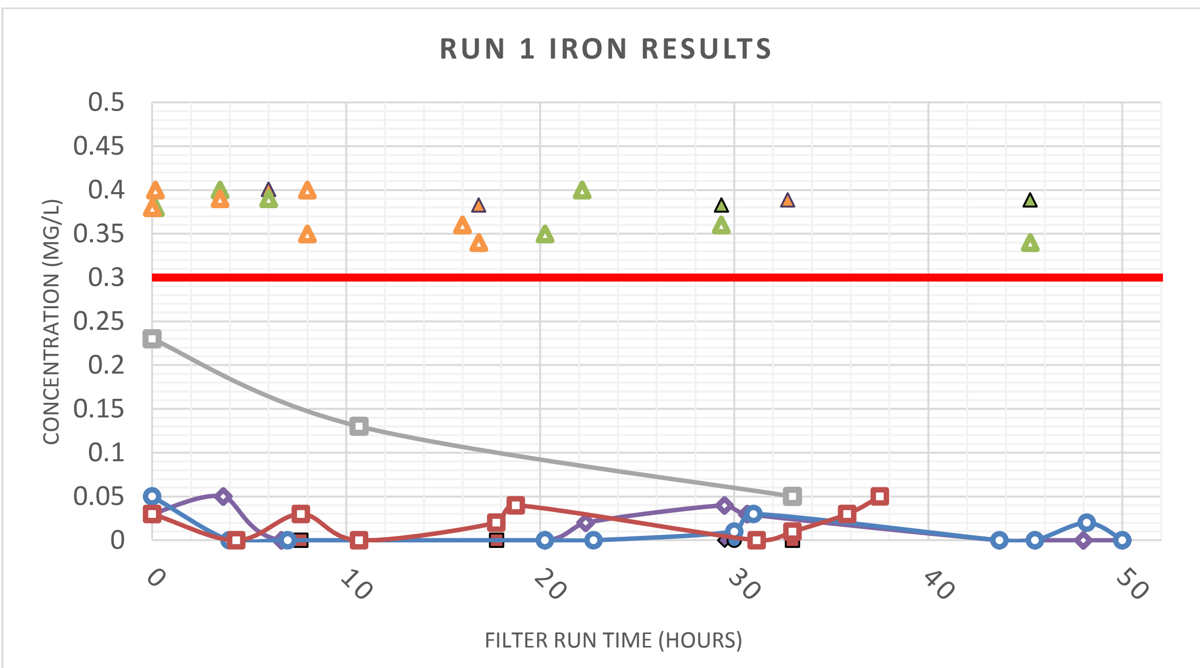

Figure 6.7 (a)

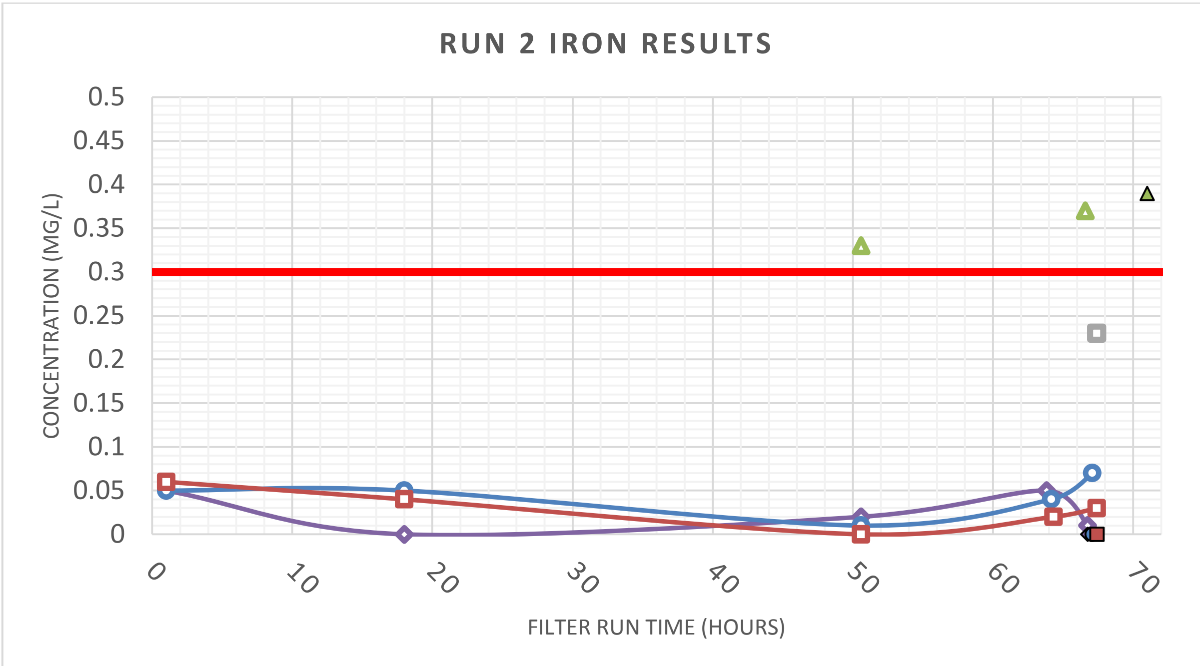

Figure 6.7 (b)

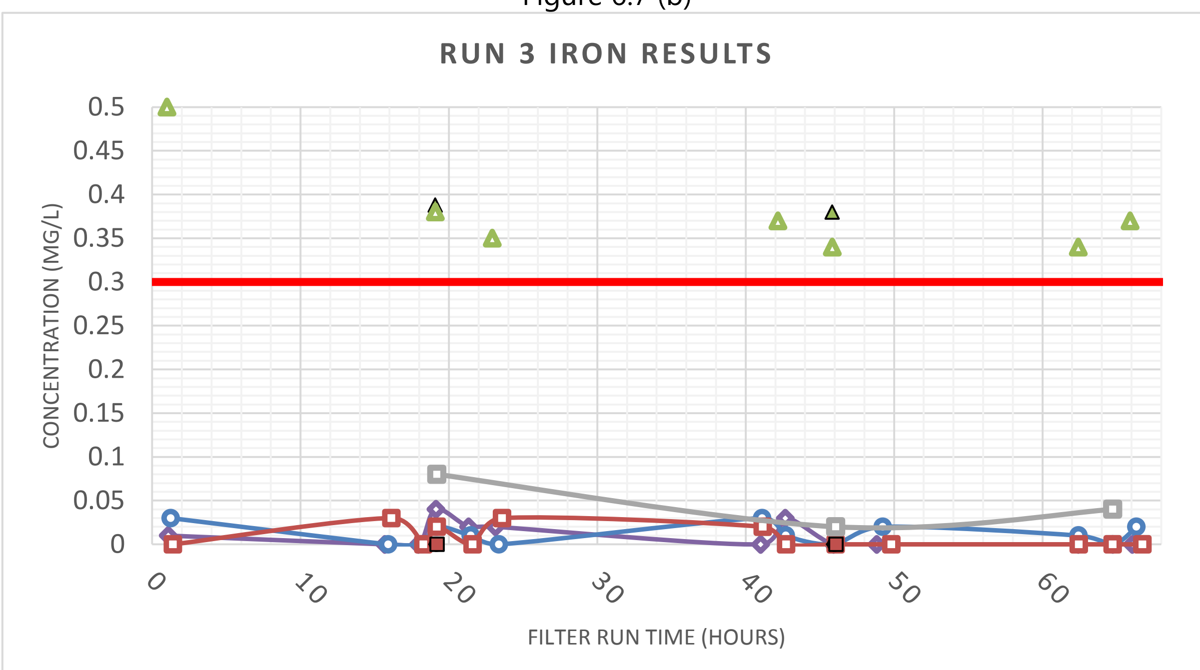

Figure 6.7 (c)

Column 1 and 2 Raw Water (Lab)
Column 3 Raw Water (Lab)
Column 1 (Lab)
Column 2 (Lab)
Column 3 (Lab)
Column 1 and 2 Raw Water (Field)
Column 3 Raw Water (Field)
Column 1 (Field)
Column 2 (Field)
Column 3 (Field)
Column 3 Aerator Effluent (Field)
Iron SMCL

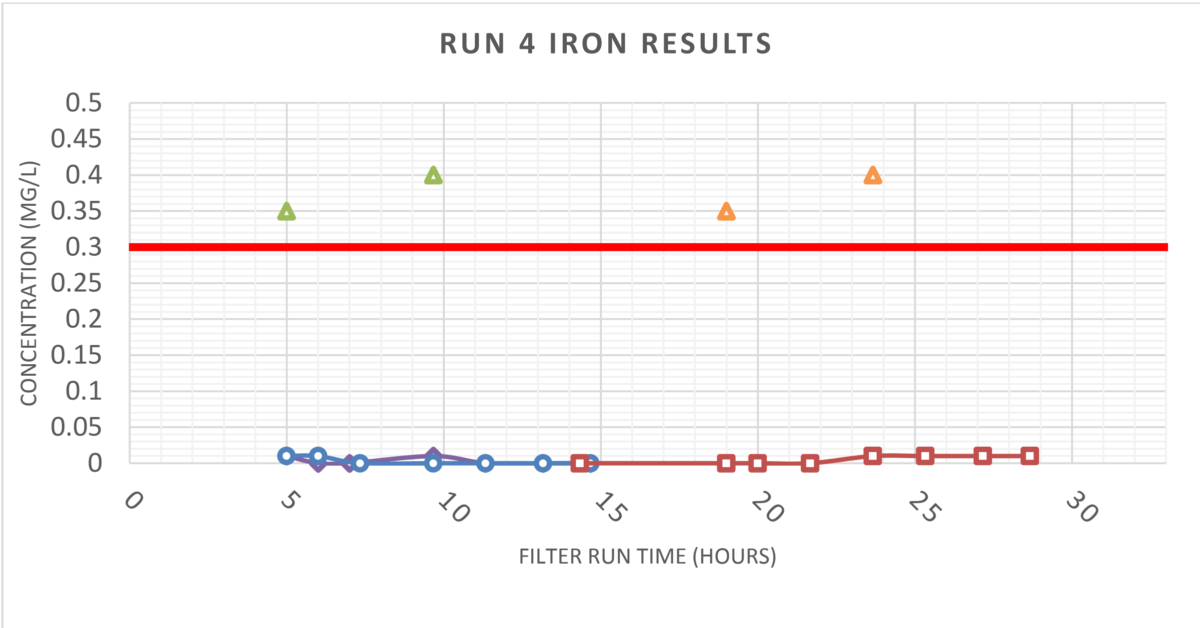

Figure 6.7 (d)

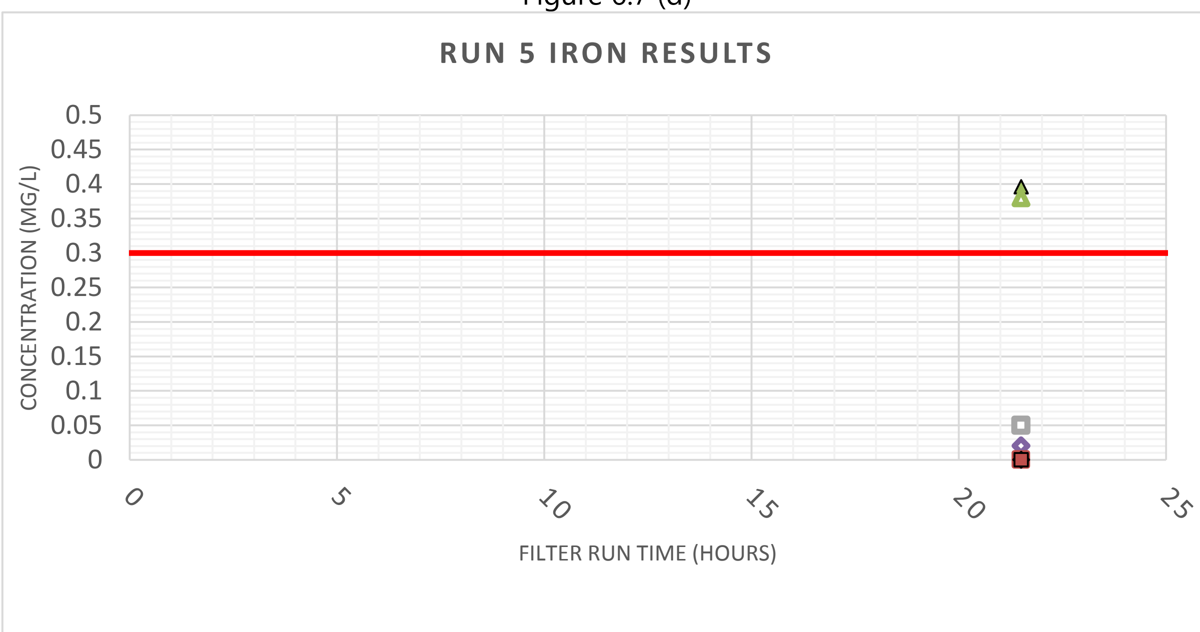

Figure 6.7 (e)

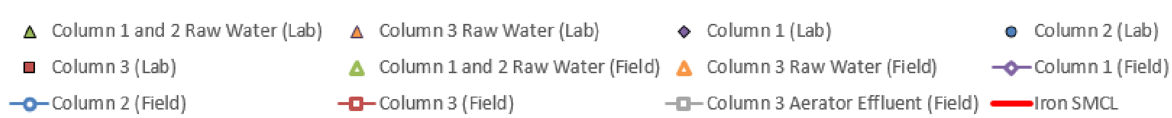

Figure 6.7 Iron Results for Each Filter Run

6.3.5.4 *Manganese Removal*

Hypochlorite and permanganate addition followed by filtration using a sand and anthracite media, whether it be IMAR™ or standard dual media with 18" silica sand (0.45 – 0.55 mm) and a 12" cap of anthracite (0.8 – 1.0 mm), was an effective means of removing manganese from the raw water when operating at the optimum permanganate dose. Raw water manganese averaged 0.202 mg/L in the field and 0.168 mg/L in lab samples. Column 1 reduced this value to 0.042 mg/L on average in the field and 0.0080 mg/L for lab samples. Column 2 filter effluent was on average 0.044 mg/L in the field and 0.0116 mg/L in the lab. Averages for Column 3 were 0.037 mg/L in the field and 0.0060 mg/L in the lab. All averages are below the 0.05 mg/L SMCL for manganese. All laboratory results are less than the samples collected in the field, likely due to field method detection limit restrictions. Laboratory results indicate better manganese removal than initially assumed from field results.

Table 6.9 summarizes the manganese results throughout each run of the pilot study and includes both field and laboratory result ranges and averages.

Table 6.9 Summary of Pilot Study Manganese Removal Results

Run	Analysis Type	Manganese (mg/L)			
		Raw	Column 1 Effluent	Column 2 Effluent	Column 3 Effluent
1	Field	0.192 – 0.226 (0.205 avg)	0.028 – 0.084 (0.049 avg)	0.026 – 0.067 (0.045 avg)	0.026 – 0.082 (0.046 avg)
	Lab	0.166 – 0.180 (0.171 avg)	0.0030 – 0.0088 (0.0055 avg)	0.0032 – 0.0094 (0.0055 avg)	0.0032 – 0.0088 (0.0055 avg)
2	Field	0.183 – 0.192 (0.188 avg)	0.022 – 0.074 (0.044 avg)	0.026 – 0.076 (0.053 avg)	<0.02 – 0.066 (0.045 avg)
	Lab	0.166	0.0287	0.0493	0.0177
3	Field	0.188 – 0.230 (0.205 avg)	<0.02 – 0.091 (0.041 avg)	<0.02 – 0.134 (0.051 avg)	<0.02 – 0.055 (0.038 avg)
	Lab	0.164 – 0.166 (0.165 avg)	0.0023 – 0.0037 (0.0030 avg)	0.0044 – 0.0049 (0.0047 avg)	0.0021 – 0.0024 (0.0023 avg)
4	Field	0.190 – 0.202 (0.196 avg)	<0.02 – 0.061 (0.040 avg)	<0.02 – 0.059 (0.041 avg)	<0.02 – 0.047 (0.026 avg)
	Lab	-	-	-	-
5	Field	0.199	0.032 – 0.045 (0.038 avg)	0.024 – 0.041 (0.032 avg)	0.026 – 0.041 (0.034 avg)
	Lab	0.169	0.0049	0.0058	0.0035

For laboratory samples, a single high effluent manganese concentration set occurred when permanganate dose was set at 0.8 mg/L during Run 2. When eliminating this data point from the sample set, lab filter effluent averages reduce to 0.0046 mg/L for Column 1, 0.0053 mg/L for Column 2, and 0.0041 mg/L for Column 3. This outlier was likely the result of excess permanganate. Laboratory samples, aside from the outlier, were on average 89% lower for Column 1 and 87% lower for Column 2 and Column 3. Based on these results, there is strong evidence that all three treatment processes will remove manganese levels below the field method detection limit of <0.02 mg/L and meet the proposed facilities treatment target goals.

Analysis of manganese results is more complicated than that of iron because the filter effluent residual may include a combination of un-oxidized raw water manganese and excess permanganate residual. Excess permanganate may lead to elevated manganese concentrations, so optimizing permanganate dose is essential for peak filter performance. Manganese breakthrough is also an indication of when to backwash the filter.

Table 6.10 summarizes the variation in average manganese removal with varying permanganate dose during each filter run. This table only presents field collected data. As noted previously, laboratory analyzed sample results are consistently less (87 to 89%) than the field sample results. The field sample sets are larger and provide more data points to calculate averages that are more representative.

Table 6.10 Summary of Average Manganese Residual with Varying Permanganate Dose

<table>
<tr><th rowspan="2">Run</th><th rowspan="2">Permanganate Dose (mg/L)</th><th colspan="4">Manganese (mg/L)</th></tr>
<tr><th>Raw</th><th>Column 1 Effluent</th><th>Column 2 Effluent</th><th>Column 3 Effluent</th></tr>
<tr><td rowspan="3">1</td><td>0.5</td><td rowspan="3">0.205</td><td>0.052</td><td>0.051</td><td>0.053</td></tr>
<tr><td>0.6</td><td>0.044</td><td>0.037</td><td>0.041</td></tr>
<tr><td>0.7</td><td>0.050</td><td>0.045</td><td>0.046</td></tr>
<tr><td rowspan="4">2</td><td>0.5</td><td rowspan="4">0.188</td><td>0.052</td><td>0.076</td><td>-</td></tr>
<tr><td>0.7</td><td>0.034</td><td>0.046</td><td>0.042</td></tr>
<tr><td>0.8</td><td>0.045</td><td>0.056</td><td>0.058</td></tr>
<tr><td>1.0</td><td>0.074</td><td>-</td><td>-</td></tr>
<tr><td rowspan="3">3</td><td>0.3</td><td rowspan="3">0.205</td><td>0.035</td><td>0.034</td><td>0.044</td></tr>
<tr><td>0.4</td><td>0.031</td><td>0.035</td><td>0.040</td></tr>
<tr><td>0.5</td><td>0.063</td><td>0.088</td><td>0.031</td></tr>
<tr><td rowspan="4">4</td><td>0.4</td><td rowspan="4">0.196</td><td>0.061</td><td>0.059</td><td>0.044</td></tr>
<tr><td>0.5</td><td>0.032</td><td>0.032</td><td>0.018</td></tr>
<tr><td>0.6</td><td>0.059</td><td>0.058</td><td>0.031</td></tr>
<tr><td>0.7</td><td>0.042</td><td>0.049</td><td>0.034</td></tr>
<tr><td rowspan="3">5</td><td>0.2</td><td rowspan="3">0.199</td><td>0.040</td><td>0.031</td><td>0.032</td></tr>
<tr><td>0.3</td><td>0.034</td><td>0.032</td><td>0.033</td></tr>
<tr><td>0.5</td><td>0.037</td><td>0.033</td><td>0.036</td></tr>
</table>

As shown in Table 6.10, filter effluent manganese residuals varied over the course of the pilot study, even at similar permanganate doses between different filter runs. Recall that operational parameters changed with each filter run. Factors that affect manganese removal include, but are not limited to, overfeeding permanganate, extended filter run time causing buildup of manganese floc eventually leading to manganese breakthrough, and potential interference of HMO on permanganate dose. Ultimately, the pilot study met the goal of removing manganese to levels below the SMCL of 0.05 mg/L. Further optimization of permanganate dose with chlorine and possible HMO feed will occur during final design and during full-scale plant operation.

Major trends realized throughout the filter runs related to manganese removal include:

1. Optimum manganese dose ranged from 0.4 to 0.7 mg/L depending on other online chemical feed, available detention time, and filter run time.
2. At a permanganate dose of >0.8 mg/L, increases in manganese residual resulted, indicating excess permanganate. The highest reported laboratory results occurred at the 0.8 mg/L dose for all three columns. Note that effluent water was never tinted pink at any feed rate analyzed throughout the pilot study.
3. Overall, all three filter effluent averages for field samples were below the 0.05 mg/L SMCL for manganese. All laboratory results were lower than lab samples, indicating better filter performance then initially assumed in the field.
4. With detention, filter effluent results were more consistent, indicating that the extended reaction time provides a buffer when making changes to chemical feed.
5. The lowest observed manganese residuals in the field occurred during Run 4 with permanganate set at 0.5 mg/L and 30 minutes of detention. The Pilot Team did not collect lab samples with this set to confirm the low field results because filter run time was minimal at the time of these low residuals.
6. The lowest laboratory manganese residual for Column 1 and Column 3 occurred during Run 3 with permanganate set at 0.4 mg/L and for Column 2 during Run 1 with permanganate set at 0.6 mg/L, both with only 7-8 minutes of detention.
7. Run 2 and Run 3 indicate potential manganese breakthrough after 60 hours of filter run time. Excess permanganate may have also caused a portion of the increase in residual because the Pilot Team tried increasing permanganate dose to understand the system’s response to chemical changes.
8. No obvious trends in filter performance resulted between varying media types.

Figure 6.8 (a) through (e) depict the manganese results throughout each of the filter runs. The legend at the bottom of each page corresponds to all five (5) graphs.

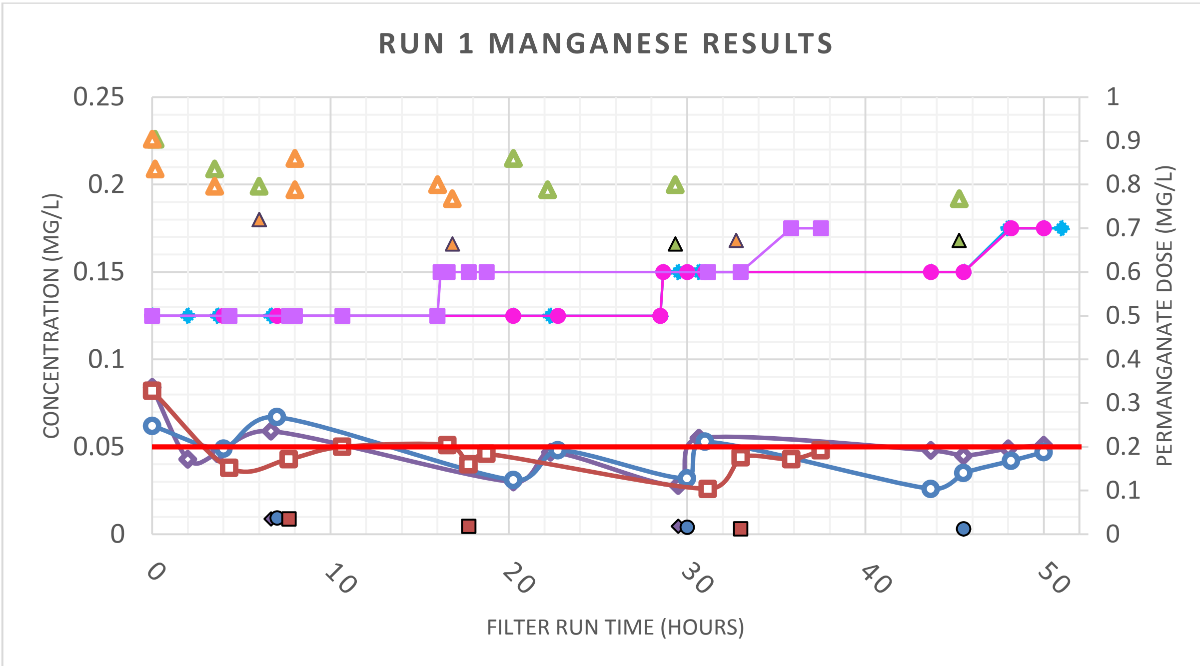

Figure 6.8 (a)

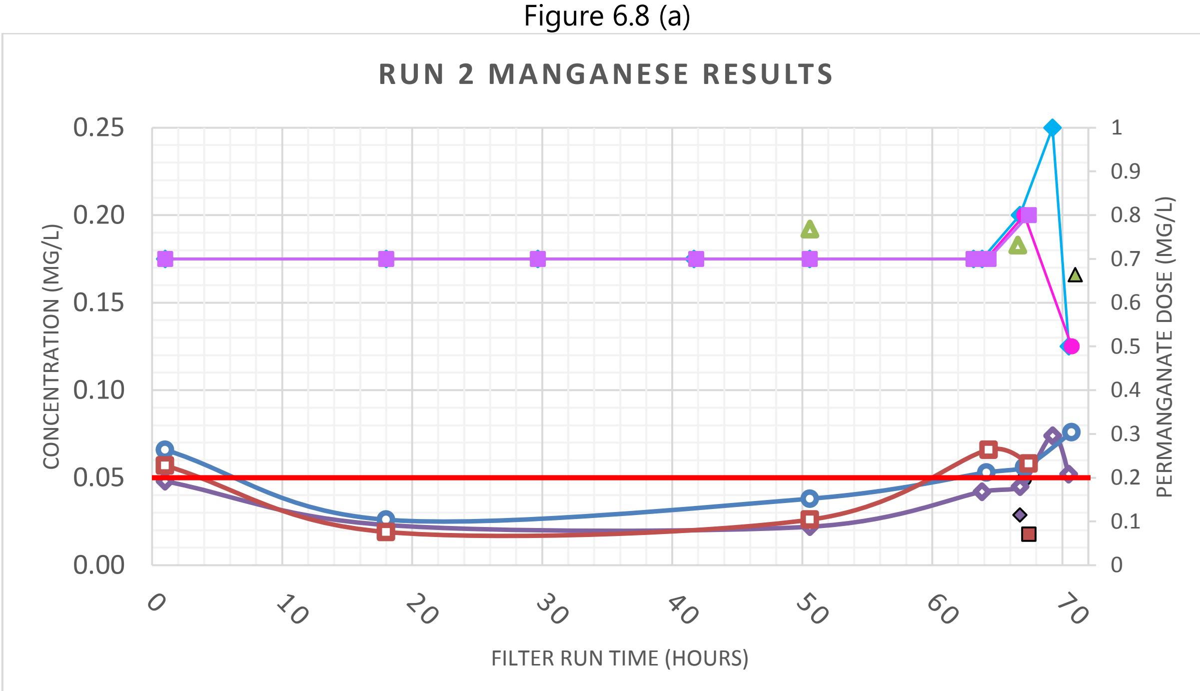

Figure 6.8 (b)

Column 1 and 2 Raw Water (Lab)
Column 3 Raw Water (Lab)
Column 1 (Lab)
Column 2 (Lab)
Column 3 (Lab)
Column 1 and 2 Raw Water (Field)
Column 3 Raw Water (Field)
Column 1 (Field)
Column 2 (Field)
Column 3 (Field)
Manganese SMCL
Column 1 Permanganate Dose
Column 2 Permanganate Dose
Column 3 Permanganate Dose

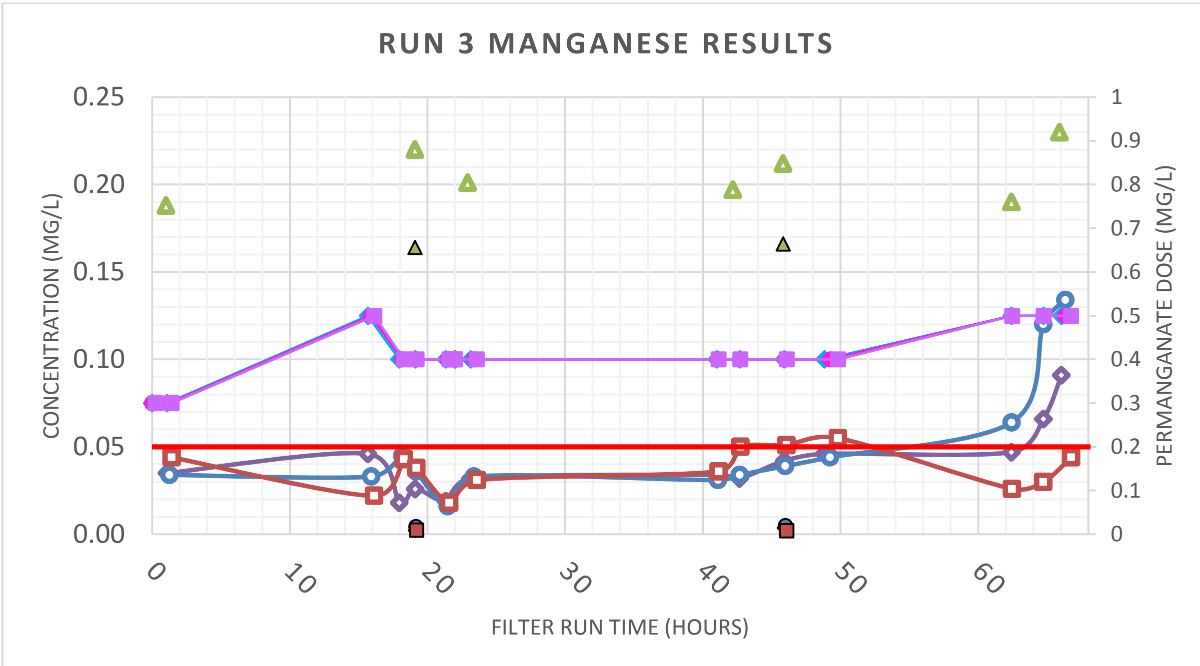

Figure 6.8 (c)

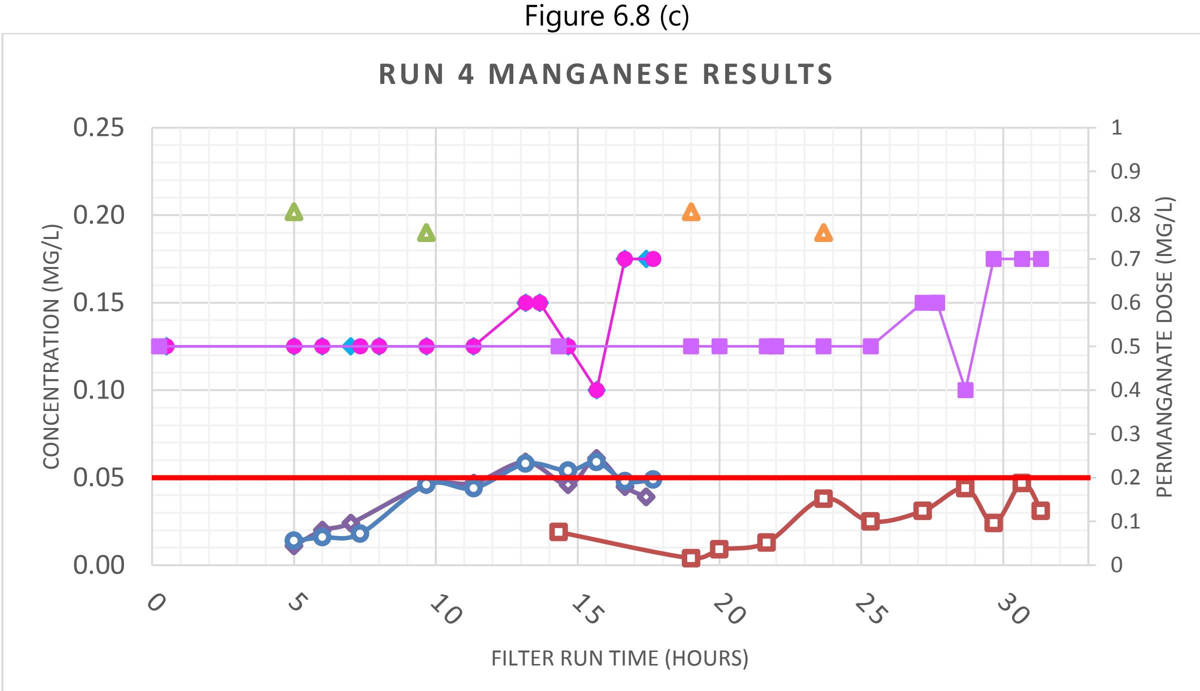

Figure 6.8 (d)

Column 1 and 2 Raw Water (Lab) | Column 3 Raw Water (Lab) | Column 1 (Lab) | Column 2 (Lab)
Column 3 (Lab) | Column 1 and 2 Raw Water (Field) | Column 3 Raw Water (Field) | Column 1 (Field)
Column 2 (Field) | Column 3 (Field) | Manganese SMCL | Column 1 Permanganate Dose
Column 2 Permanganate Dose | Column 3 Permanganate Dose

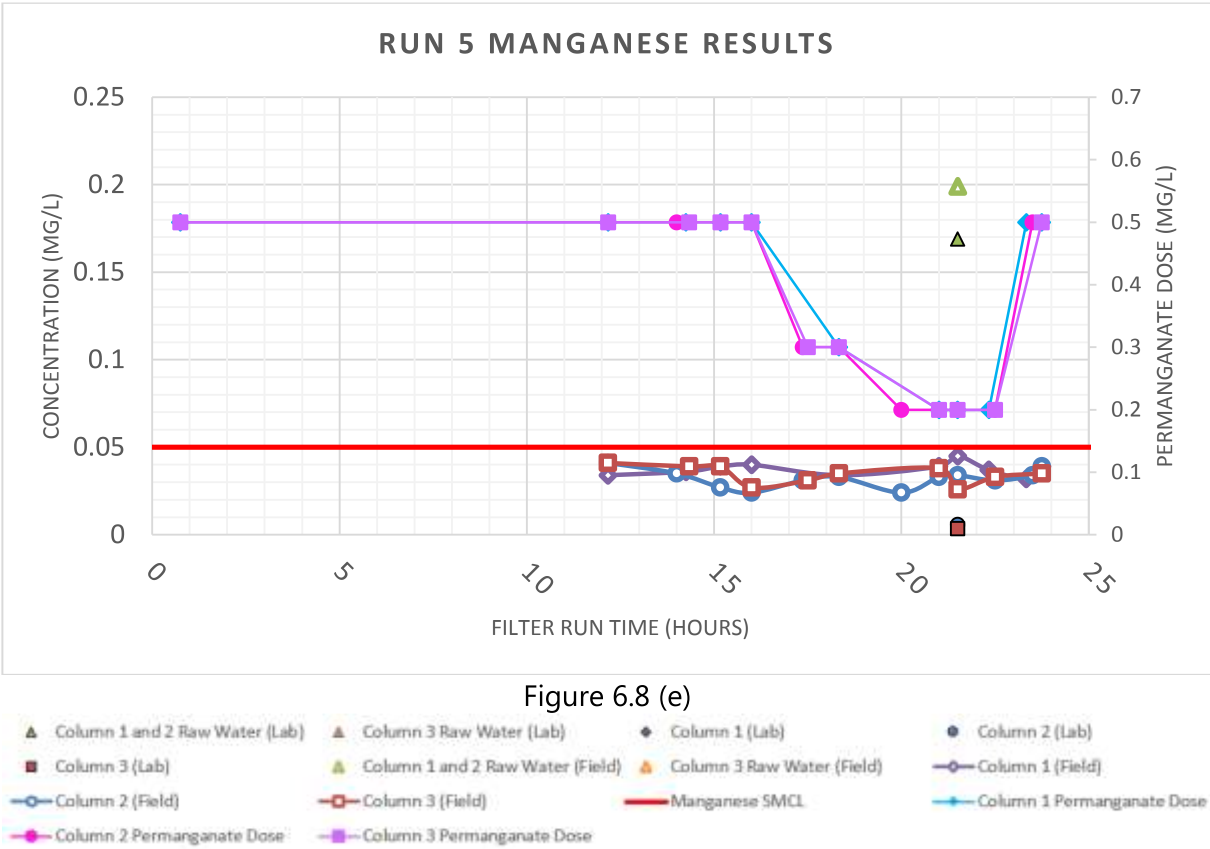

Figure 6.8 (e)

Figure 6.8 Manganese Results for Each Filter Run

6.3.5.5 *Radium Removal*

The Pilot Team took three sets of radium samples and sent them to Eurofins Laboratory for analysis to confirm the effectiveness of permanganate alone to remove radionuclides, whether a higher HMO dose will remove more radium, and review impacts of detention time on radium removal. Each sample set included a raw water sample, a Column 1 effluent sample, and a Column 2 effluent sample. This allows comparison of the two filter media types used for piloting.

During preliminary bench scale testing, the results indicated that permanganate removed gross alpha and combined radium to some degree at a 0.5 mg/L dose. The HMO dose analyzed during bench scale testing was 0.2 mg/L. These samples underwent a 30 minute reaction period after chemical addition and prior to filtration. As part of the pilot study, AE2S took the following sample sets:

1. Permanganate dose of 0.5 mg/L with only 7 to 8 minutes of detention time,

2. Permanganate dose of 0.4 mg/L, HMO dose of 1.0 mg/L with only 7 to 8 minutes of detention time, and;
3. Permanganate dose of 0.3 mg/L, HMO dose of 1.0 mg/L with 30 minutes of detention time.

Table 6.11 summarizes the results of the pilot study radionuclide testing conducted on the raw water from Well No. 18.

Table 6.11 Pilot Study Radium Removal for Well No. 18

Sample Set	Sample ID	Gross Alpha (pCi/L)	Radium-226 (pCi/L)	Radium-228 (pCi/L)	Combined Radium (pCi/L)
1	Raw Water	6.2 ± 2.0	2.9 ± 0.5	2.3 ± 0.6	5.2 ± 0.7
	Column 1 Effluent	3.5 ± 1.7	1.8 ± 0.6	1.2 ± 0.5	3.0 ± 0.7
	Column 2 Effluent	6.0 ± 2.1	1.9 ± 0.4	1.5 ± 0.5	3.4 ± 0.6
2	Raw Water	12.2 ± 2.6	2.9 ± 0.4	2.4 ± 0.6	5.3 ± 0.8
	Column 1 Effluent	4.0 ± 1.7	0.67 ± 0.22	0.68 ± 0.52	1.35 ± 0.56
	Column 2 Effluent	2.9 ± 1.4	0.72 ± 0.23	2.1 ± 0.7	2.82 ± 0.70
3	Raw Water	6.9 ± 2.1	2.9 ± 0.6	-2.8 ± 0.7	2.9 ± 0.9
	Column 1 Effluent	0.76 ± 1.20	0.49 ± 0.19	0.38 ± 0.59	0.87 ± 0.62
	Column 2 Effluent	3.0 ± 2.0	0.55 ± 0.19	0.69 ± 0.63	1.24 ± 0.66

Note that the negative reported value for raw water Radium-228 during the third sample set indicates that the radioactivity counted on the sample was slightly less than the background correction used to "zero" the instrument, according to Eurofins Laboratories.

Table 6.12 summarizes the average percent removal of the radionuclides during the pilot study.

Table 6.12 Average Percent Removal of Radionuclide during Pilot Testing

Sample Set	Sample ID	Gross Alpha (%)	Radium-226 (%)	Radium-228 (%)	Combined Radium (%)
1	Column 1 Effluent	46	39	49	43
	Column 2 Effluent	4	35	36	35
2	Column 1 Effluent	58	77	74	75
	Column 2 Effluent	70	76	14	47
3	Column 1 Effluent	93	84	NA	73
	Column 2 Effluent	59	81	NA	59

Based on the data presented in Table 6.12, it appears that gross alpha and combined radium removal increased with addition of HMO compared to only permanganate. The permanganate only sample set indicates permanganate alone removes a portion of the gross alpha and combined radium but not consistently to the desired extent for meeting the treatment goals of WTP No. 5. Additional removal benefits resulted with the addition of extended detention time.

Due to the variability of radionuclide concentrations found during pilot testing, another metric of comparison analyzed is the resulting percentage below the regulated MCLs of each filter effluent sample. The regulated MCL for gross alpha is 15.0 pCi/L and for combined radium is 5.0 pCi/L. Recall that the treatment goal for WTP No. 5 is removal of gross alpha and combined radium to at least half of the MCL. Table 6.13 summarizes these results.

Table 6.13 Result Percentage below the Regulated MCL

Sample Set	Sample ID	Gross Alpha (%)	Combined Radium (%)
1	Column 1 Effluent	77	40
	Column 2 Effluent	60	32
2	Column 1 Effluent	73	73
	Column 2 Effluent	81	44
3	Column 1 Effluent	95	83
	Column 2 Effluent	80	75

This data presents findings that indicate permanganate alone removes radionuclides to some extent for both analyzed filter media types. Then, consistent or more removal occurs with addition of HMO. The lowest resulting radionuclides occurred with addition of HMO, permanganate, and extended detention time.

While IMAR™ removed more combined radium during piloting, the increased filter head loss and lack of additional iron and manganese removal with the media, compared to conventional sand and anthracite, makes IMAR™ less desirable for use in the full scale facility. Refer to Section 6.3.5.7 for additional information regarding filter head loss and run times. Conventional media removed iron, manganese, and radium below the WTP No. 5 treatment goals with HMO addition and extended detention time. For this reason, the Project Team believes the facility should include equipment for HMO addition as part of the initial treatment train for the proposed facility. The Project Team recommends that the City optimize the HMO dose required to reach the treatment goal of half the established MCL to minimize costs of operating the HMO feed system.

It is also important to note that two of the raw water samples resulted in combined radium concentrations above the 5.0 pCi/L MCL for Well No. 18. This may be the result of extended operation of the well, indicating that raw water radionuclides may increase over time. This provides more evidence that inclusion of an HMO feed system is a responsible decision for WTP No. 5.

6.3.5.6 *Ammonia*

Raw water ammonia levels stayed consistent throughout the pilot study with a field average of 0.24 mg/L and a laboratory average of 0.26 mg/L. The pilot study targeted peak chloramination with a target chlorine dose of 1.2 mg/L set based on preliminary bench breakpoint chlorination analyses. The Pilot Team analyzed filter effluent total chlorine, free chlorine, free ammonia, and monochloramine residuals at least daily to gauge the maximum chloramine residual Well No. 18's raw water ammonia produces.

By keeping the target feed rate consistent at 1.2 mg/L for all three columns, the Pilot Team monitored the additional chlorine demand that iron has on the raw water. This simply equaled the total chlorine of Column 3 effluent minus the total chlorine of Column 1 or 2 effluent. On average, this was approximately 0.1 mg/L total chlorine. The amount of chlorine savings realized by the aerator on an annual basis is not significant. Free ammonia residuals above 0.01 mg/L, or the minimum detection limit of the field test method, indicate additional chloramine formation is available with addition of more chlorine.

Table 6.12 summarizes the ammonia, total chlorine, and monochloramine results for the raw water and filter column effluent samples. The Pilot Team only collected raw water samples for laboratory analysis because the lab's analysis method for ammonia measures total ammonia, including chloramines. All raw water and filter column effluent results would have the same result, providing no substantial data for use in pilot study result analysis. Instead, the Team conducted a field method periodically to measure free ammonia and monochloramine. Total chlorine and monochloramine should be similar while operating at peak chloramination. All free chlorine samples were non-detects as expected.

Table 6.14 Summary of Pilot Study Chloramination Results

		Ammonia (mg/L) – Total for Raw Water, Free for Column Effluent				
Filter Run		1	2	3	4	5
Raw Water	Field	0.22 – 0.24 (0.23 avg)	0.24	0.22 – 0.23 (0.23 avg)	-	0.27
	Lab	0.24 – 0.25 (0.25 avg)	0.26	0.27 – 0.28 (0.28 avg)	-	0.26
Column 1	Field	<0.01 – 0.08 (0.03 avg)	<0.01	0.01 – 0.03 (0.02 avg)	-	-
Column 2		<0.01 – 0.10 (0.06 avg)	0.01	0.03 – 0.04 (0.04 avg)	-	-
Column 3		<0.01 – 0.13 (0.07 avg)	0.01	0.02 – 0.05 (0.04 avg)	-	-
		Total Chlorine (mg/L)				
Filter Run		1	2	3	4	5
Column 1	Field	0.054 – 0.95 (0.79 avg)	0.9 – 0.91 (0.91 avg)	0.77 – 0.95 (0.86 avg)	0.84 – 0.85 (0.85 avg)	0.59 – 0.77 (0.68 avg)
Column 2		0.41 – 1.00 (0.82 avg)	0.80 – 0.89 (0.85 avg)	0.80 – 0.95 (0.87 avg)	0.74	0.71 – 0.77 (0.74 avg)
Column 3		0.51 – 1.15 (0.92 avg)	0.86	0.76 – 1.07 (0.97 avg)	0.80	0.70
		Monochloramine (mg/L)				
Filter Run		1	2	3	4	5
Column 1	Field	0.60 – 1.02 (0.82 avg)	0.96	0.87 – 0.95 (0.91 avg)	-	-
Column 2		0.52 – 0.97 (0.78 avg)	1.00	0.86 – 0.89 (0.88 avg)	-	-
Column 3		0.57 – 1.12 (0.90 avg)	1.10	0.99 – 1.02 (1.01 avg)	-	-

The results presented in Table 6.12 indicated that the inclusion of aeration reduced the chlorine demand from iron and other constituents, such as hydrogen sulfide, by approximately 0.1 mg/L. At the start of the pilot study, a Hach method for hydrogen sulfide detection estimated raw water concentrations of less than 0.1 mg/L. Raw water sampling confirmed rotten egg odor, indicating hydrogen sulfide presence.

Stoichiometrically, the chlorine to ammonia ratio is 5:1 to achieve peak chloramination. Raw water ammonia averages of 0.24 mg/L in the field and 0.26 mg/L in the lab equate to 1.25 mg/L chlorine required to reach peak chloramination based on stoichiometrics. Additional chlorine demands include iron, manganese, and hydrogen sulfide. Iron requires a 0.63:1 chlorine to iron ratio and manganese requires a 1.3:1 chlorine to manganese ratio. The oxidation of manganese by chlorine does not occur rapidly, but is more rapid between iron and chlorine. Permanganate is a stronger oxidant than chlorine, and with it directly downstream of the pilot chlorine injection location, the Pilot Team assumed chlorine demand is limited to ammonia, iron, and the trace amounts of hydrogen sulfide present. In general, pilot results indicate chlorine demand consumption from iron and hydrogen sulfide at 0.4 mg/L without aeration, and 0.3 mg/L with aeration.

Chloramination analysis mostly took place during Runs 1 and 3 when total chlorine residual monitoring was more frequent. The Pilot Team attributes the dips in monochloramine and total chlorine during Run 1 to inaccurate chlorine feed that at the time was only at approximately 0.8 mg/L, compared to the 1.2 mg/L dose verified throughout the rest of the pilot study. The Team consistently checked chemical feed rates with every new bulk solution batch and chemical feed rate adjustment by a calibration column drawdown analysis. The results depict the systems response to changes in chlorine feed.

Figure 6.9 shows the chloramination results for Run 1 and 3, where the majority of peak chloramination analysis occurred. Monochloramine and total chlorine trended together and when low, free ammonia concentrations increased as expected.

The major takeaway from the ammonia and peak chloramination portions of the pilot study are that the raw water ammonia is not high enough to provide a conservative chloramine residual of at least 1.5 mg/L leaving the treatment facility. In most cases, the Project Team recommends a facility effluent chloramine residual of 2.0 mg/L. To reach these residuals, the proposed facility will require supplemental ammonia addition if Edina maintains a peak chloramination disinfection strategy. In general, another 1.0 mg/L total chlorine provides this residual, indicating a 0.2 mg/L ammonia dose (1.0 dividing by the 5:1 chlorine to ammonia ratio).

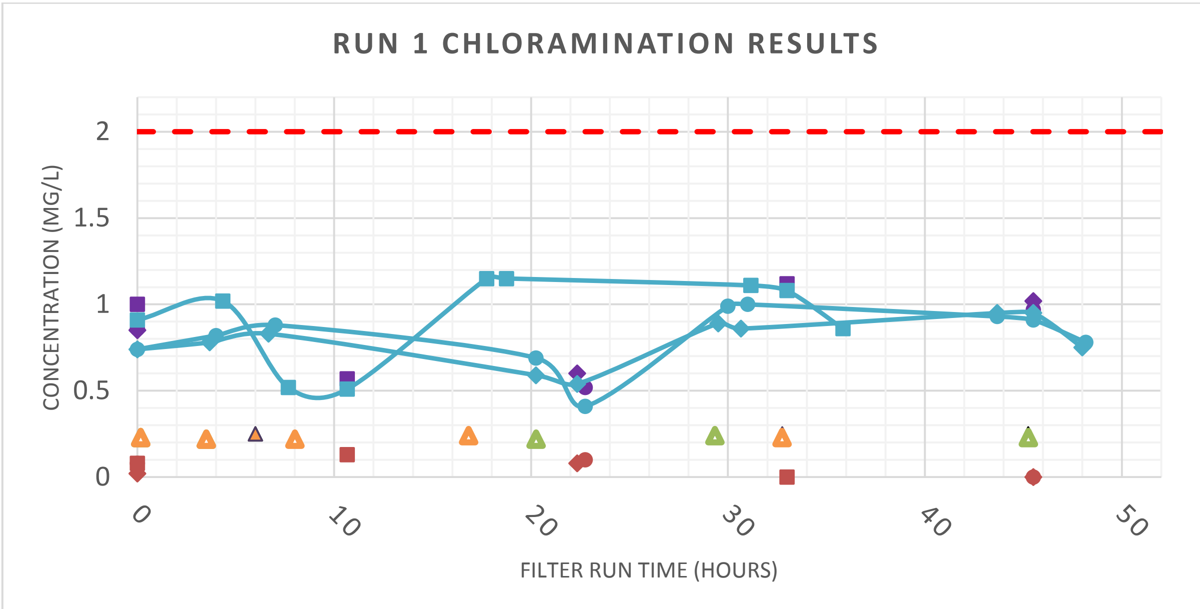

Figure 6.9 (a)

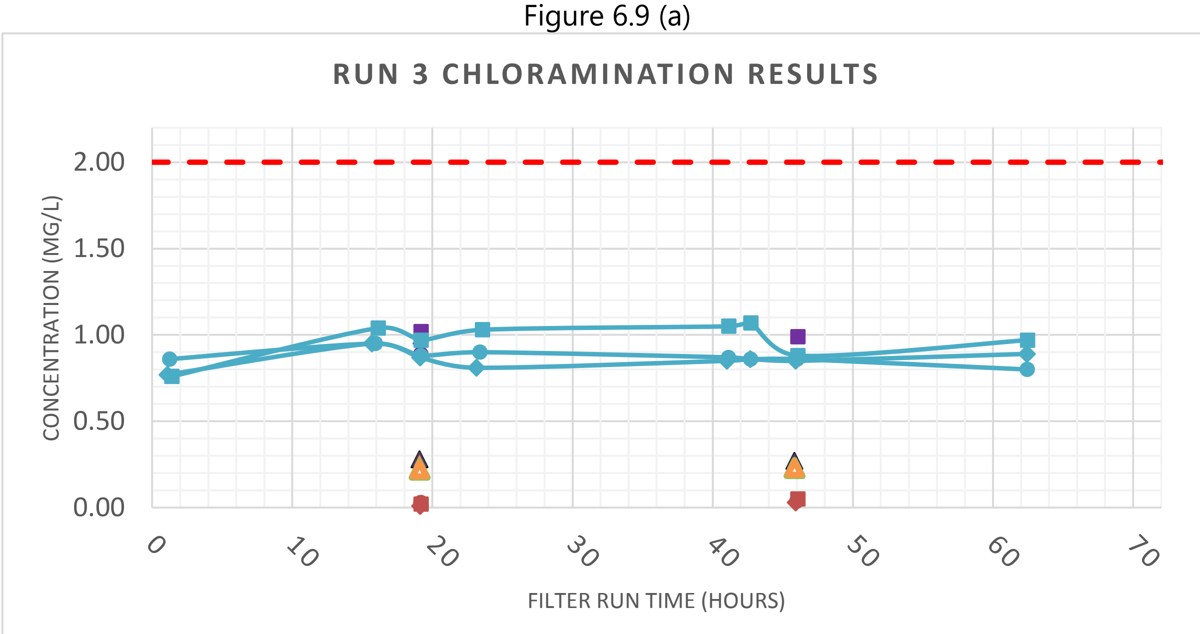

Figure 6.9 (b)

- Column 1 and 2 Raw Water Ammonia (Lab)
- Column 3 Raw Water Ammonia (Lab)
- Column 1 and 2 Raw Water Ammonia (Field)
- Column 3 Raw Water Ammonia (Field)
- Column 1 Free Ammonia
- Column 1 Monochloramine
- Column 2 Free Ammonia
- Column 2 Monochloramine
- Column 3 Free Ammonia
- Column 3 Monochloramine
- Column 1 Total Chlorine
- Column 2 Total Chlorine
- Column 3 Total Chlorine
- Recommended Total Chlorine Residual

Figure 6.9 Pilot Study Filter Run 1 and Run 3 Peak Chloramination Results

6.3.5.7 *Filter Head Loss and Run Time*

The five filter runs completed achieved run times of 51 hours, 71 hours, 67 hours, 18 hours, and 24 hours for Columns 1 and 2. For Column 3, filters achieved run times of 39 hours, 71 hours, 67 hours, 32 hours, and 24 hours. Tonka Water, Inc. suggested a terminal head loss buildup of 200 inches of water prior to backwash. No column during any of the five runs reached this head loss amount. This terminal head loss value is typical for pressure filters.

For gravity filtration, head loss build up is limited to the available water depth above the top of the media bed. The water depth must be great enough to overcome the head loss development over the filter run and the clean bed head loss that accounts for losses through the media layers and underdrain system. As a conservative estimate, a suggested 48 inches, or 4 feet of head loss development triggers a backwash in a gravity filtration system. This value will vary based on the filter operating level and final media bed gradation.

Figure 6.10 depicts the head loss development for Column 1, which contained IMAR™ media. Head loss development for Run 1 and Run 2 occurred more rapidly than other filter runs. This trend is expected, and it is typical for pilot systems to experience a declining head loss trend with successive filter runs due to the media fines present in new, unconditioned, filter media.

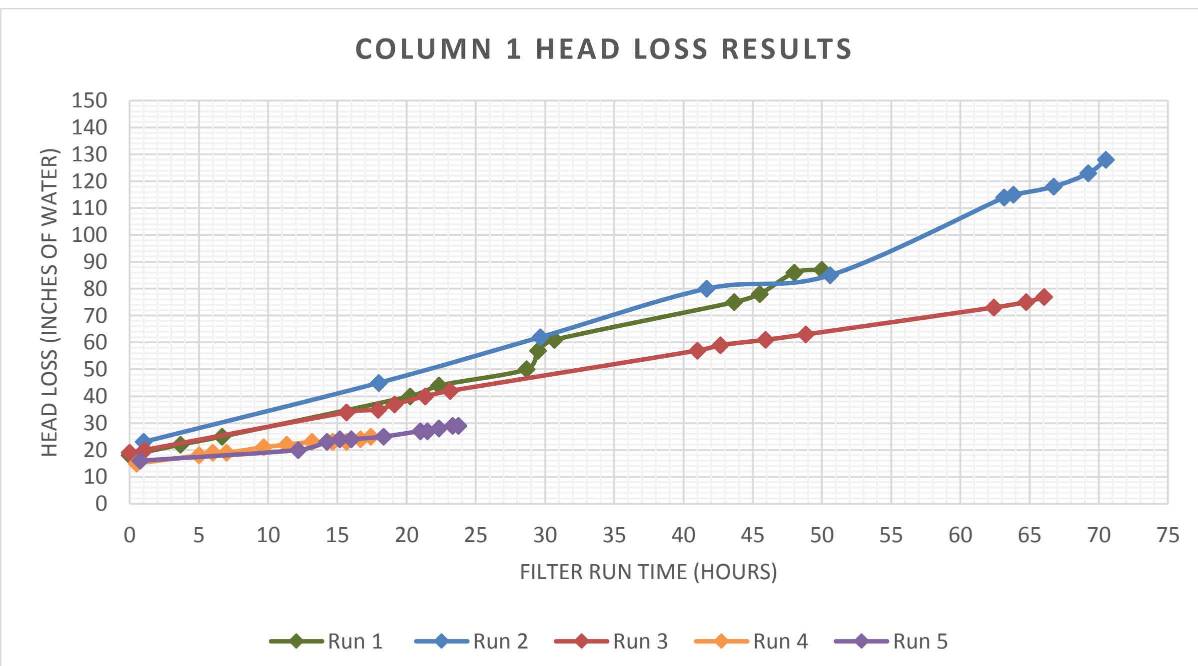

Figure 6.10 Pilot Study Column 1 Head Loss Development

The trajectory of Run 3 is a conservative head loss development curve to use in filter run time evaluations for Column 1. At the suggested 48 inches of head loss development for gravity filtration, IMAR™ media limits filter run time to 55 hours. This calculation assumes the Run 3 trajectory and head loss equals 68 inches of head. This considers an initial head loss of 20 inches at the start of the run.

Run 4 and Run 5 included the addition of detention time to provide a 30 minute period for chemical reactions to take place. Initial review of the trajectories indicate that detention slows the buildup of head loss. During Run 4, the pilot system was offline for two days over the weekend after starting Thursday afternoon (7/27/17) for Column 3 and Friday morning (7/28/17) for Column 1 and 2. During operation of the second half of Run 4 the Monday after the weekend, the Pilot Team noticed a decline in filter performance throughout the entire day, even at relatively short filter run times. The Team attributed the poor performance to the >60 hours of downtime that the system sat dormant. At the end of the day, the Team backwashed the filters and, upon completion, began Run 5. If this had not occurred, Run 4 would have extended through the end of the pilot study. Without the extended run time analysis with 30 minutes of detention, AE2S cannot draw definitive conclusions on whether detention provides reduction in head loss development.

It is also important to note that the minimal velocity through the detention tanks also may have caused settling of permanganate and HMO floc in the basin, rather than the filter. This occurrence artificially reduces the head loss development within the filter, which, unless the design team specifically intends for the detention tank to settle out floc prior to filtration, will not occur in full-scale operation.

Figure 6.11 and Figure 6.12 show the head loss development for Column 2 and Column 3, which both contained standard silica sand and anthracite media. Similar head loss development trends occurred for all five runs, but at a lesser magnitude than Column 1. At the suggested 48 inches of head loss development for gravity filtration, sand and anthracite filter provides run times of over 70 hours based on the Run 3 trajectories and initial head losses of 5 inches for Column 2 and 3. The impacts of detention on head loss development for Column 2 and 3 are again inconclusive due to the same reasons previously discussed for Column 1.

The graphs for Run 2 and 3 that are part of Figure 6.8, indicate that breakthrough of manganese may have occurred after run times of approximately 60 to 65 hours. It is important to note that permanganate dose adjustments may have also contributed to the manganese residual increases, so breakthrough from extended filter run time is not definitive. Regardless, 60 hours of run time for a gravity filter is common. In general, the Pilot Team terminated filter runs to demonstrate the success of backwash, re-establish manganese following a backwash, and meet other pilot objectives by adjusting operating parameters.

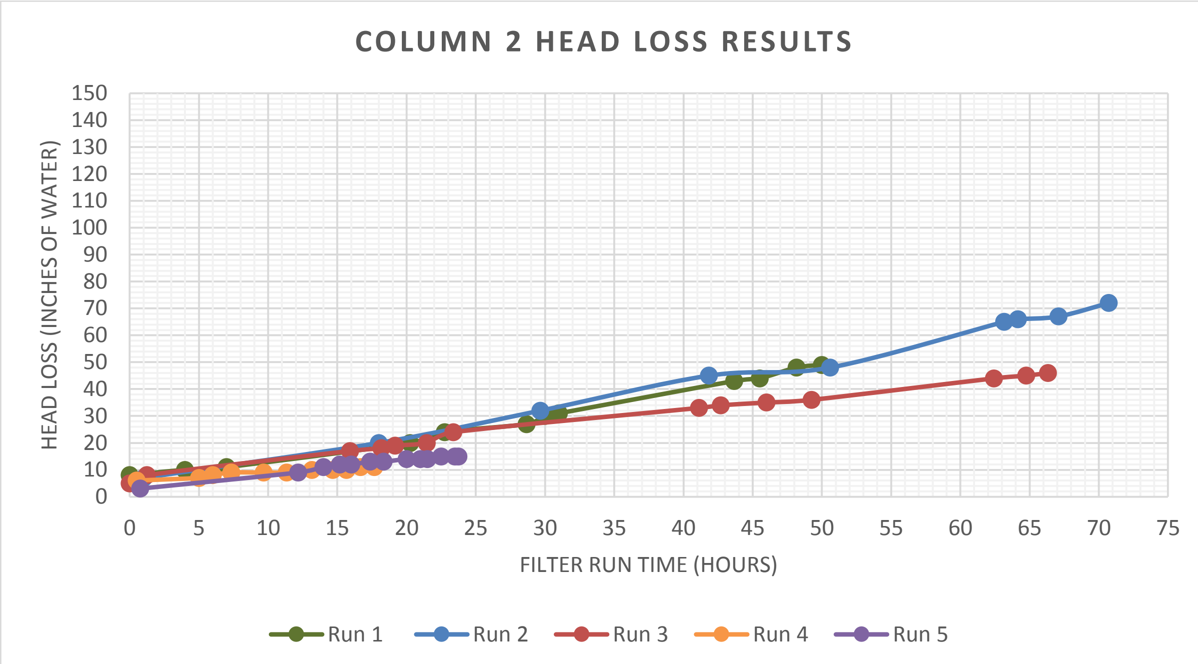

Figure 6.11 Pilot Study Column 2 Head Loss Development

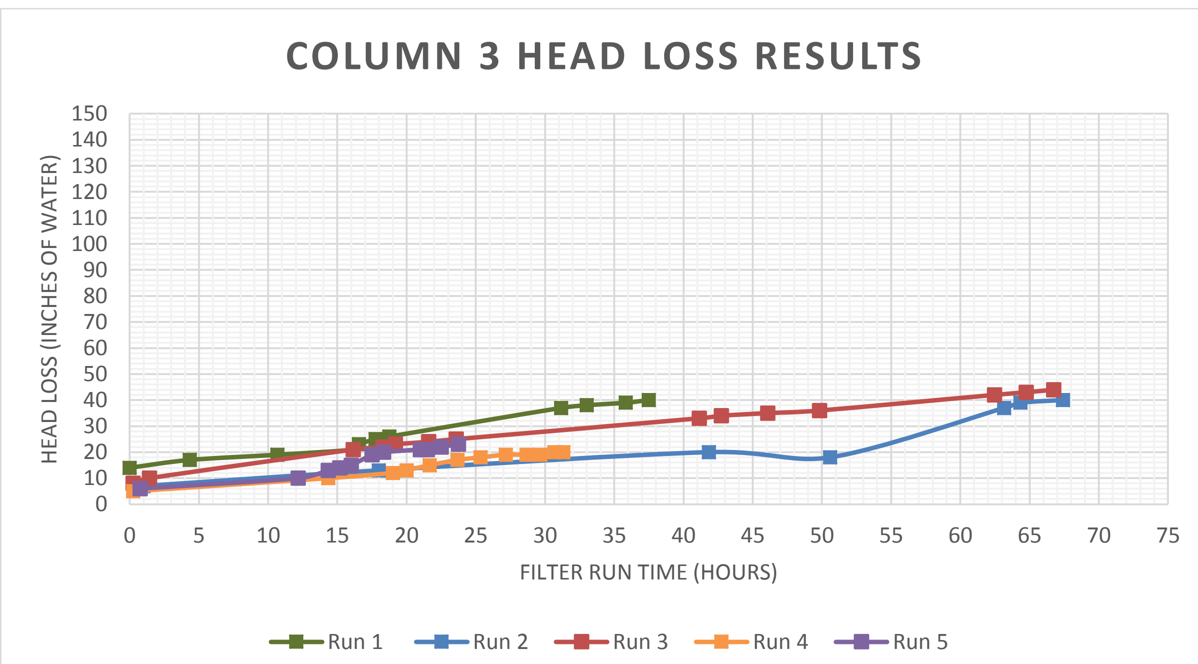

Figure 6.12 Pilot Study Column 3 Head Loss Development

6.3.5.8 *Detention Performance*

With detention online during Run 4 and 5, iron and manganese results were consistently lower than the 0.3 mg/L SMCL for iron and the 0.05 mg/L SMCL for manganese. Runs without detention also reached these performance levels. Pilot results indicate that the system is more sensitive to changes in chemical feed with the 7 to 8 minute detention time, providing evidence that extending detention time buffers changes in chemical feed.

Radium removal results presented in Section 6.3.5.5 indicate that extended detention time increased gross alpha and combined radium removal results for both filter media types analyzed. The one exception is gross alpha removal from Column 2 during the third sample set. It is important to not that raw water radionuclide concentrations were lower during this sample set. When comparing the results in terms of percentage below the regulated MCL, gross alpha removal was consistent between sample set 2 and 3.

Literature related to radium removal indicates increased difficulty in radium removal with presence of lower radionuclide concentrations in the raw water. This shows that removal of radionuclides may be difficult at very low concentrations. In summary, with extended detention, all radionuclides analyzed from the filter effluent resulted in concentrations equal to at least 75% below the regulated MCL. This provides evidence of extended detention time benefitting radium removal.

6.3.5.9 *Filter Backwash*

Each of the filters underwent six (6) backwashes during the course of the pilot study. The first two (2) occurred prior to the start of Run 1. On the second day of pilot commissioning and setup (7/18/17), the Tonka Water, Inc. field representative trained AE2S staff on backwashing procedures. This backwash also aimed to remove some of the initial media fines present with any unconditioned media. The second backwash occurred first thing on 7/19/17 to ensure fresh media beds for Run 1 start-up. The other four (4) backwashes occurred after Run 1, Run 2, Run 3, and Run 4.

The backwash consisted of a 10 minute Simul-Wash™ air-water backwash with a water rate of approximately 3 gpm/ft^2 (1.05 gpm) and air rate of 3 cfm/ft^2 (1.05 cfm). For the first three backwashes, all columns then underwent a 2 minute air purge with water only at the 3 gpm/ft^2 followed by a 3 minute restratification step with water only at 10 gpm/ft^2 (3.5 gpm). After consultation with Tonka Water, Inc., AE2S confirmed that standard silica sand and anthracite media requires 13 to 15 gpm/ft^2 (4.5 to 5.2 gpm) flow for adequate restratification. The Pilot Team replaced the backwash flow control rotameter prior to backwashing at the end of Run 1 to provide this elevated flow to Columns 2 and 3 for future backwashes. For IMAR™, all backwashes used the 10 gpm/ft^2 rate for the restratification step.

To develop a preliminary understanding of the backwash settling time of the source water, AE2S set up a backwash settling column consisting of an approximately five (5) foot tall, four (4) inch diameter clear PVC column with a sealed end cap and removable top cap. After Run 1 and Run 3, the Pilot Team collected adequately mixed backwash effluent samples and filled the settling column. In both analyses, the majority of settling occurred by the following day, indicating an estimated four (4) feet of settling occurs within the first twenty-four (24) hours. Figure 6.13 shows the progression of the backwash settling results for Column 1 after Run 3. The second image shows large floc images settling within the first few hours.

7/27/17 at 13:30 **7/27/17 at 15:00** **7/28/17 at 8:00**

Figure 6.13 Backwash Settling Results after Run 3 for Column 1.

6.3.6 Pilot Study Conclusions

Conclusions drawn from the twelve (12) day pilot study include the following:

1. All three process trains consisting of aeration (Column 3 only), chlorine and permanganate followed by a minimum 7 to 8 minutes of detention prior to filtration through IMAR™ or standard silica sand and anthracite media effectively removed iron and manganese to below the respective 0.3 mg/L SMCL for iron and 0.05 mg/L SMCL for manganese from Well No. 18 in Edina, MN.
2. The use of aeration upstream of chemical pre-oxidant addition removed, on average, 73% of the raw water iron and reduced chlorine demand by approximately 0.1 mg/L. Aeration did not successfully remove manganese.
3. All aerator and filter effluent samples resulted in iron residuals below the 0.3 mg/L SMCL.
4. Hydrogen sulfide presence has a minor impact on the required chlorine dose to reach peak chloramination.
5. No significant difference in filter performance resulted between the two filter media types analyzed in the pilot study in terms of iron and manganese removal.
6. All three columns consistently removed manganese to levels below the 0.05 mg/L SMCL when operating at optimum permanganate dose. Optimum manganese dose ranged from 0.4 to 0.7 mg/L depending on other chemical feed, detention time, and filter run times.
7. Field and lab sampling indicated manganese breakthrough at a permanganate dose of over 0.8 mg/L.
8. Lab manganese results were substantially lower than field results, indicating better manganese removal than initially assumed from field results.
9. Permanganate alone removed gross alpha and combined radium to below the regulated MCL's, but not consistently to the desired treatment goal.
10. IMAR™ media outperformed conventional sand and anthracite for the majority of the radionuclide results, but the additional head loss build up and lack of enhanced performance by this filter does not outweigh the improved radionuclide removal.
11. HMO improved radionuclide removal to the treatment goal of half the regulated MCL. The Pilot Team recommends inclusion of HMO equipment in WTP No. 5 for consistent removal of radionuclides to half the MCL. The Pilot Team recommends optimization of HMO dose to minimize chemical acquisition costs.

12. The system responded as expected while operating at peak chloramination. Total chlorine and monochloramine trended together and when low, free ammonia concentrations increased.
13. The chloramination analysis confirmed that the raw water ammonia is not high enough to provide the recommended 2.0 mg/L total chlorine residual leaving the treatment facility.
14. Run 4 and 5 provided 30 minutes of detention and during these two runs, all field and lab samples resulted in iron residuals below method detection limits. Field manganese results were more consistent during these runs, indicating that the extended detention time may provide a buffer when making chemical feed changes.
15. Head loss development occurred more rapidly for IMAR™ media than standard silica sand and anthracite during all five (5) filter runs.
16. Approximate filter run time for IMAR™ media is 50 hours and for standard sand and anthracite is over 70 hours when looking at head loss alone. Pilot results did not definitively indicate manganese breakthrough, but manganese residual concentrations increased after 60 hours of run time during Run 2 and Run 3.

6.4 Recommended Preliminary Treatment Train

The Pilot Team developed the preliminary treatment train for WTP No. 5 based on preliminary bench scale testing, the pilot study investigation and discussions with City of Edina staff. The following recommendations detail the system component for each treatment goal:

1. Iron and Manganese Removal: use chlorine and permanganate as pre-oxidants to oxidize iron, manganese, and hydrogen sulfide prior to filtration
2. Radium Removal: use permanganate and HMO followed by the extended detention time for consistent radionuclide removal to half the regulated MCLs.
3. Detention: provide 30 minutes of detention to allow additional time for pre-oxidation reactions to take place, provide a buffer for permanganate, and offer treatment flexibility for the unknown water quality of future Well No. 21.
4. Filtration:
    a. Size filters to operate at a 3 gpm/ft^2 loading rate.
    b. Load filters with 18″ of silica sand (0.45 – 0.55 mm) and a 12″ cap of anthracite (0.8 – 1.0 mm).
    c. Install a sustainable simultaneous air and water backwash system to ensure thorough cleaning of the filter media and reduce backwash waste water. A

preliminary backwash sequence includes 10 minutes of simultaneous air and water wash at 3 gpm/ft^2 and 3 cfm/ft^2, a 2 minute air purge at 3 gpm/ft^2 and a 3 minute media restratification at 13 to 15 gpm/ft^2.

   d. Size the backwash reclaim system to provide enough storage for backwashing all filters once and allowing two days of settling before reclaim.

   e. Expect filter run lengths of at least 60 hours. Extended run times may occur with detention and permanganate optimization.

5. Disinfection: provide the chemical feed systems necessary to operate at either peak chloramination or breakpoint chlorination.

   a. Peak chloramination: requires chlorine and supplemental ammonia at doses that provide a recommended 2.0 mg/L total chlorine leaving the facility.

   b. Breakpoint chlorination: requires chlorine at a dose that provides a recommended 1.0 mg/L free chlorine residual leaving the facility.

   c. Size the chlorine system to feed at least 4.0 mg/L of available chlorine and the ammonia system to feed at least 1.0 mg/L of available ammonia. Actual feed rates will vary based on well operation and chosen disinfection method.

6. Additional chemical feed post-filtration includes fluoride and an ortho/poly blend for corrosion control.

Figure 6.14 shows a process flow diagram for the recommended WTP No. 5 treatment train.

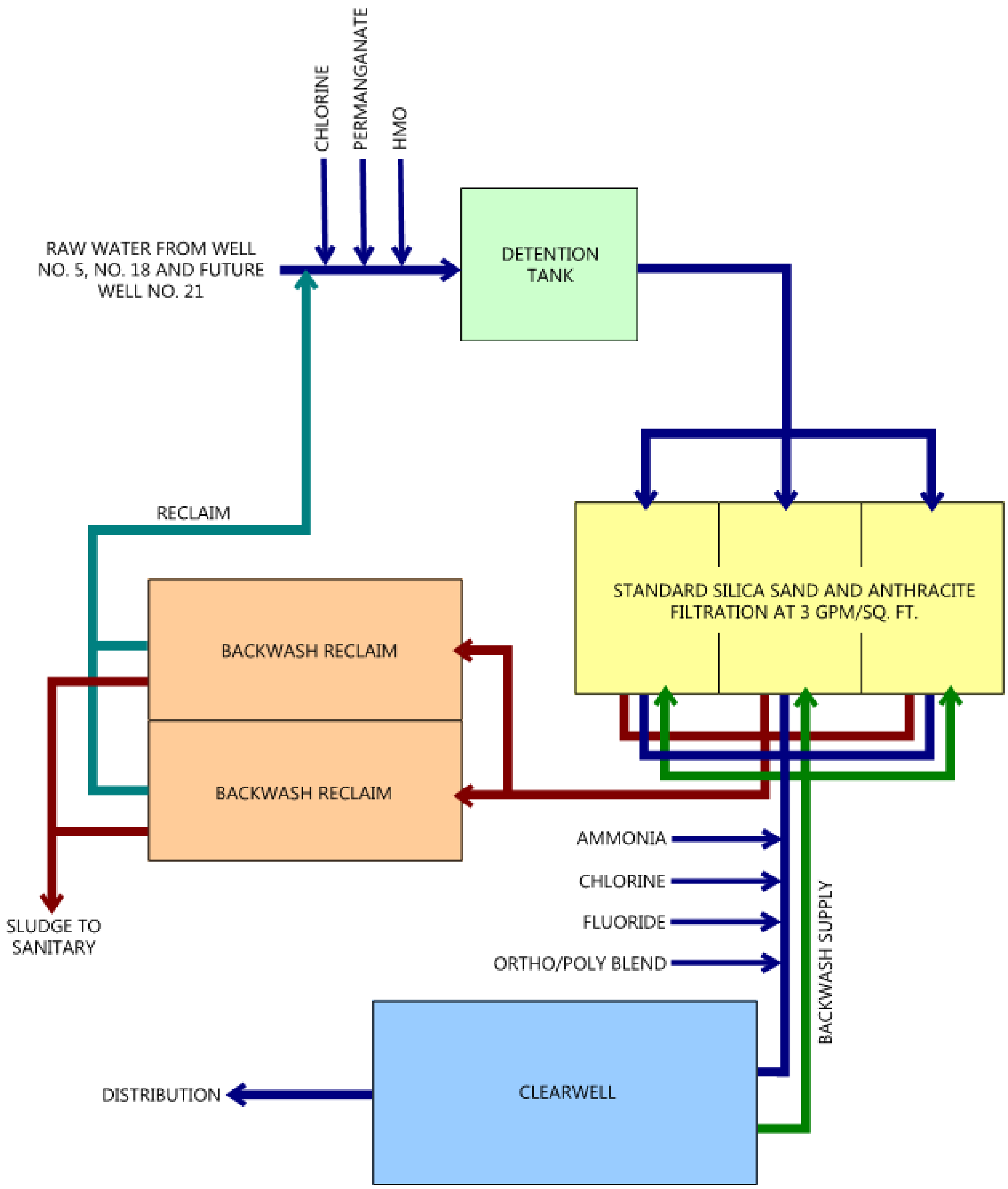

Figure 6.14 Preliminary Recommended Treatment Train Diagram

CHAPTER 7 TREATMENT TECHNOLOGY EVALUATIONS

7.1 Treatment Technology Evaluations

The following sections evaluate the treatment technology alternatives described in detail within Chapter 5. This chapter focuses on life cycle costs of each alternative to make decisions on technologies included in the base facility assumed for preliminary site layouts and capital costs. Individual technology capital costs, operation, and maintenance (O&M) costs are the primary focus of this chapter.

The values provided throughout this chapter are conservative values, assuming that WTP No. 5 operates 24-hours per day for all 365 days of the year. This will not be the case, acknowledging that operations staff will cycle use of the facility with seasonal variation, system demands, and other maintenance considerations. Staff may choose operation of another facility over WTP No. 5 to limit annual chemical feed costs, considering the higher acquisition costs of sodium permanganate and HMO, which will reduce the amount of water in the distribution system treated for manganese removal. This is an example of an internal tradeoff that the water utility will determine day by day.

The Project Team based the life cycle evaluations on a 30-year planning period, which is an industry standard, and an assumed 3% annual inflation rate. The fixed O&M expenses account for the future expense value based on these parameters. Total annual O&M expenses also account for inflation over the planning period.

Capital costs listed within this chapter only include the equipment and installation costs for each alternative, unless otherwise specified directly. The overall base facility capital costs provided in Chapter 11 factor in additional costs related to the building, mechanical, or electrical components.

7.2 Chlorine Alternatives

Section 5.3.2 identified gas chlorination, bulk sodium hypochlorite, and onsite generation of sodium hypochlorite as alternative chlorine technologies for further evaluation. The following sections evaluate the life cycle costs for a 30-year planning period for each alternative. The evaluation determines the chlorine alternative selection used in the preliminary base facility designs.

7.2.1 Gas Chlorination

7.2.1.1 *Capital Costs*

Estimation of the capital cost associated with chlorine gas feed and storage equipment is $252,000 (2017 dollars). This cost includes the gas chlorinator and injection system including all scales, injectors, and feeders, chlorine gas detection and emergency system with chlorine scrubber, and chlorine gas ton cylinder unloading system with beam, hoist, and winch. The estimates do not include contingencies or engineering, administrative and legal costs, but Chapter 11 estimates do include these costs.

7.2.1.2 *O&M Costs*

O&M costs associated with a gas chlorination system primarily consist of the cost for chemical and preparation and maintenance of a chlorine Risk Management Plans (RMP). For the purposes of this analysis, O&M costs do not include routine maintenance of the system, assuming relatively similar annual maintenance time required for all considered alternatives.

Assuming peak chloramination and applying a 2.3 mg/L chlorine dose at the 3,000 gpm plant production capacity, WTP No. 5 requires approximately 15.1 tons of chlorine gas annually. Using the current chlorine gas price of $0.45 per pound, the estimated annual O&M cost associated with maintaining an adequate supply of chlorine gas onsite is $13,606. With annual inflation of 3% per year, the future expense value of the annual O&M expense over a 30-year planning period is $647,000.

Assuming a FAC dosage of 2.3 mg/l, the incremental cost of gaseous chlorine is approximately $0.009 per 1,000-gallons.

'Fixed' O&M costs associated with gas chlorine include development and periodic update of an Environmental Protection Agency (EPA) Risk Management Plan, and Occupational Safety and Health Administration (OSHA) Process Safety Management Plan, both required in conjunction with storage and handling of chlorine gas at the Edina WTP. Additional 'fixed' O&M costs include approximately $36,000 every 10 years for gas chlorine system replacement and an estimated $15,000 for initial development of an EPA/OSHA Risk Management and Process Safety Management Plan, with plan updates and associated submittals every five (5) years in the range of $5,000 per update. The total 'fixed' O&M cost inflates future expenses at a 3% annual rate up to the anticipated year of expense.

Based on these assumptions, the total 30-year life cycle cost for a gas chlorination system is $1,159,000. Table 7.1 summarizes the life cycle costs for the gas chlorine option.

Table 7.1 Life Cycle Costs – Gas Chlorine

Chlorine System	A	B	C	D	E = A + B + D
	Capital Costs	**Fixed O&M Expense**	**Present Annual O&M Expense**	**Annual O&M Expense Over 30 years**	**Capital + O&M Expense**
Gaseous Chlorine	$252,000	$260,000	$13,606	$647,000	$1,159,000

7.2.2 Bulk Sodium Hypochlorite

7.2.2.1 *Capital Costs*

Estimation of the capital cost associated with bulk sodium hypochlorite feed and storage equipment is $40,000 (2017 dollars). This cost represents 4,000 gallons of bulk storage, a 350-gallon day tank, transfer pump(s), peristaltic or diaphragm chemical applications pumps, ultrasonic tank level monitoring equipment and readouts, associated piping, valves, and appurtenances, and an injection quill. The estimates do not include miscellaneous safety requirements, contingencies or engineering, administrative and legal costs, but Chapter 11 estimates do include these costs.

7.2.2.2 *O&M Costs*

O&M costs associated with a bulk NaOCl system primarily consist of the cost for chemical. For the purposes of this analysis, O&M costs do not include routine maintenance of the system, assuming relatively similar annual maintenance time required for all considered alternatives.

Industry standard considers approximately 0.88 gallons of 12.5% NaOCl equals the oxidizing power of 1 pound (lb) of FAC. Assuming peak chloramination and applying a 2.3 mg/L chlorine dose at the 3,000 gpm plant production capacity, WTP No. 5 requires approximately 26,477 gallons of 12.5% NaOCl annually. Using the current approximate price for bulk 12.5% NaOCl of $1.00 per gallon, the estimated annual O&M cost associated with maintaining an adequate supply of bulk solution onsite is $26,477. With annual inflation of 3% per year, the future expense value of the annual O&M expense over a 30-year planning period is $1,260,000. Assuming a FAC dosage of 2.3 mg/l, the incremental cost of bulk 12.5% NaOCl is approximately $0.017 per 1,000-gallons.

Fixed O&M expense associated with a bulk NaOCl system involves the replacement of liquid chemical metering pumps every 10 years at $4,500 each and polyethylene storage tank replacement every 7 years at $6,000 per tank. The total 'fixed' O&M cost inflates future expenses at a 3% annual rate up to the anticipated year of expense.

Based on these assumptions, the total 30-year life cycle cost for a bulk sodium hypochlorite system is $1,366,000. Table 7.2 summarizes the life cycle costs for this system.

Table 7.2 Life Cycle Costs – Bulk Sodium Hypochlorite

Chlorine System	A	B	C	D	E = A + B + D
	Capital Costs	**Fixed O&M Expense**	**Present Annual O&M Expense**	**Annual O&M Expense Over 30 years**	**Capital + O&M Expense**
Bulk Sodium Hypochlorite	$40,000	$66,000	$26,477	$1,260,000	$1,366,000

7.2.3 Onsite Hypochlorite Generation

7.2.3.1 *Capital Costs*

Estimation of the capital cost associated with an onsite hypochlorite generation (OSHG) system is $476,000 (2017 dollars). This cost corresponds to a total FAC production capacity of 400 PPD, with two (2) redundant 200 PPD NaOCl generation trains. This provides the required 200 PPD capacity with one (1) train out of service. The cost is representative of equipment only, and includes a salt brine storage tank, two (2) 200 PPD NaOCl generation trains, controls, process water softener, process water heater, hydrogen detector and dilution/vent system, bulk NaOCl storage (day) tanks, diaphragm chemical feed pumps, and miscellaneous system valves and monitoring equipment.

Due to the nature of onsite NaOCl generation, industry standard recommends providing redundant generation equipment in order to provide at least the existing maximum day FAC production capacity required with one (1) generation unit/train out of commission. Maximum FAC production for WTP No. 5 is approximately 85 PPD if operating at peak chloramination and 115 PPD at breakpoint chlorination disinfection strategies.

7.2.3.2 *O&M Costs*

O&M costs associated with an onsite generation system primarily consist of the cost for salt and power to operate the system. It is important to note that the other systems evaluated, which include gaseous chlorine and bulk NaOCl delivery, did not have a power cost incorporated into the O&M cost estimation because the cost to operate the systems is considered negligible. However, an onsite generation system involves a more significant power requirement, and as such, estimations include power costs. Again, for the purposes of this analysis, O&M costs do not include routine maintenance of the system, assuming relatively similar annual maintenance time required for all considered alternatives.

Generation of 1 lb of FAC requires 3 lbs salt, 2 kilowatt-hours (kWh) of power, and 15 gallons of softened water. Given these salt and power requirements and applying the assumed 2.3 mg/L chlorine dosage for peak chloramination and 3,000 gpm water production, annual salt requirements are 90,700 lbs and annual power requirements are 60,500 kWh. Using current quoted prices for salt and power of $0.06 per lb salt and $0.04 per kWh, respectively, the annual estimated O&M cost associated with onsite generation of 0.8% NaOCl is $7,860 per year. Assuming a FAC dosage of 2.3 mg/l, the incremental cost of onsite generation is approximately $0.007 per 1,000-gallons.

Onsite generation catalytic cells have a replacement life cycle of approximately every 7-10 years. As such, 'fixed' O&M considerations for an onsite generation system include the replacement of two (2) 200 PPD generation cells on a 7-year cycle at approximately $30,000 per cell. Additional 'fixed' O&M expenses include the replacement of highly utilized system valves at $500 annually, brine pump replacement on a 5-year cycle at $2,000 per pump, and switchgear maintenance on a 10-year cycle at $3,000 per event.

Based on these assumptions, the total 30-year life cycle cost for an OSHG system is $1,223,000. Table 7.3 summarizes the systems life cycle costs.

Table 7.3 Life Cycle Costs – Onsite Hypochlorite Generation System

Chlorine System	A	B	C	D	E = A + B + D
	Capital Costs	**Fixed O&M Expense**	**Present Annual O&M Expense**	**Annual O&M Expense Over 30 years**	**Capital + O&M Expense**
OSHG System	$476,000	$373,000	$7,861	$374,000	$1,223,000

7.2.4 Chlorine Alternative Selection

The following sections compare the three alternatives previously described. Based on the life cycle cost comparison and other factors associated with each alternative, the Project Team assumed a gas chlorine system for inclusion in the preliminary base facilities.

7.2.4.1 *Chlorine Alternative Cost Comparison*

The 30-year life cycle evaluation compared the capital and estimated O&M cost expenditures for each chlorine alternative. As noted previously, the annual O&M expenses over the 30-year period account for a 3% annual inflation. This does not account for potentially variable chemical costs in the future.

Table 7.4 summarizes the life cycle cost evaluation. Recall that all values assume a FAC dosage of 2.3 mg/L for peak chloramination and a plant capacity of 3,000 gpm for WTP No. 5.

Table 7.4 Summary of Chlorine Alternative Life Cycle Costs

Chlorine System	A	B	C	D	E = A + B + D
	Capital Costs	**Fixed O&M Expense**	**Present Annual O&M Expense**	**Annual O&M Expense Over 30 years**	**Capital + O&M Expense**
Gaseous Chlorine	$252,000	$260,000	$13,606	$647,000	$1,159,000
Bulk Sodium Hypochlorite	$40,000	$66,000	$26,477	$1,260,000	$1,366,000
OSHG System	$476,000	$373,000	$7,861	$374,000	$1,223,000

From a capital cost perspective, the gaseous chlorine and onsite generation systems are significantly more expensive than a bulk NaOCl system at $252,000 and $476,000, respectively. Capital costs associated with a bulk NaOCl system are in the range of $40,000. Preceding sections discussed fixed O&M expenses associated with each option. Overall, it appears that the fixed O&M expenses anticipated for bulk NaOCl system ($66,000) is low compared to the fixed expenses ($260,000) for gaseous chlorine and ($373,000) for an OSHG system over the 30-year life cycle.

Production-based O&M costs, or those primarily related to chemical acquisition or generation under each disinfection system vary significantly. Onsite generation incurs the least chemical O&M expenses in conjunction with salt and power requirements ($374,000 over 30 years). Gaseous chlorine is twice as expensive for delivery of 1-ton gaseous chlorine cylinders ($647,000 over 30 years). A bulk NaOCl system by far incurs the most chemical O&M expense for delivery of 12.5% liquid NaOCl ($1,260,000 over 30 years).

The life cycle cost evaluation indicates that a gaseous chlorine system is the least expensive system over the 30-year life cycle period, with an estimated combined capital and O&M

expense of $1,159,000. An OSHG system is slightly more expensive with an estimated combined capital and O&M expense of $1,223,000. A bulk NaOCl delivery system is most expensive over the life cycle, primarily due to the annual chemical expense, with an estimated combined capital and O&M expense of approximately $1,366,000.

It is important to note that the Project Team based the life cycle costs on the best preliminary data available at this time and subsequent planning level cost analyses; as such, the estimates are relative figures used as a basis for comparison.

7.2.5 Additional Evaluation Factors

Delivery and storage of gaseous chlorine poses a significant safety and health concern, especially in light of the potential location of the proposed Edina WTP No. 5 in very close proximity to a shopping center, office buildings, and community recreational areas. The gaseous chlorine system includes a wet scrubber for emergency protection per the NFPA fire code. The City currently uses gas chlorine for all other facilities, so operators are very familiar with the product O&M procedures.

Consider onsite generation and bulk delivery of NaOCl "inherently safer" chlorine alternatives in comparison to gaseous chlorine. Recent developments at the regulatory level have indicated a strong preference for "inherently safer technologies", making both onsite generation of 0.8% NaOCl and bulk delivery of 12.5% NaOCl favored as primary and residual disinfection processes.

An onsite generation system provides a continuously fresh supply of dilute NaOCl that would not require EPA or OSHA regulatory compliance. Operation of the system is highly automated given the provision of an integrated control system, and the Edina staff may become comfortable operating the onsite generation system components quickly. Onsite generation facilitates a high level of public health and safety. The only chemical delivery required as part of an onsite generation system is bulk water softener grade salt, which minimizes the risk of a spill or leak of a hazardous chemical in the delivery area of the proposed WTP. Components of an onsite generation system do not require structural separation from other WTP systems, and the process is safe for Edina staff operation; however, an onsite generation system does require many process components to operate and maintain in comparison to a simple bulk liquid NaOCl delivery and feed system.

Bulk delivery of NaOCl provides a safer chlorine option for the proposed WTP, but the increase of the long term chemical expense proves to be a significant long term disadvantage. A bulk NaOCl system is quite simple. While the system still requires Risk Management documentation, it is relatively simple and inexpensive to develop and maintain in comparison to the requirements for a gaseous chlorination system. The estimated annual O&M costs associated with a bulk delivery system are twice those for a gas system.

The Project Team recommends implementation of a gaseous chlorination system in the new WTP employing the use of a wet scrubber at this time based on operator familiarity, non-degradation of the chemical, and life cycle costs. Future discussions with local fire officials and City staff may indicate the acceptance of an emergency shutoff system in place of the currently proposed chlorine scrubber. This system has a lower initial capital investment and smaller long term maintenance cost, which cost savings associated with the gaseous chlorine system makes it even more favorable as the selected alternative.

7.3 Ammonia Alternatives

Section 5.3.3 identified anhydrous ammonia, aqua ammonia, and dry or liquid ammonium sulfate as alternative ammonia technologies. Due to the instability and corrosive nature of aqua ammonia, no further evaluation of the option took place. The following sections evaluate the life cycle costs for a 30-year planning period for anhydrous ammonia and the two ammonium sulfate options. The evaluation determines the ammonia alternative selection used in the preliminary base facility designs.

7.3.1 Anhydrous Ammonia

7.3.1.1 *Capital Costs*

Estimation of the capital cost associated with anhydrous ammonia feed and storage equipment is $181,000 (2017 dollars). This cost includes the gas chlorinator and injection system including all scales, injectors, and feeders, ammonia gas detection and emergency system with ammonia scrubber, and anhydrous ammonia 140 lb cylinder scales. Additionally, an anhydrous ammonia system requires softened carrier water, so the capital cost includes the cost of a 50 gpm ion exchange system. The estimates do not include contingencies or engineering, administrative and legal costs, but Chapter 11 estimates do include these costs.

7.3.1.2 *O&M Costs*

O&M costs associated with an anhydrous ammonia system primarily consist of the cost for chemical. For the purposes of this analysis, O&M costs do not include routine maintenance of the system, assuming relatively similar annual maintenance time required for all considered alternatives.

Assuming peak chloramination and applying a 0.3 mg/L ammonia dose at the 3,000 gpm plant production capacity, WTP No. 5 requires approximately 3,944 pounds of ammonia gas annually. Using the current industry anhydrous ammonia price of $2.00 per pound, the estimated annual O&M cost associated with maintaining an adequate supply of ammonia gas onsite is $7,887. With annual inflation of 3% per year, the future expense value of the annual O&M expense over a 30-year planning period is $375,000. Assuming an ammonia dosage of 0.3 mg/l, the incremental cost of gaseous ammonia is approximately $0.005 per 1,000-gallons.

Limited regulatory/risk management compliance would likely be required for an anhydrous ammonia system unless the system size increases significantly in the future and onsite storage amounts to over 10,000 pounds of stored product. 'Fixed' O&M costs associated with gas ammonia include approximately $25,000 every 10 years for anhydrous ammonia system replacement and ion exchange system resin replacement estimates at $2,000 every five years. The total 'fixed' O&M cost inflates future expenses at a 3% annual rate up to the anticipated year of expense.

Based on these assumptions, the total 30-year life cycle cost for an anhydrous ammonia system is $716,000. Table 7.5 summarizes the life cycle costs for this option.

Table 7.5 Life Cycle Costs – Anhydrous Ammonia

Ammonia System	A	B	C	D	E = A + B + D
	Capital Costs	**Fixed O&M Expense**	**Present Annual O&M Expense**	**Annual O&M Expense Over 30 years**	**Capital + O&M Expense**
Anhydrous Ammonia	$181,000	$160,000	$7,887	$375,000	$716,000

7.3.2 Liquid Ammonium Sulfate

7.3.2.1 *Capital Costs*

Estimation of the capital cost associated with bulk, liquid ammonium sulfate feed and storage equipment is $20,000 (2017 dollars). This cost represents 1,000 gallons of bulk storage, peristaltic or diaphragm chemical applications pumps, ultrasonic tank level monitoring equipment and readouts, associated piping, valves, and appurtenances, and an injection quill. The estimates do not include miscellaneous safety requirements, contingencies or engineering, administrative and legal costs, but Chapter 11 estimates do include these costs.

7.3.2.2 O&M Costs

O&M costs associated with a bulk ammonium sulfate system primarily consist of the cost for chemical. For the purposes of this analysis, O&M costs do not include routine maintenance of the system, assuming relatively similar annual maintenance time required for all considered alternatives.

Industry standard considers approximately 1.35 gallons of 35% $(NH_4)_2SO_4$ equals the oxidizing power of 1 pound (lb) of anhydrous ammonia. Assuming peak chloramination and applying a 0.3 mg/L ammonia dose at the 3,000 gpm plant production capacity, WTP No. 5 requires approximately 5,308 gallons of 35% $(NH_4)_2SO_4$ annually. Using the current approximate price for bulk 35% $(NH_4)_2SO_4$ of $4.00 per gallon, the estimated annual O&M cost associated with maintaining an adequate supply of bulk solution onsite is $21,231. With annual inflation of 3% per year, the future expense value of the annual O&M expense over a 30-year planning period is $1,010,000. Assuming an ammonia dosage of 0.3 mg/l, the incremental cost of 35% $(NH_4)_2SO_4$ is approximately $0.013 per 1,000-gallons.

Limited regulatory/risk management compliance would likely be required for a bulk $(NH_4)_2SO_4$ system unless the system size increases significantly in the future and onsite storage amounts to over 20,000 pounds of stored product. Estimated 'fixed' O&M expense associated with a bulk $(NH_4)_2SO_4$ system involves the replacement of liquid chemical metering pumps every 10 years at $4,500 each and polyethylene storage tank replacement every 7 years at $3,000 per tank. The total 'fixed' O&M cost inflates future expenses at a 3% annual rate up to the anticipated year of expense.

Based on these assumptions, the total 30-year life cycle cost for a bulk ammonium sulfate system is $1,076,000. Table 7.6 summarizes the life cycle costs for this system.

Table 7.6 Life Cycle Costs – Bulk Liquid Ammonium Sulfate

Ammonia System	A	B	C	D	E = A + B + D
	Capital Costs	**Fixed O&M Expense**	**Present Annual O&M Expense**	**Annual O&M Expense Over 30 years**	**Capital + O&M Expense**
Liquid Ammonium Sulfate	$20,000	$46,000	$21,231	$1,010,000	$1,076,000

7.3.3 Dry Ammonium Sulfate

7.3.3.1 *Capital Costs*

Estimation of the capital cost associated with a dry ammonium sulfate batch system is $70,000 (2017 dollars). This cost includes a dry chemical hopper, automatic volumetric screw feeder, solution tank/dissolver with mixer, a control panel for operation of the system, diaphragm chemical feed pumps, associated piping, valves, and appurtenances, and an injection quill.

7.3.3.2 *O&M Costs*

O&M costs associated with a dry ammonium sulfate system primarily consist of the cost for chemical. For the purposes of this analysis, O&M costs do not include routine operation maintenance of the system, although dry feed systems require significantly more operator involvement for operating and maintaining the batch feed system.

Approximately 1.34 pounds of dry ammonium sulfate equals the oxidizing power of 1 pound (lb) of anhydrous ammonia. Assuming peak chloramination and applying a 0.3 mg/L ammonia dose at the 3,000 gpm plant production capacity, WTP No. 5 requires approximately 16,144 pounds of dry $(NH_4)_2SO_4$ annually. Using the current approximate price for dry $(NH_4)_2SO_4$ of $0.70 per pound, the estimated annual O&M cost associated with maintaining an adequate supply of batched solution onsite is $11,301. With annual inflation of 3% per year, the future expense value of the annual O&M expense over a 30-year planning period is $538,000. Assuming an ammonia dosage of 0.3 mg/l, the incremental cost of dry $(NH_4)_2SO_4$ is approximately $0.002 per 1,000-gallons.

'Fixed' O&M considerations for a batch dry ammonium sulfate system include full system replacement on a 10-year cycle at approximately $50,000 per system. Additional 'fixed' O&M expenses include the replacement of chemical pump replacement on a 10-year cycle at $4,500 per pump. Based on these assumptions, the total 30-year life cycle cost for a batch dry ammonium sulfate system is $912,000. Table 7.7 summarizes the systems life cycle costs.

Table 7.7 Life Cycle Costs – Dry Ammonium Sulfate

Ammonia System	A	B	C	D	E = A + B + D
	Capital Costs	**Fixed O&M Expense**	**Present Annual O&M Expense**	**Annual O&M Expense Over 30 years**	**Capital + O&M Expense**
Dry Ammonium Sulfate	$70,000	$304,000	$11,301	$538,000	$912,000

7.3.4 Ammonia Alternative Selection

The following sections compare the three ammonia alternatives previously described. Based on the life cycle cost comparison and other factors associated with each alternative, the Project Team assumed a liquid ammonium sulfate system for inclusion in the preliminary base facilities.

7.3.4.1 *Ammonia Alternative Cost Comparison*

The 30-year life cycle evaluation compared the capital and estimated O&M cost expenditures for each ammonia alternative. As noted previously, the annual O&M expenses over the 30-year period account for a 3% annual inflation. This does not account for potentially variable chemical costs in the future.

Table 7.8 summarizes the life cycle cost evaluation. Recall that all values assume an ammonia dosage of 0.3 mg/L for peak chloramination and a plant capacity of 3,000 gpm for WTP No. 5.

Table 7.8 Summary of Ammonia Alternative Life Cycle Costs

Ammonia System	A	B	C	D	E = A + B + D
	Capital Costs	**Fixed O&M Expense**	**Present Annual O&M Expense**	**Annual O&M Expense Over 30 years**	**Capital + O&M Expense**
Anhydrous Ammonia	$181,000	$160,000	$7,887	$375,000	$716,000
Liquid Ammonium Sulfate	$20,000	$46,000	$21,231	$1,010,000	$1,076,000
Dry Ammonium Sulfate	$70,000	$304,000	$11,301	$538,000	$912,000

From a capital cost perspective, the anhydrous ammonia system is significantly more expensive than a liquid or dry ammonium sulfate system at $181,000. Capital costs associated with the ammonium sulfate systems range $20,000 to $70,000. Preceding sections discussed fixed O&M expenses associated with each option. Overall, it appears that the fixed O&M expenses anticipated for a liquid ammonium sulfate system ($46,000) are low compared to the fixed expenses ($160,000) for gaseous ammonia and ($304,000) for a dry ammonium sulfate over the 30-year life cycle.

Production-based O&M costs, or those primarily related to chemical acquisition, under each disinfection system vary significantly. Anhydrous ammonia incurs the least chemical O&M expenses due to small chemical quantities required for a relatively pure product ($375,000 over

30 years). Dry ammonium sulfate is nearly one and a half times as expensive ($538,000 over 30 years). A liquid ammonium sulfate system by far incurs the most chemical O&M expense for delivery of 35% liquid ammonium sulfate ($1,010,000 over 30 years).

The life cycle cost evaluation indicates that a gaseous ammonia system is the least expensive system over the 30-year life cycle period, with an estimated combined capital and O&M expense of $716,000. A dry ammonium sulfate system is more expensive with an estimated combined capital and O&M expense of $912,000. A liquid ammonium sulfate delivery system is most expensive over the life cycle, primarily due to the annual chemical expense, with an estimated combined capital and O&M expense of approximately $1,076,000.

It is important to note that the Project Team based the life cycle costs on the best preliminary data available at this time and subsequent planning level cost analyses; as such, the estimates are relative figures used as a basis for comparison.

7.3.5 Additional Evaluation Factors

Delivery and storage of gaseous ammonia poses a significant safety and health concern, especially in light of the potential location of the proposed Edina WTP No. 5 in very close proximity to a shopping center, office buildings, and community recreational areas. The gaseous ammonia system includes an ammonia scrubber for emergency protection per the NFPA fire code. The City currently uses gas chlorine at other facilities, so operators are familiar with gas injection systems. Gas ammonia systems require continuously softened carrier water otherwise the chemical injector scales up from water hardness. The water softening system adds another O&M component to the system, making it less desirable than a simple bulk liquid chemical feed system.

Operator safety is a concern with the dry ammonium sulfate batch system. Powder chemicals pose inhalation risks for operators during chemical batching, typically requiring use of personal protective equipment (PPE). The life cycle analysis also does not account for the time required for operations staff to batch the dry ammonium sulfate solution. This chemical typically comes in 50 pound bags and with the daily feed rate of 44 PPD, a dry ammonium sulfate system requires daily batching by operations staff. This factor alone makes a dry ammonium sulfate system very undesirable for the City of Edina.

Anhydrous ammonia and dry ammonium sulfate systems require many process components to operate and maintain in comparison to a simple bulk liquid ammonium sulfate delivery and feed system. Bulk delivery of liquid ammonium sulfate provides a safer ammonia option for the proposed WTP, but the increase of the long term chemical expense does prove to be a long term disadvantage.

The Project Team recommends implementation of a liquid ammonium sulfate system in the new WTP based on ease of operation, maintenance, and non-degradation of the chemical. The long term annual O&M expenses are less desirable than other ammonia alternatives, but the lower initial capital investment and smaller fixed O&M costs make this system a favorable option for WTP No. 5. It is important to note that future Well No. 21 may influence the required ammonia feed. If the raw water is high in ammonia, WTP No. 5 ammonia feed requirements will decrease, and if raw water ammonia is low, will increase ammonia feed.

7.4 Pre-Oxidation Alternatives

As noted in Section 5.1, there are multiple options to achieve pre-oxidation of raw water. Both aeration and chemical oxidation are possible alternatives. The pilot study evaluated both alternatives and this life cycle cost analysis compares the use of induced draft aeration and chlorine as pre-oxidants for iron removal. The pilot study confirmed manganese and any remaining iron removal by permanganate and Section 7.5.1 details life cycle costs associated with this chemical. This section only compares aeration and chlorine for the purposes of determining whether aeration is necessary for inclusion in the preliminary base facilities for WTP No. 5.

7.4.1 Capital Costs

Normal installation of aerators is inside of the WTP to prevent freezing. They are large pieces of equipment, often 10 ft. cubed or larger. Edina WTP No. 5 would need two (2) aeration units sized to handle approximately 1,500 gpm each, requiring an estimated capital cost of $350,000 for the aeration equipment and electrical and mechanical upgrades.

The use of chlorine or sodium permanganate will be included regardless of the pre-oxidation processes chosen, so an initial capital investment for these chemical feeds is not included as a separate line item.

7.4.2 O&M Costs

As noted in Chapter 6 that describes the pilot study results, aeration reduces the chlorine demand by approximately 0.1 mg/L FAC. This reduction is primarily due to iron oxidation and potential stripping of hydrogen sulfide through the aeration process. Without aeration, increased chlorine dose meets these chlorine demands.

Section 7.2.1 estimates annual chlorine costs for WTP No. 5 of $13,606 for gas chlorine. The addition of aeration would reduce this chlorine cost by approximately $600 annually.

Operating costs for the aeration units include the power draw of the aerator blower system. Preliminary sizing estimates 54 kW hours of power consumption per day, equating to a present day value of $800 per year based on an estimated $0.04 per kWh price. Comparing the power consumption of $800 per year to the $600 per year additional chlorine consumption makes aeration less feasible in terms of annual O&M costs.

7.4.3 Life Cycle Cost Comparison

Comparing life cycle costs for aeration versus additional chlorine demand on a 30-year planning period and an assumed 3% annual inflation rate indicates that aeration is a less feasible option for WTP No. 5. Total annual O&M expenses account for inflation over the planning period.

Table 7.9 summarizes the life cycle costs evaluated for aeration and additional chlorine feed.

Table 7.9 Life Cycle Costs – Aeration vs. Additional Chlorine

Pre Oxidation System	A	B	C	D = A + C
	Capital Costs	**Present Annual O&M Expense**	**Annual O&M Expense Over 30 years**	**Capital + O&M Expense**
Aeration	$350,000	$1,600	$76,000	$426,000
Chlorine	$0	$600	$29,000	$29,000

Comparing these costs indicates that the more feasible option for WTP No. 5 is feeding the additional chlorine. The proposed WTP No. 5 base facility includes a gas chlorine system for disinfection purposes, regardless of addition of an aerator. Calculations indicate that the breakeven additional chlorine dose required to offset the aerator life cycle costs is 1.5 mg/L FAC. Based on preliminary bench scale testing and the pilot study, aeration only reduced the chlorine demand by approximately 0.1 mg/L. Based on this comparison, the Pilot Team does not recommend inclusion of aerators for pre-oxidation at this time.

7.5 Additional Treatment Chemicals

This section discusses the selection of additional treatment chemicals associated with WTP No. 5 and provides estimated life cycle costs for expected expenses over a 30-year planning period, factoring in a 3% inflation rate.

7.5.1 Permanganate

Section 5.1.2 discussed permanganate options, which include potassium permanganate and sodium permanganate. While both options are feasible for WTP No. 5, potassium permanganate is a dry form of the chemical and requires dissolving in a batch tank prior to dosing to the water. To eliminate batching requirements, which increase operator's time commitment to one chemical and pose inherent safety risks associated with dry, strong oxidizing chemicals, the Project Team recommends the use of sodium permanganate delivered in a pre-mixed, bulk solution. While chemical acquisition costs are more expensive with this option, the simplicity of O&M and enhanced operator safety components of the bulk solution system make it the preferred option for WTP No. 5.

7.5.1.1 *Capital Costs*

Estimation of the capital cost associated with bulk sodium permanganate feed and storage equipment is $15,000 (2017 dollars). This cost represents 755-gallons of bulk storage, peristaltic or diaphragm chemical applications pumps, ultrasonic tank level monitoring equipment and readouts, associated piping, valves, and appurtenances, and an injection quill. The estimates do not include miscellaneous safety requirements, contingencies or engineering, administrative and legal costs, but Chapter 11 estimates do include these costs.

7.5.1.2 *O&M Costs*

O&M costs associated with a bulk $NaMnO_4$ system primarily consist of the cost for chemical acquisition. For the purposes of this analysis, O&M costs do not include routine maintenance of the system, assuming relatively similar annual maintenance time required for all considered alternatives.

Based on bench scale testing and the pilot study, optimum permanganate dose for the raw water is between 0.4 and 0.7 mg/L, which differed depending on which chemicals operated simultaneously and detention time. Assuming a base facility including 30 minutes of detention, a conservative permanganate dose is 0.5 mg/L. At the 3,000 gpm plant production capacity, WTP No. 5 requires approximately 4,054 gallons of 20% $NaMnO_4$ annually. Using the current approximate price for bulk 20% $NaMnO_4$ of $17.00 per gallon, the estimated annual O&M cost associated with maintaining an adequate supply of bulk solution onsite is $68,912. With annual inflation of 3% per year, the future expense value of the annual O&M expense over a 30-year planning period is $3,279,000.

Fixed O&M expenses associated with a bulk $NaMnO_4$ system involves the replacement of liquid chemical metering pumps every 10 years at $4,500 each and polyethylene storage tank replacement every 7 years at $2,000 per tank. The total 'fixed' O&M cost inflates future expenses at a 3% annual rate up to the anticipated year of expense.

Based on these assumptions, the total 30-year life cycle cost for a bulk sodium permanganate system is $3,333,000. Table 7.10 summarizes the life cycle costs for this system.

Table 7.10 Life Cycle Costs – Bulk Sodium Permanganate

Chemical Feed System	A	B	C	D	E = A + B + D
	Capital Costs	**Fixed O&M Expense**	**Present Annual O&M Expense**	**Annual O&M Expense Over 30 years**	**Capital + O&M Expense**
Bulk Sodium Permanganate	$15,000	$39,000	$68,912	$3,279,000	$3,333,000

7.5.2 HMO

Radium removal technology selected for inclusion in WTP No. 5 is preformed HMO based on operator familiarity with Edina's other treatment facilities and ease of O&M.

7.5.2.1 *Capital Costs*

Estimation of the capital cost associated with an HMO feed and storage system is $80,000 (2017 dollars). This cost represents the feed panel and control system, two 1000 gallon tanks with mixers and stands, and associated valves, piping and appurtenances. The estimates do not include miscellaneous safety requirements, contingencies or engineering, administrative and legal costs, but Chapter 11 estimates do include these costs.

7.5.2.2 *O&M Costs*

O&M costs associated with an HMO system primarily consist of the cost for chemical acquisition. For the purposes of this analysis, O&M costs do not include routine maintenance of the system, assuming relatively similar annual maintenance time required for all considered alternatives.

Based on the pilot study, an HMO dose of 1.0 mg/L paired with permanganate addition and extended detention time and conventional sand and anthracite media reduces combined radium to 75% of the regulated MCL and gross alpha to 80% of the regulated MCL. Treatment target goals aim to reduce effluent combined radium and gross alpha concentrations to at least half of the MCL of 5.0 pCi/L and 15.0 pCi/L respectively.

At the City's existing facilities that treat for radium, operators have the HMO system set to dose 0.2 mg/L of HMO. The City should optimize the required dose for consistent removal of at least half the regulated MCL. As a conservative estimate, the O&M costs assume an HMO dose of 0.7 mg/L.

To understand the cost implications of an HMO system, the Project Team conducted a life cycle cost analysis. Assuming an HMO dose of 0.7 mg/L, at the 3,000 gpm plant production capacity, WTP No. 5 requires approximately 34,054 gallons of preformed HMO annually. Using the current approximate price for preformed HMO of $13.00 per gallon, the estimated annual O&M cost associated with maintaining an adequate supply of bulk solution onsite is $442,708. With annual inflation of 3% per year, the future expense value of the annual O&M expense over a 30-year planning period is $21,062,000.

Fixed O&M expenses associated with an HMO system involves an assumed full replacement of the system every 10 years at $70,000 each. The total 'fixed' O&M cost inflates future expenses at a 3% annual rate up to the anticipated year of expense.

Based on these assumptions, the total 30-year life cycle cost for an HMO system is $21,532,000. Table 7.11 summarizes the life cycle costs for this system.

Table 7.11 Life Cycle Costs – Preformed HMO

Chemical Feed System	A	B	C	D	E = A + B + D
	Capital Costs	**Fixed O&M Expense**	**Present Annual O&M Expense**	**Annual O&M Expense Over 30 years**	**Capital + O&M Expense**
HMO	$80,000	$390,000	$442,708	$21,062,000	$21,532,000

7.5.3 Fluoride

7.5.3.1 *Capital Costs*

Estimation of the capital cost associated with a fluoride feed and storage system is $20,000 (2017 dollars). This cost represents a 450-gallon storage tank, 100-gallon day tank, transfer pump and chemical metering pumps, a weight scale, and associated valves, piping and appurtenances. The estimates do not include miscellaneous safety requirements, protective coatings, contingencies or engineering, administrative and legal costs, but Chapter 11 estimates do include these costs.

7.5.3.2 *O&M Costs*

O&M costs associated with a fluoride system primarily consist of the cost for chemical acquisition. For the purposes of this analysis, O&M costs do not include routine maintenance of the system, assuming relatively similar annual maintenance time required for all considered alternatives.

Based on the existing well raw water fluoride concentrations of approximately 0.2 mg/L and a desired 0.7 mg/L target residual, the proposed feed rate for the future facility is 0.5 mg/L fluoride. At the 3,000 gpm plant production capacity, WTP No. 5 requires approximately 3,607 gallons of hydrofluorosilicic acid annually. Using the current approximate price of $3.00 per gallon, the estimated annual O&M cost associated with maintaining an adequate supply of bulk solution onsite is $10,821. With annual inflation of 3% per year, the future expense value of the annual O&M expense over a 30-year planning period is $515,000.

Fixed O&M expenses associated with a bulk fluoride system involves the replacement of liquid chemical metering pumps every 10 years at $4,500 each and polyethylene storage tank replacement every 7 years at $1,700 per tank. The total 'fixed' O&M cost inflates future expenses at a 3% annual rate up to the anticipated year of expense.

Based on these assumptions, the total 30-year life cycle cost for the fluoride system is $572,000. Table 7.12 summarizes the life cycle costs for this system.

Table 7.12 Life Cycle Costs – Fluoride

	A	B	C	D	E = A + B + D
Chemical Feed System	**Capital Costs**	**Fixed O&M Expense**	**Present Annual O&M Expense**	**Annual O&M Expense Over 30 years**	**Capital + O&M Expense**
Fluoride	$20,000	$37,000	$10,821	$515,000	$572,000

7.5.4 Orthophosphate / Polyphosphate Blend

7.5.4.1 *Capital Costs*

Estimation of the capital cost associated with an orthophosphate / polyphosphate blend feed and storage system is $12,000 (2017 dollars). This cost represents a 155-gallon storage tank, chemical metering pumps, and associated valves, piping and appurtenances. The estimates do not include miscellaneous safety requirements, contingencies or engineering, administrative and legal costs, but Chapter 11 estimates do include these costs.

7.5.4.2 *O&M Costs*

O&M costs associated with an ortho/poly blend system primarily consist of the cost for chemical acquisition. For the purposes of this analysis, O&M costs do not include routine maintenance of the system, assuming relatively similar annual maintenance time required for all considered alternatives.

The corrosion control chemical blend assumed for WTP No. 5 is consistent with the 50/50 ortho/poly blend used throughout the rest of the City's system. Annual chemical acquisition costs assume a 1.3 mg/L as orthophosphate dose. At the 3,000 gpm plant production capacity, WTP No. 5 requires approximately 1,485 gallons of ortho/poly blend annually. Using the current approximate price of $5.00 per gallon, the estimated annual O&M cost associated with maintaining an adequate supply of bulk solution onsite is $7,424. With annual inflation of 3% per year, the future expense value of the annual O&M expense over a 30-year planning period is $353,000.

Fixed O&M expenses associated with the corrosion control system involves the replacement of liquid chemical metering pumps every 10 years at $4,500 each and polyethylene storage tank replacement every 7 years at $1,000 per tank. The total 'fixed' O&M cost inflates future expenses at a 3% annual rate up to the anticipated year of expense.

Based on these assumptions, the total 30-year life cycle cost for the ortho/poly blend system is $397,000. Table 7.13 summarizes the life cycle costs for this system.

Table 7.13 Life Cycle Costs – Orthophosphate / Polyphosphate Blend

Chemical Feed System	A	B	C	D	E = A + B + D
	Capital Costs	**Fixed O&M Expense**	**Present Annual O&M Expense**	**Annual O&M Expense Over 30 years**	**Capital + O&M Expense**
Ortho/Poly Blend	$12,000	$32,000	$7,424	$353,000	$397,000

7.6 Summary of Treatment Chemical Life Cycle Costs

This section summarizes the life cycle costs for all five (5) selected chemical systems for the base facilities designed for WTP No. 5. Chemicals include gas chlorine, liquid ammonium sulfate, sodium permanganate, fluoride, and an ortho/polyphosphate blend.

Table 7.14 provides the life cycle cost summary.

Table 7.14 Summary of Selected Treatment Chemical Life Cycle Costs

Chemical Feed System	A	B	C	D	E = A + B + D
	Capital Costs	**Fixed O&M Expense**	**Present Annual O&M Expense**	**Annual O&M Expense Over 30 years**	**Capital + O&M Expense**
Gaseous Chlorine	$252,000	$260,000	$13,606	$647,000	$1,159,000
Liquid Ammonium Sulfate	$20,000	$46,000	$21,231	$1,010,000	$1,076,000
Sodium Permanganate	$15,000	$39,000	$68,912	$3,279,000	$3,333,000
HMO	$80,000	$390,000	$442,708	$21,062,000	$21,532,000
Fluoride	$20,000	$37,000	$10,821	$515,000	$572,000
Ortho/Poly Blend	$12,000	$32,000	$7,424	$353,000	$397,000
Total	**$399,000**	**$804,000**	**$564,702**	**$26,866,000**	**$28,069,000**

As presented above, the gaseous chlorine system and the HMO feed system account for the majority of chemical feed system capital costs and fixed O&M expenses. Liquid ammonium sulfate and sodium permanganate account for large portions of the annual O&M expense. Above all, the HMO system contributes the largest annual O&M expense due to higher chemical acquisition costs associated with this product. The Project Team recommends optimization of all chemical feed systems to reduce annual operating costs.

7.7 Filtration System Alternatives

Selecting filtration alternatives involves a choice of filter media and a type of filter system (pressure vs. gravity). The pilot study results affirmed the use of a dual filter media, consisting of anthracite coal and standard silica sand, which is consistent regardless of the selected filter type. Chapter 11 details the capital costs associated with the selection of a gravity or pressure filter system. This cost comparison looks at the life cycle fixed and annual O&M costs over an industry standard, 30-year planning period assuming a 3% annual inflation.

7.7.1 O&M Costs

Operation and maintenance costs related to the two filtration technologies differ. Preliminary estimates indicate that steel pressure filter construction requires regular touch-up and continued maintenance than concrete, which requires almost no annual maintenance.

Typical pressure filters require media replacement, air scour and underdrain maintenance after approximately 15 years of use, totaling $140,000 (2017 dollars). The service life of a steel pressure filter is approximately 30 years at which time the filter requires a more in-depth refurbishment. This refurbishment typically consists of media removal, underdrain replacement, sand blasting, and re-painting of the entire structure. This refurbishment at the 30-year period would cost roughly $420,000 (2017 dollars). Annual steel maintenance at $2,000 per year amounts to an additional $95,000 over the course of the 30-year period.

Comparatively, a gravity filtration system requires almost no maintenance in the first 15 years of life other than a media replacement at approximately $40,000 (2017 dollars). The filter troughs have stainless steel construction so limited corrosion occurs. The underdrain system may require replacement within 30 years, but inspection and select repair may occur with a media replacement at the 15 year checkpoint. Concrete wear and tear should be very limited in the next 15 years and should have an expected service life of 50 years or more. The media, support gravel, and complete underdrain replacement cost is approximately $280,000 (2017 dollars)

As summarized in Table 7.15 gravity filtration is less expensive over a 30-year lifespan based on 2017 dollars. Gravity filtration versus pressure filtration realizes a savings of nearly $433,000.

Table 7.15 Summary of Filtration Alternatives O&M Life Cycle Costs

Filter O&M Analysis (2047)		
Item Description	**Pressure Filter**	**Gravity Filter**
Life Cycle Period (years)	30	30
15 Yr. - Media and Support Gravel Replacement	$62,000	$62,000
15 Yr. - Air Scour and Underdrain Nozzle Replacement	$156,000	$0
30 Yr. - Media and Support Gravel Replacement w/ Complete Steel Rehab	$534,000	$97,000
30 Yr. - Complete Underdrain Replacement	$328,000	$583,000
O&M - Annual Steel Repair ($2,000/year)	$95,000	$0
TOTAL O&M EXPENSE VALUE OVER 30 YEARS	*$1,175,000*	*$742,000*

7.8 Backwash Reclaim System Alternatives

Selecting a backwash reclaim alternative for WTP No. 5 depends on the space available for backwash reclaim tanks. In some cases, when site size limitations exist, an above ground plate settler becomes a cost effective alternative. The following sections provide a capital, O&M, and life cycle cost analysis for traditional backwash reclaim versus an above ground plate settler system.

7.8.1 Traditional Backwash Reclaim System

7.8.1.1 *Capital Costs*

Estimation of the capital cost associated with a traditional backwash reclaim system is $835,000 (2017 dollars). This cost includes the construction of the tanks, reclaim and sludge pumps, and associated piping, valves, appurtenances, and basin wash down systems. The capital costs associated with excavation were not include because these costs vary significantly on a site by site basis based on site footprint limitations. The estimates also do not include contingencies or engineering, administrative and legal costs, but Chapter 11 estimates do account for these costs. It is important to note that capital costs associated with the tank construction are part of the concrete division in the Chapter 11 estimates. For this reason, the capital costs listed in Table 7.16 do not match those listed in Chapter 11, Division 46 subtotals.

7.8.1.2 *O&M Costs*

O&M costs associated with a traditional backwash reclaim system are limited. Preliminary evaluations indicate that a traditional backwash recovery tank requires almost no maintenance in the first 15 years. Fixed O&M costs include approximately $40,000 every 15 years for reclaim and sludge pump replacement and $2,000 every 5 years for miscellaneous maintenance on the basin wash down system and tanks.

Another fixed O&M expense during the initial connection of the backwash reclaim system to the sanitary system is the Sewer Availability Charge (SAC). Metropolitan Council defines one SAC unit as 274 gallons of maximum potential daily wastewater flow. Assuming WTP No. 5 will backwash one filter per day at 30,000 gallons of backwash and an 85% backwash water recovery with this reclaim system, the facility will produce approximately 4,500 gallons of wastewater flow per day, or 18.2 SAC units. The rate for one SAC unit in 2017 is $2,485, so the fixed O&M cost for the wastewater connection is $45,300. Due to the typical concentrations of total suspended solids (TSS) and chemical oxygen demand (COD) present in a traditional backwash reclaim system sludge, an annual strength charge is not likely for this type of system.

Concrete wear and tear should be very limited in the next 15 years and should have an expected service life of 50 years or more. At that time, periodic inspections may be necessary to verify structural integrity and possible repair.

Based on these assumptions, the total 30-year life cycle cost for a traditional backwash reclaim system is $1,060,000. Table 7.16 summarizes the life cycle costs for a traditional backwash reclaim system.

Table 7.16 Life Cycle Costs – Traditional Backwash Reclaim System

	A	B	C	D	E = A + B + D
Chlorine System	**Capital Costs**	**Fixed O&M Expense**	**Present Annual O&M Expense**	**Annual O&M Expense Over 30 years**	**Capital + O&M Expense**
Traditional Backwash System	$835,000	$225,000	$0	$0	$1,060,000

7.8.2 Above Grade Plate Settler Reclaim System

7.8.2.1 *Capital Costs*

Estimation of the capital cost associated with an above grade plate settler system is $835,000 (2017 dollars). This cost includes the construction of the reclaim tank, a tank recirculation system, a pumping system to send reclaim water through the plate settler, the above grade plate settler unit, additional building materials to house the system, and associated piping, valves, appurtenances, and basin wash down systems. The capital costs associated with excavation were not include because these costs vary significantly on a site by site basis based on site footprint limitations. The estimates also do not include contingencies or engineering, administrative and legal costs, but Chapter 11 estimates do account for these costs. It is important to note that capital costs associated with the tank construction and additional building materials are part of the concrete and masonry divisions in the Chapter 11 estimate for Option 1C. For this reason, the capital costs listed in Table 7.17 do not match those listed in Chapter 11, Division 46 subtotals.

7.8.2.2 *O&M Costs*

Fixed O&M costs include approximately $20,000 every 15 years for pump replacement and $2,000 every 5 years for miscellaneous maintenance on the basin recirculation system and tanks. In addition, fixed costs include $2,000 annually for steel repair. Concrete wear and tear should be very limited in the next 15 years and should have an expected service life of 50 years or more. At that time, periodic inspections may be necessary to verify structural integrity and possible repair.

Another fixed O&M expense during the initial connection of the backwash reclaim system to the sanitary system is the Sewer Availability Charge (SAC). Metropolitan Council defines one SAC unit as 274 gallons of maximum potential daily wastewater flow. Assuming WTP No. 5 will backwash one filter per day at 30,000 gallons of backwash and a 95% backwash water recovery with this reclaim system, the facility will produce approximately 1,500 gallons of wastewater flow per day, or 6.1 SAC units. The rate for one SAC unit in 2017 is $2,485, so the fixed O&M cost for the wastewater connection is $15,200.

Annual O&M costs associated with an above grade plate settler system are limited to the costs of polymer chemical acquisition and the strength charge for the concentrated wastewater effluent produced by this system. Assuming the system requires a small amount of polymer for coagulation enhancement, the estimated annual polymer chemical acquisition cost is $2,000. The strength charge formula used by Metropolitan Council is below:

Strength Charge = [V * (TSS – 250) * 8.34 * TSS Rate] + [V * (COD – 500) * 8.34 * COD Rate]

Where V is equal to the volume of discharge in millions of gallons, TSS and COD are the effluent concentrations in mg/L, the TSS Rate is equal to $0.22 per pound of excess TSS, the COD Rate is equal to $0.11 per pound of excess COD, and 8.34 is the conversion factor for mg/L and gallons to pounds. Using an average plate settler sludge TSS concentration of 2,000 mg/L and COD concentration of 900 mg/L, the annual strength charge in 2017 dollars is $2,100. With annual inflation of 3% per year, the future expense value of the annual O&M expense over a 30-year planning period is $195,000.

Based on these assumptions, the total 30-year life cycle cost for an above grade plate settler system is $986,000. Table 7.17 summarizes the life cycle costs for the system.

Table 7.17 Life Cycle Costs – Above Grade Plate Settler Reclaim System

Chlorine System	A	B	C	D	E = A + B + D
	Capital Costs	**Fixed O&M Expense**	**Present Annual O&M Expense**	**Annual O&M Expense Over 30 years**	**Capital + O&M Expense**
Above Grade Plate Settler System	$580,000	$211,000	$4,100	$195,000	$986,000

7.8.3 Life Cycle Cost Comparison

Comparing life cycle costs for the two analyzed backwash reclaim systems on a 30-year planning period and an assumed 3% annual inflation rate indicates that an above grade plate settler system is a more cost efficient option for WTP No. 5. Total annual O&M expenses account for inflation over the planning period.

Table 7.18 summarizes the life cycle costs evaluated for traditional backwash reclaim and above grade plate settler reclaim systems.

Table 7.18 Life Cycle Costs – Aeration vs. Additional Chlorine

Backwash Reclaim System	A	B	C	D	E = A + B + D
	Capital Costs	**Fixed O&M Expense**	**Present Annual O&M Expense**	**Annual O&M Expense Over 30 years**	**Capital + O&M Expense**
Traditional Backwash Reclaim	$835,000	$225,000	$0	$0	$1,060,000
Above Grade Plate Settler System	$580,000	$211,000	$4,100	$195,000	$986,000

Comparing these costs indicates that the more feasible option for WTP No. 5 is an above grade plate settler system. The majority of the base facilities include traditional backwash reclaim systems because the site has the space available to install traditional reclaim tanks. For sites with small footprints, the above grade plate settler system may realize additional cost savings by minimizing excavation and the required shoring system to accommodate steep slopes. The total probable construction cost estimates provided in Chapter 11 factor in these additional cost tradeoffs.

CHAPTER 8 SITE ALTERNATIVES

8.1 Introduction

Siting and planning for WTP No. 5 began over a decade ago. Since then, the City, with their consultants, completed a feasibility study in 2007, conducted a water system demand and capacity analysis in 2013, and developed various architectural concepts over the years, all related to WTP No. 5. The City also secured easements and partially extended raw water pipes to the preferred site property adjacent the Southdale Tower, known as the Southdale Site.

With economic development and environmental sustainability in mind, the City determined alternative sites for consideration in the preliminary design report. Originally, these sites included the Yorktown Site, located near the YMCA and Fire Station No. 2, and the Median Site, located along West 69th Street, directly east of the wellhouse of Well No. 5. Throughout the preliminary design report (PDR) process, the City added a fourth site, the Fred Richards Site, located immediately adjacent to the existing WTP No. 3, which would potentially take the place of WTP No. 3 in the future.

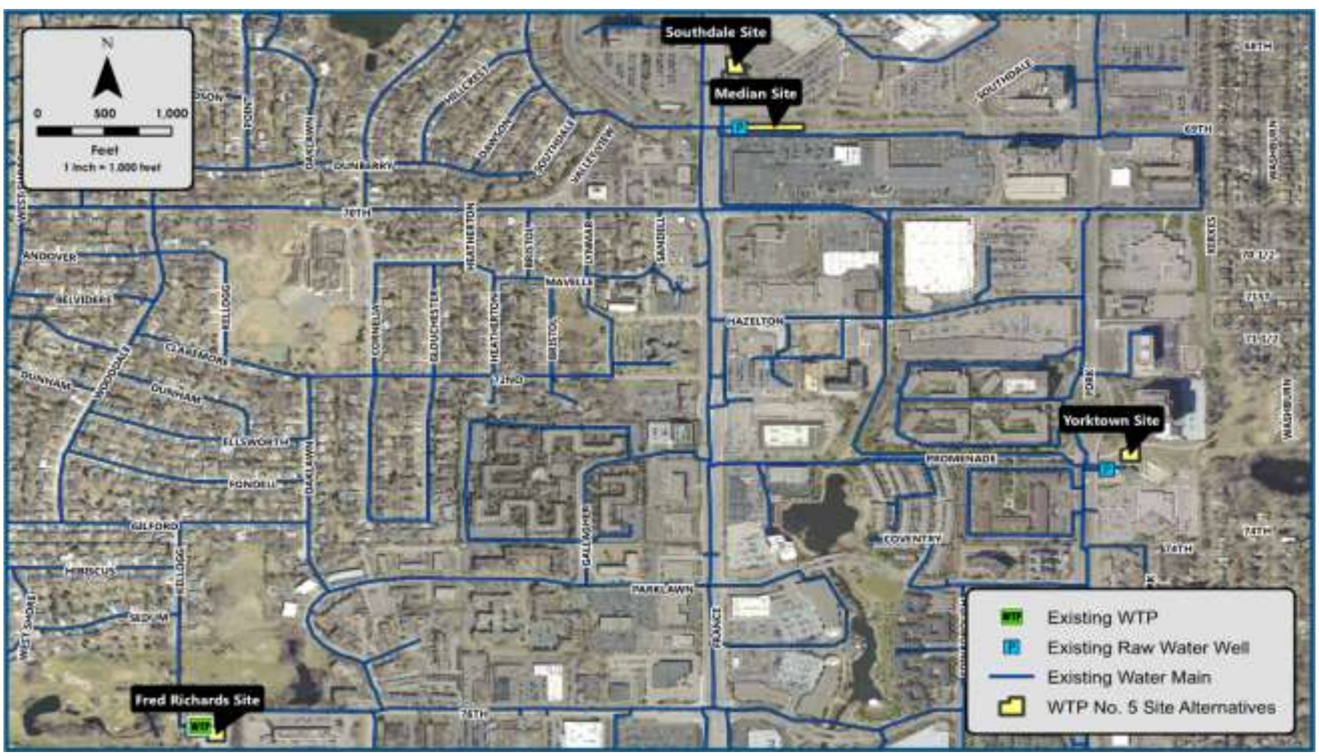

Figure 8.1 Overview of WTP No. 5 Site Alternatives

Figure 8.1 provides an overview of the four (4) sites considered in this PER. The well located near the Southdale and Median Sites is Well No. 5 and the well located near the Yorktown Site is Well No. 18. The existing facility adjacent the Fred Richards Site is WTP No. 3, which includes Well No. 10 and No. 11.

The following sections briefly describe the alternatives and introduce the base facility layouts developed for each site. All sites assume that Well No. 5, No. 18 and future Well No. 21 will connect into WTP No. 5, providing a 3,000 gpm ultimate plant capacity. The firm capacity of the plant is technically 2,000 gpm, or equal to the plant's capacity with one (1) filter offline or in backwash. Each site includes a gravity and pressure option, with the only exception being the Median Site that can only accommodate a pressure filter facility.

Chapter 9 provides details related to facility integration into the City's existing system, Chapter 10 evaluates non-financial considerations for each site, and Chapter 11 provides capital cost estimations for every alternative. Chapter 12 then uses the technical, non-financial, and financial evaluation results to select a preferred site alternative and preliminary base facility design for WTP No. 5.

8.2 Option 1 – Southdale Site

The City identified the Southdale Site as the preferred alternative since the early stages of planning for WTP No. 5. This site is located just north of the Southdale Tower in an existing parking lot that totals just under a half-acre of land for facility construction.

The site offers a unique location for a water treatment facility in a highly commercial area planned for extensive re-development in the near future. Initial visions of the City and the design team included a shared-use facility integrating the water treatment plant into a mixed use commercial and residential building. As the PDR process progressed, the Southdale Mall owners, Simon Properties, Inc., acknowledged that every lease holder within the mall has access to the frontage road that surrounds the north and east Southdale Site boundaries, so making adjustments to the frontage road requires approval by all lease holders. This circumstance contributed to Simon Properties decision to deny moving forward with making plans for a shared-use facility at this site.

8.2.1 Option 1A

Option 1A is the Southdale Site with gravity filters. ***Appendix G*** provides a preliminary site layout and plan views of the upper and lower levels of the facility. Figure 8.2 depicts the general site requirements for Option 1A.

Below grade components include two (2) backwash reclaim tanks each sized to hold a backwash from each of the three (3) gravity filters, a pipe gallery, and a clearwell to store finished water prior to pumping into the distribution system.

Main level components include chemical feed rooms, high service and backwash supply pumps, office and lab space, a pipe gallery, and electrical and mechanical equipment. The main level extends upward for extra detention tank depth that provides 30 minutes of detention at the 3,000 gpm plant capacity, which flows by gravity into the three (3) 1,000 gpm filters. The upper level overlooks the pipe gallery and provides overhead views of the filters.

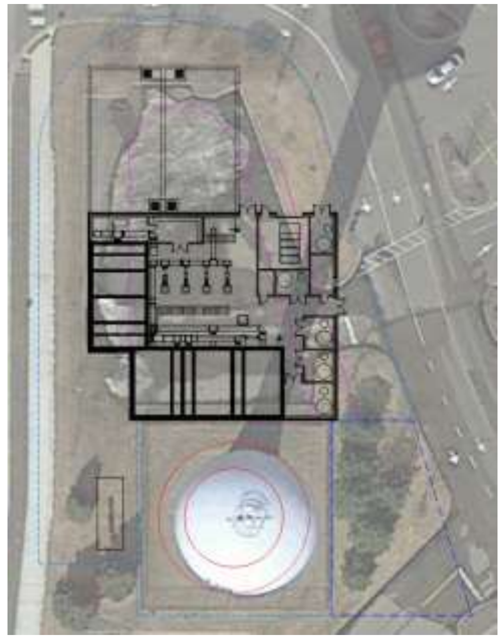

Figure 8.2 Option 1A – Southdale Site with Gravity Filters

8.2.2 Option 1B

Option 1B is the Southdale Site with pressure filters. ***Appendix H*** provides a preliminary site layout and plan views of the upper and lower levels of the facility. Figure 8.3 depicts the general site requirements for Option 1B.

Below grade components include a 30 minute detention tank, two (2) backwash reclaim tanks each sized to hold a backwash from each of the three (3) gravity filters, and a pipe gallery to house backwash reclaim system equipment.

Main level components include chemical feed rooms, post-detention pumps, office, lab, and lavatory space, three (3) 1,000 gpm pressure filters, an electrical room, a mechanical room, and a larger chemical delivery area. This option does not require an upper level for any of the currently proposed treatment technologies.

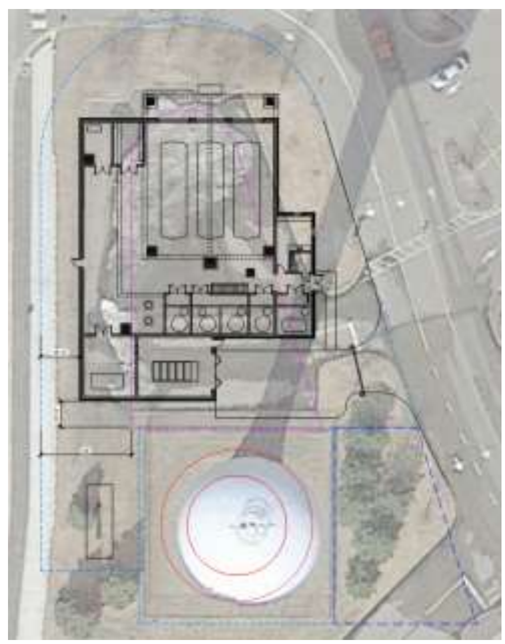

Figure 8.3 Option 1B – Southdale Site with Pressure Filters

8.2.3 Option 1C

Option 1C is the Southdale Site with gravity filters and an above ground plate settler backwash reclaim system. This option surfaced during base facility cost estimation because the Southdale Site's small footprint requires an extensive shoring system for installation of the below ground tanks that are part of the backwash reclaim system. With an above ground plate settler system, the plate settler equipment and smaller reclaim tank associated with this system eliminate the extensive shoring system costs.

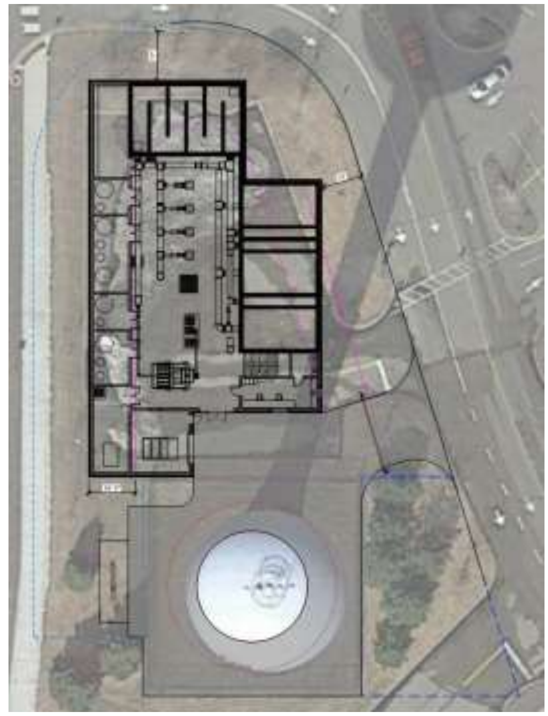

Figure 8.4 Option 1C – Southdale Site with Gravity Filters and Above Ground Plate Settlers

Appendix I provides a preliminary site layout and plan views of the upper and lower levels of the facility. Figure 8.4 depicts the general site requirements for Option 1B.

Below grade components include one (1) backwash reclaim tank sized to hold a backwash from two (2) of the three (3) gravity filters, a pipe gallery, and a clearwell to store finished water prior to pumping into the distribution system.

Main level components include chemical feed rooms, high service and backwash supply pumps, office and lab space, a pipe gallery, the above ground plate settler equipment and electrical and mechanical equipment. The main level extends upward for extra detention tank depth that provides 30 minutes of detention at the 3,000 gpm plant capacity, which flows by gravity into the three (3) 1,000 gpm filters. The upper level overlooks the pipe gallery and provides overhead views of the filters.

8.3 Option 2 – Yorktown Site

The second site alternative is the Yorktown Site located along York Avenue just north of the Southdale YMCA and Fire Station No. 2. The site, currently called Yorktown Park, is an open green space with trails and walking paths connected to other major parks within the Southdale Area, including the Edina Promenade, Centennial Lakes Park, and Adams Hill Park of Richfield, MN.

This site offers the opportunity to integrate the water treatment facility into the City's existing parks and recreation amenities by creating a trailhead or public water station feature. Additionally, the adjacent fire station makes the proposed facility's architecture fit in well with surrounding infrastructure.

8.3.1 Option 2A

Option 2A is the Yorktown Site with gravity filters. ***Appendix J*** provides a preliminary site layout and plan views of the upper and lower levels of the facility. Figure 8.5 depicts the general site requirements for Option 2A.

Below grade components include two (2) backwash reclaim tanks each sized to hold a backwash from each of the three (3) gravity filters, a pipe gallery, and a clearwell to store finished water prior to pumping into the distribution system.

Main level components include chemical feed rooms, high service and backwash supply pumps, office and lab space, a pipe gallery, and electrical and mechanical equipment. The main level extends upward for extra detention tank depth that provides 30 minutes of detention at the 3,000 gpm plant capacity, which flows by gravity into the three (3) 1,000 gpm filters. The upper level overlooks the pipe gallery and provides overhead views of the filters.

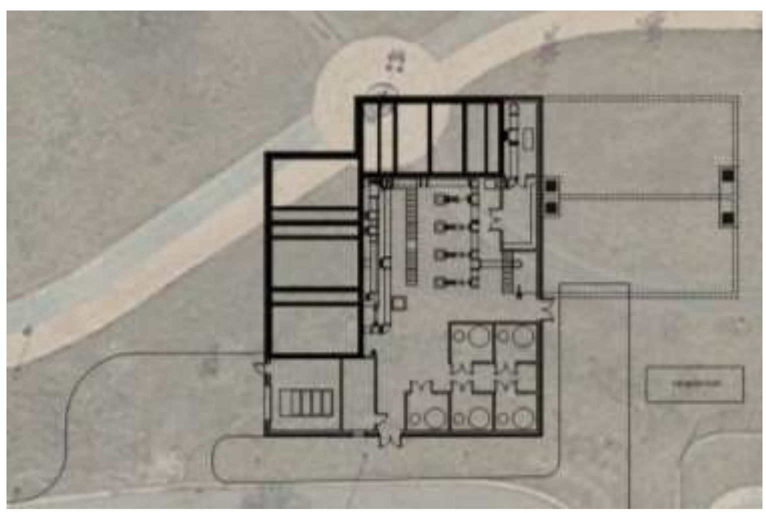

Figure 8.5 Option 2A – Yorktown Site with Gravity Filters

8.3.2 Option 2B

Option 2B is the Yorktown Site with pressure filters. ***Appendix K*** provides a preliminary site layout and plan views of the upper and lower levels of the facility. Figure 8.6 depicts the general site requirements for Option 2B.

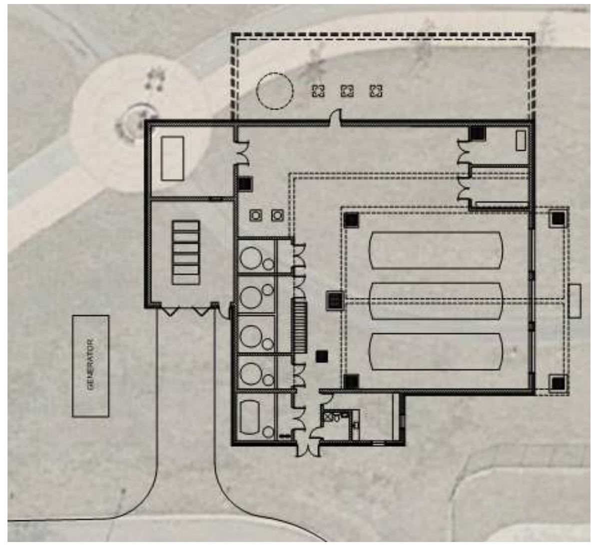

Figure 8.6 Option 2B – Yorktown Site with Pressure Filters

Below grade components include a 30 minute detention tank, two (2) backwash reclaim tanks each sized to hold a backwash from each of the three (3) gravity filters, and a pipe gallery to house backwash reclaim system equipment.

Main level components include chemical feed rooms, post-detention pumps, office, lab, and lavatory space, three (3) 1,000 gpm pressure filters, an electrical room, a mechanical room, and a large chemical delivery area. This option does not require an upper level for any of the currently proposed treatment technologies. This option also includes provisions for addition of future air stripping towers in the event that the VOC plume present at Well No. 7 migrates south far enough to influence raw water quality of Well No. 5 or No. 18. Air stripping towers would require a clearwell and high service pumps to pump finished water into the system.

8.4 Option 3 – Median Site

The third site alternative is the Median Site located within the median of South 69th Street, directly east of Well No. 5. This site offers the opportunity to integrate a water treatment facility into an area typically deemed undevelopable in a similar road corridor. This site is compact compared to the others, limiting the layout, size, and treatment flexibility of the facility.

8.4.1 Option 3A

Option 3A is the Median Site with pressure filters. ***Appendix L*** provides a preliminary site layout and plan views of the facility. Figure 8.7 depicts the general site requirements for Option 3A.

System components include two (2) backwash reclaim tanks each sized to hold a backwash from each of the three (3) gravity filters, a pipe gallery tunnel connecting the reclaim tanks to the main facility, chemical feed rooms, three (3) 1,000 gpm pressure filters, an electrical room, mechanical equipment, and a chemical delivery access lane.

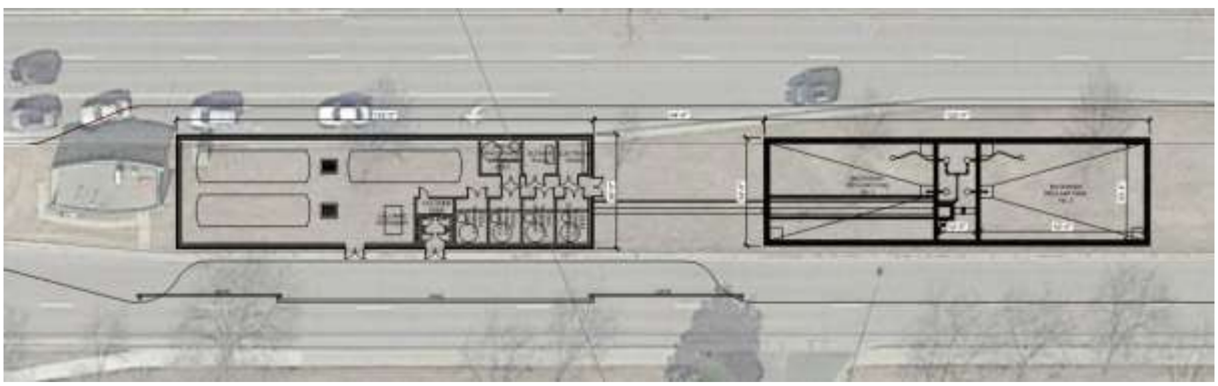

Figure 8.7 Option 3A – Median Site with Pressure Filters

8.5 Option 4 – Fred Richards Site

The fourth, and final, site alternative is the Fred Richards Site located near the golf course facilities that closed down back in 2014. The site includes WTP No. 3, which treats raw water from wells No. 10 and No. 11. This site became an alternative throughout the PDR process as City public works staff looked to streamline facility operations by adding to an existing plant, versus building a new facility in a separate location. Additionally, at first, the City saw the planning for re-development of the golf course into multi-use infrastructure including athletic fields, parks, restaurants, event space, and more as a unique opportunity for a shared-use facility for WTP No. 5. Unfortunately, recent approval of a master plan for the park without the footprint of the facility included decreases the likelihood of integrating the new facility into the shared-use development.

8.5.1 Option 4A

Option 4A is the Fred Richards Site with gravity filters. ***Appendix M*** provides a preliminary site layout and plan views of the upper and lower levels of the facility. Figure 8.8 depicts the general site requirements for Option 4A.

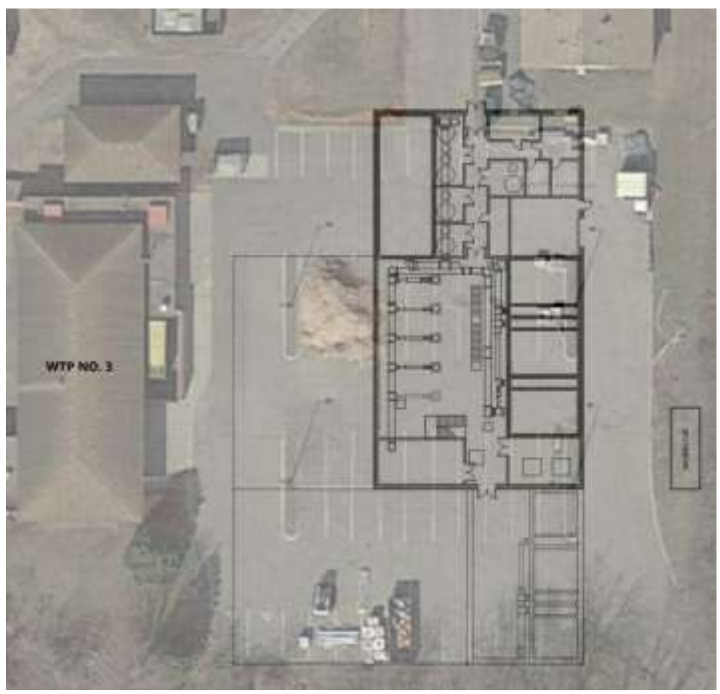

Figure 8.8 Option 4A – Fred Richards Site with Gravity Filters

Below grade components include two (2) backwash reclaim tanks each sized to hold a backwash from each of the three (3) gravity filters, a pipe gallery, a clearwell, and a pumping chamber to store finished water prior to pumping into the distribution system.

Main level components include chemical feed rooms, high service and backwash supply pumps, office, lab and lavatory space, a pipe gallery, and electrical and mechanical equipment. The main level extends upward for extra detention tank depth that provides 30 minutes of detention at the 3,000 gpm plant capacity, which flows by gravity into the three (3)

1,000 gpm filters. The upper level overlooks the pipe gallery and provides overhead views of the filters.

The site offers the opportunity to expand this 3,000 gpm facility into a 5,000 gpm facility once the City desires decommissioning of WTP No. 3. Future infrastructure includes two (2) additional backwash reclaim tanks, a clearwell extension, three (3) additional filters and a fourth high service pump.

8.5.2 Option 4B

Option 4B is the Fred Richards Site with pressure filters. ***Appendix N*** provides a preliminary site layout and plan views of the upper and lower levels of the facility. Figure 8.9 depicts the general site requirements for Option 4B.

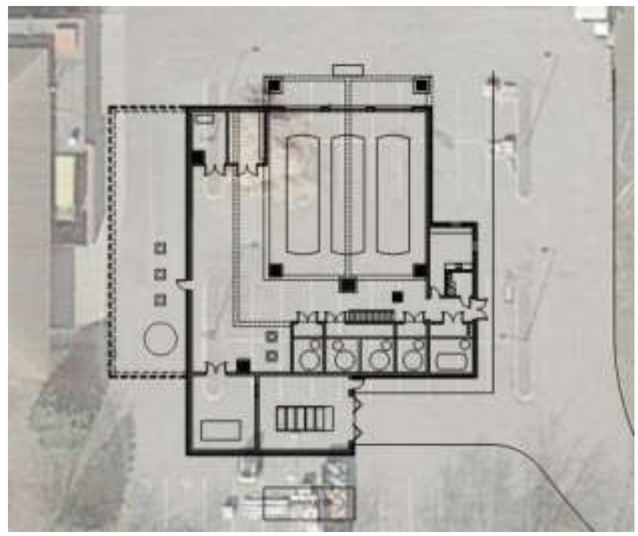

Figure 8.9 Option 4B – Fred Richards Site with Pressure Filters

Below grade components include a 30 minute detention tank, two (2) backwash reclaim tanks each sized to hold a backwash from each of the three (3) gravity filters, and a pump chase to house backwash reclaim system equipment.

Main level components include chemical feed rooms, post-detention pumps, office, lab, and lavatory space, three (3) 1,000 gpm pressure filters, an electrical room, a mechanical room, and a large chemical delivery area. This option does not require an upper level for any of the currently proposed treatment technologies. This option also includes provisions for addition of future air stripping towers in the event that the VOC plume present at Well No. 7 migrates south far enough to influence raw water quality of Well No. 5 or No. 18. Air stripping towers require a clearwell and high service pumps to pump finished water into the system. For this option, WTP No. 3 remains standalone from WTP No. 5.

CHAPTER 9 FACILITY INTEGRATION

Integration with the existing infrastructure is a critical part of this preliminary design process. Using the existing infrastructure that is functioning well will help to conserve costs and allow the City of Edina to spend money on infrastructure that brings long term value to its Utility. Currently, the City has two (2) raw water wells that will provide water to WTP No. 5. In addition to these wells, the new WTP will need to operate seamlessly with the distribution system. The Project Team must consider the impact to adjacent infrastructure such as the Southdale Tower, adjacent roads, and adjacent buildings.

9.1 Wells 5 and 18

The proposed WTP will require 1,000 gpm of water from each Well 5 and Well 18. Currently, Well No. 5 and No. 18 each produce 1,000 gpm and pump directly into the distribution system. The distribution system pressure in these areas operates between 90 and 100 psi. Integrating each of these wells into a new WTP may require modification to the well. This depends on the chosen WTP site and the filtration process. In addition, detention ahead of the filter will also influence the need to modify the well.

The new hydraulics each well must integrate in to determine the required well modifications. The components that factor into this calculation are:

1. The static elevation differences between each site, and the well water depth,
2. Minor losses caused by the additional pipeline that will be required to transport the water from the well to the WTP, and;
3. The final pressure that the well needs to pump against to filter the water or enter a detention tank.

Table 9.1 indicates the ground, pump setting, and pumping water elevations at each well.

Table 9.1 Well Site Elevations

Well	Site Elevation (ft.)	Settling Elevation (ft.)	Pumping Water Elevation (ft.)
Well No. 5	876	723	786
Well No. 18	862	742	759

Each WTP configuration will require different pumping requirements from Wells No. 5 and No. 18. Table 9.2 and Table 9.3 illustrate these head loss variations for Well No. 5 and Well No. 18 respectively. The abbreviation TDH stands for total dynamic head, which is the summation of the static head, well pipe and pump losses, filtration pressure converted to feet of water, and pipeline losses.

Table 9.2 Well No. 5 Hydraulic Analysis

WTP Site	Site Elevation (ft.)	WTP Inlet Elevation (ft.)	Static Head (ft.)	Well Pipe and Pump Losses (ft.)	Filtration Pressure (psi)	Pipeline Losses (ft.)	TDH Requirements
Southdale Gravity Filtration w/ Detention	880	898	112	10	0	2.7 (12"HDPE, 1,000gpm, 500ft)	124.7
Southdale Pressure Filtration w/ Detention	880	876	90	10	0 (Secondary Pumping)	2.7 (12"HDPE, 1,000gpm, 500ft)	102.7
Yorktown Gravity Filtration w/ Detention	860	878	92	10	0	32 (12"HDPE, 1,000gpm, 6,000ft)	134
Yorktown Pressure Filtration w/ Detention	860	856 (lower than well house site)	70	10	0 (Secondary Pumping)	32 (12"HDPE, 1,000gpm, 6,000ft)	112
Median Site Pressure Filtration w/o Detention	876	886	100	10	100	0	341
Fred Richards Gravity Filtration w/ Detention	828	846	60	10	0	57 (12"HDPE, 1,000gpm, 3,000ft) & (20"HDPE, 3,000gpm, 7,200ft)	127
Fred Richards Pressure Filtration w/ Detention	828	824	38	10	0 (Secondary Pumping)	57 (12"HDPE, 1,000gpm, 3,000ft) & (20"HDPE, 3,000gpm, 7,200ft)	105

Table 9.3 Well No. 18 Hydraulic Analysis

WTP Site	Site Elevation (ft.)	WTP Inlet Elevation (ft.)	Static Head (ft.)	Well Pipe and Pump Losses (ft.)	Filtration Pressure (psi)	Pipeline Losses (ft.)	TDH Requirements
Southdale Gravity Filtration w/ Detention	880	898	139	10	0	32 (12"HDPE, 1,000gpm, 6,000ft)	181
Southdale Pressure Filtration w/ Detention	880	876	117	10	0 (Secondary Pumping)	32 (12"HDPE, 1,000gpm, 6,000ft)	159
Yorktown Gravity Filtration w/ Detention	860	878	119	10	0	0.34 (20"DI, 3,000gpm, 250ft)	129.34
Yorktown Pressure Filtration w/ Detention	860	856 (lower than well house site)	97	10	0 (Secondary Pumping)	0.34 (20"DI, 3,000gpm, 250ft)	107.34
Median Site Pressure Filtration w/o Detention	876	886	127	10	100	32 (12"HDPE, 1,000gpm, 6,000ft)	400
Fred Richards Gravity Filtration w/ Detention	828	846	87	10	0	57 (12"HDPE, 1,000gpm, 3,000ft) & (20"HDPE, 3,000gpm, 7,200ft)	154
Fred Richards Pressure Filtration w/ Detention	828	824	65	10	0 (Secondary Pumping)	57 (12"HDPE, 1,000gpm, 3,000ft) & (20"HDPE, 3,000gpm, 7,200ft)	132

Based on preliminary calculations, the existing wells, motors, and pumps should have adequate brake horsepower (BHP) to pump to most of the proposed WTP sites except for the median option. It is important to note that each well may experience a decrease in the observed efficiency and flow due to the change in pumping conditions. Table 9.4 examines the current motors installed in each well to verify their operating characteristics under the new pumping conditions should the hydraulic grade line increase or decrease. Typically, calculations for all motors on VFD's include a 1.15 service factor. The 1.15 service factor provides a 15% factor of safety on the motor size, which helps to protect possible overload of the motor.

Table 9.4 Well Motor Capacity Analysis

Well	Current Motor Hp	Current Design Point BHP	Current Pump NOL Pwr.	New Design Point BHP	Recommended Motor Hp *(With 1.15 Service Factor)*
Well No. 5	100	100.4	100		
Southdale Gravity				40.4	50
Southdale Pressure				33.2	40
Yorktown Gravity				43.4	50
Yorktown Pressure				36.3	50
Median Site Pressure				110.4	125*
Fred Richards Gravity				41.1	50
Fred Richards Pressure				34.0	40
Well No. 18	125	103.6	125		
Southdale Gravity				58.6	75
Southdale Pressure				51.5	60
Yorktown Gravity				41.9	50
Yorktown Pressure				34.8	40
Median Site Pressure				129.5	150*
Fred Richards Gravity				49.9	60
Fred Richards Pressure				42.7	50

* Recommended replacement prior to new WTP operation

Based on the results shown above, the motor sizes would decrease for all alternatives except the median site. Because the pumping head requirements will significantly decrease, there is a high risk of the pump operating too far to the right of the curve, causing impeller cavitation. The City may replace, for each well, the pump, motor, VFD, and electrical wiring with a more appropriate size to prevent this from happening. The reduced horsepower savings equates to approximately $20,000/year assuming an average horsepower reduction of 50Hp and 24 hours operation. Approximate budgetary costs to complete these well house modifications would be $100,000. This would include pump impeller replacement, motor replacement, VFD replacement, and new electrical wiring.

9.1.1 Conversion of Well No. 5 to Submersible Pump

The City's current capital improvements plan (CIP) indicates that the planned rehabilitation of Well No. 5 includes conversion of the existing vertical turbine pump to a pitless, submersible pump. The City communicated that they aim to maintain the well's current location, but make it less visible, seeing as the wells location is within a highly commercial are of Edina. The visibility of the new pump would be limited to the well cap and cover, which appropriately placed landscape architecture features may hide. Previous experience of the Project Team indicates that this conversion is feasible for the Southdale and Median Sites due to the relatively close proximity of the well to the proposed facility. For the Yorktown and Fred Richards sites, conversion of the pump will requires housing VFD above ground next to the pitless unit, negating the goal of the pump conversion.

The Southdale Site options require a smaller pump due to the reduced head conditions entering the proposed facility. With the site approximately 600-feet away from the well location, a sine wave filter is recommended for elimination of harmonics that could otherwise cause problems with the submersible pump. The costs associated with a submersible pump, sine wave filter, pitless unit, well house demolition, and site restoration increase the previously assumed $100,000 Well No. 5 rehabilitation estimate by $100,000 for the Southdale Site.

The Median Site requires a larger pump because the proposed facility uses system pressure and head conditions. This site is located directly adjacent the existing Well No. 5 location, requiring only a DV/DT filter for proper operation of the submersible pump with a VFD. The costs associated with this larger submersible pump, the pitless unit, well house demolition, and site restoration increase the previously assumed $100,000 Well No. rehabilitation estimate by $100,000 for the Median Site.

9.2 Onsite Storage Feasibility

The investigation of additional water storage on each of the WTP sites is an established goal of this PER. An analysis of storage needs conducted in Section 3.4 indicated an adequate amount of storage currently exists in the distribution system, assuming that the Dublin reservoir pump capacity meets the maximum hour demands (MHD) typically seen by Edina. If the Dublin Reservoir is not able to meet these pumping requirements, the City should consider making pumping improvements to the associated pump station or construct additional elevated storage.

Similar to the evaluation criteria of the Dublin reservoir, ground storage at the WTP is only a benefit to the City of Edina if sizing of the high service pumps inside of the WTP meets the maximum hour demands.

Typical MHD are often 50-percent more than the maximum day demands seen by a distribution system. Utilizing this logic, the capacity of the WTP is 3,000 gpm, so a MHD high service pump would need to be capable of producing 4,500 gpm and pumping into a 100 psi system. This requires a 450Hp vertical turbine pump, which providing firm MHD capacity requires a second pump of this size. The associated costs for just this pump and motor are approximately $200,000. In addition to this, to power this pump, the electrical system requires an enhanced electrical system. The Project Team recommends budgeting approximately $600,000 for accomplishing max hour demand production at the WTP.

The proposed WTP layouts previously discussed all include a clearwell of approximately 150,000 gallons. This provides approximately 100 minutes of MHD pumping, assuming the WTP is operating at maximum capacity. As noted in Section 3.4, this contributes to the equalization storage of the system for this period.

If the City requests additional ground storage volume, only the Fred Richards site would accommodate ground storage. A significant barrier to ground storage is available space and offset requirements to sanitary and storm sewers. This requirement also applies to other below grade water tanks required for the WTP. Section 9.7 discusses further analysis of potential utility relocation needs.

9.3 Operation of Water Towers and Distribution System

At the request of the Project Team, the City's water distribution system consultant completed an analysis on the impacts of the proposed WTP No. 5 at the various sites and under multiple scenarios. The analysis determined the impacts of the facility during average day and peak demands and identified concerns related to existing infrastructure size and operation. The following sections briefly describe scenarios analyzes and the major takeaways for each related to the operation of water towers and the distribution system for the various scenarios. Section 9.4, Section 9.5, and Section 9.6 detail the distribution system improvements necessary for each site determined by this analysis. ***Appendix O*** provides a copy of the water distribution system analysis report.

9.3.1 Evaluation Assumptions

Overall, the analysis included nine (9) different scenarios. For the Southdale, Median, and Yorktown scenarios, the analysis assumed addition of 3,000 gpm at the analyzed entry point into the distribution system. The Fred Richards scenarios assumed 5,000 gpm of total plant capacity, which includes 3,000 gpm from WTP No. 5 and 2,000 gpm from existing WTP No. 3.

Analysis included an Extended Period Simulation (EPS) for water tower operation comparison over a three consecutive day run with average July water demand. This analysis assumed continuous operation of WTP No. 5, other treatment plants operating based on water tower levels, and initial tower levels set to 10-feet below overflow.

In addition, the evaluation looked at a maximum day demand simulation to determine the impacts on the distribution system and identify infrastructure improvement needs.

9.3.2 Southdale Site

The Southdale Site scenarios indicate that tying the facility into the existing 12-inch pipe along France Ave is most desirable. This connection will help maintain water levels in the Southdale Tower. Tying into the 16-inch line that feeds the tower may result in increased water age over time because the plant will tend to operate based on demand rather than tower level.

In terms of distribution system impacts, the Southdale scenarios indicate that tying the facility into the existing 12-inch pipe along France Avenue increases pressures minimally by approximately 2 psi and pipe velocities exceed the 5 feet per second and less than 2 to 3 feet per 1000 feet head loss thresholds, but are manageable. Maximum day demands do not appear to increase system pressure.

9.3.3 Median Site

The Median Site analysis indicated that existing infrastructure along W 69th St. would not accommodate the 3,000 gpm plant capacity, requiring installation and connection of an upsized pipe into the 12-inch main along France Ave. The current 8-inch pipe limits maximum plant flow to approximately half of the design capacity. With these infrastructure improvements, the consultant anticipates the distribution system and water tower operation to be similar to the Southdale Site results.

9.3.4 Yorktown Site

Existing system pressures in this part of the City are over 90 psi, limiting the tolerance of this area for additional capacity. Without increasing the existing 10-inch water main along York Ave, average day demands increase pressures by approximately 17 psi. The most desirable scenario results occurred with the York Ave water main increased to 16-inch, but system pressure increases still resulted. Another alternative analyzed included extending a 16-inch water main from York Ave to France Ave. This direct connection may result in manageable pressure and head loss increases.

Overall, this site requires extensive infrastructure upsizing before eliminating any pressure related concerns in the distribution system. All scenarios analyzed resulted in balanced water tower operation across the system.

9.3.5 Fred Richards Site

The Fred Richards Site analysis indicates that WTP No. 5 may increase system pressures in the vicinity by 23 psi, elevating system pressures to over 135 psi. The analysis also resulted in unbalanced tower operation for the Southdale Tower. Infrastructure improvement alternatives indicated that even with extensive finished water main improvements, maximum plant capacities may be limited to 3,900 gpm, or 1,100 gpm less than the desired capacity for the site. In general, this site is the most undesirable in terms of required distribution system improvements and water tower operation.

9.4 Raw Water Transmission

Existing Wells No. 5 and No. 18 will require appropriately sized raw water transmission lines for connection in to WTP No. 5. In anticipation for WTP No. 5, Edina staff took steps to plan for a portion of the necessary raw water piping and installed a 3,000 foot 12-inch HDPE Raw water line from Well No. 18 to the Southdale Site. This pipeline will serve to transport the raw water from Well No. 18 to the Southdale site or Well No. 5 to the Yorktown site. Figure 9.1, Figure 9.2, and Figure 9.3 illustrate this pipeline in yellow.

9.4.1 Southdale Site Raw Water Transmission

Raw water supply to the Southdale site is a relatively straight forward installation. As noted above and illustrated in Figure 9.1 below, an existing 12-inch HDPE raw water line (yellow) already exists for the purposes of transporting raw water from Well No. 18 and No. 5 to the Southdale site. A single 12-inch HDPE pipe for 2,000 gpm may produce pipe velocities above the recommended values. The Project Team assumed installation by directional drilling of an additional 12-inch HDPE pipeline (Red Line) 500-feet under W 69th St to the west side of the proposed WTP site for Well No. 5. Anticipated costs for this pipeline are $145,000.

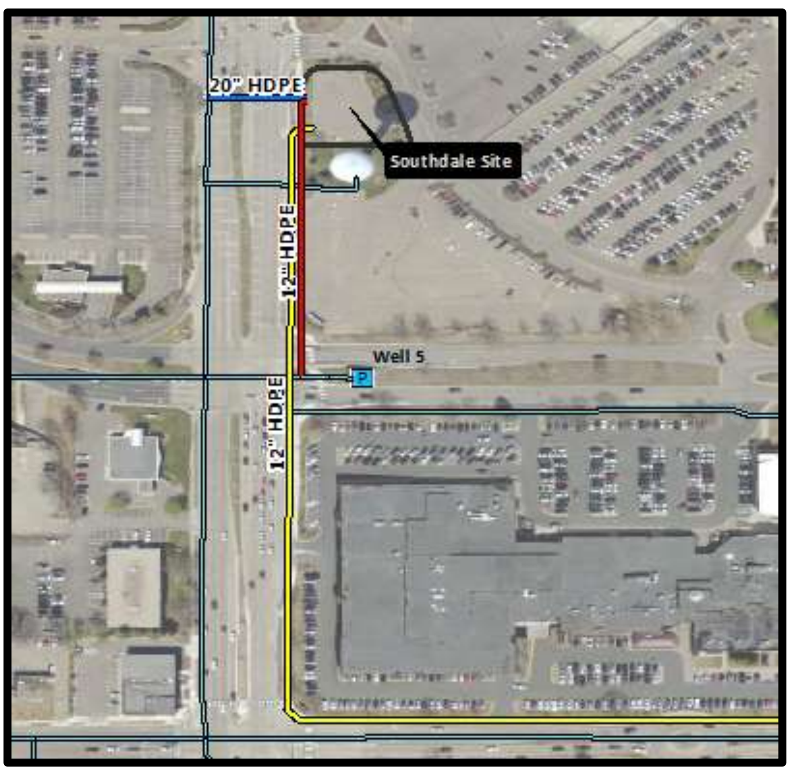

Figure 9.1 Southdale Site Raw Water Transmission

9.4.2 Yorktown Site Raw Water Transmission

Raw water supply to the Yorktown site will also use the previously installed 12-inch raw water line (yellow). This raw water line currently connects to the discharge piping from Well No. 18. Utilizing it to transport water from Well No. 5 to the Yorktown site will require a 250-foot, 20-inch ductile iron pipe installed from the existing piping to the WTP located on the Yorktown site. Figure 9.2 illustrates the location of this pipe as the red line. Anticipated costs for this pipeline are $65,000.

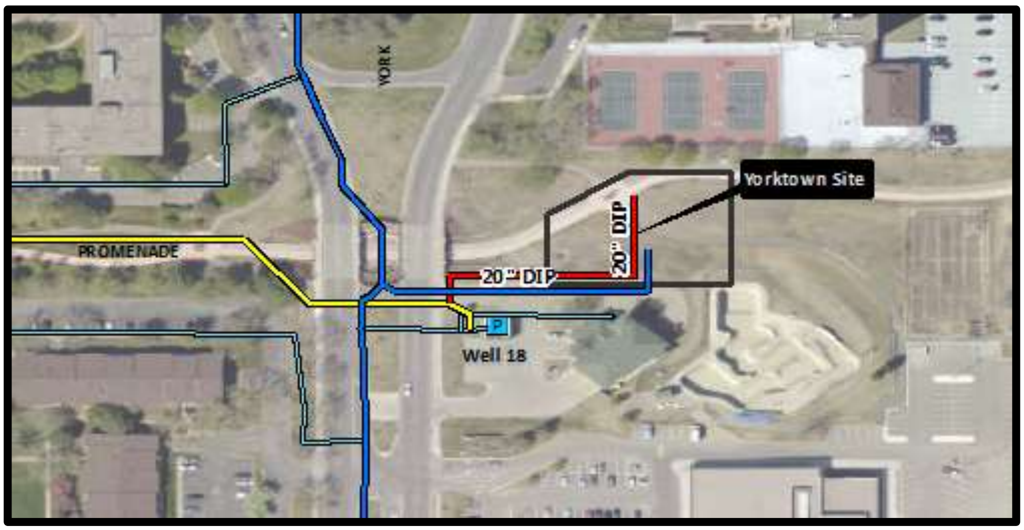

Figure 9.2 Yorktown Site Raw Water Transmission

9.4.3 Median Site Raw Water Transmission

Raw water supply to the Median site will also use the previously installed 12-inch raw water line. Raw water supply to the Median site requires limited piping modifications. Well No. 5 already exists on the site so the only piping required is a connection to the existing 12-inch HDPE pipe. Anticipated costs for this pipeline are $35,000.

9.4.1 Fred Richards Site Raw Water Transmission

Raw water supply to the Fred Richards site requires significant piping installations. To minimize the installation of new piping, the Project Team recommends utilizing the existing 12-inch HDPE (yellow) pipeline for transporting water from both Well No. 5 and Well No. 18. This requires a connection to the pipe made at Hazelton Rd. and a new 20-inch HDPE pipe (Red) installed west along Hazelton Rd, south on France Ave, west on West 72nd St, south on Oaklawn Ave, west on Gilford Ave, and south on Kellogg Ave, eventually reaching the proposed WTP

No. 5 site on the south side of the Fred Richards Golf Course. The new 20-inch HDPE pipeline length is approximately 7,200-feet long. Anticipated costs for this pipeline are $2,300,000.

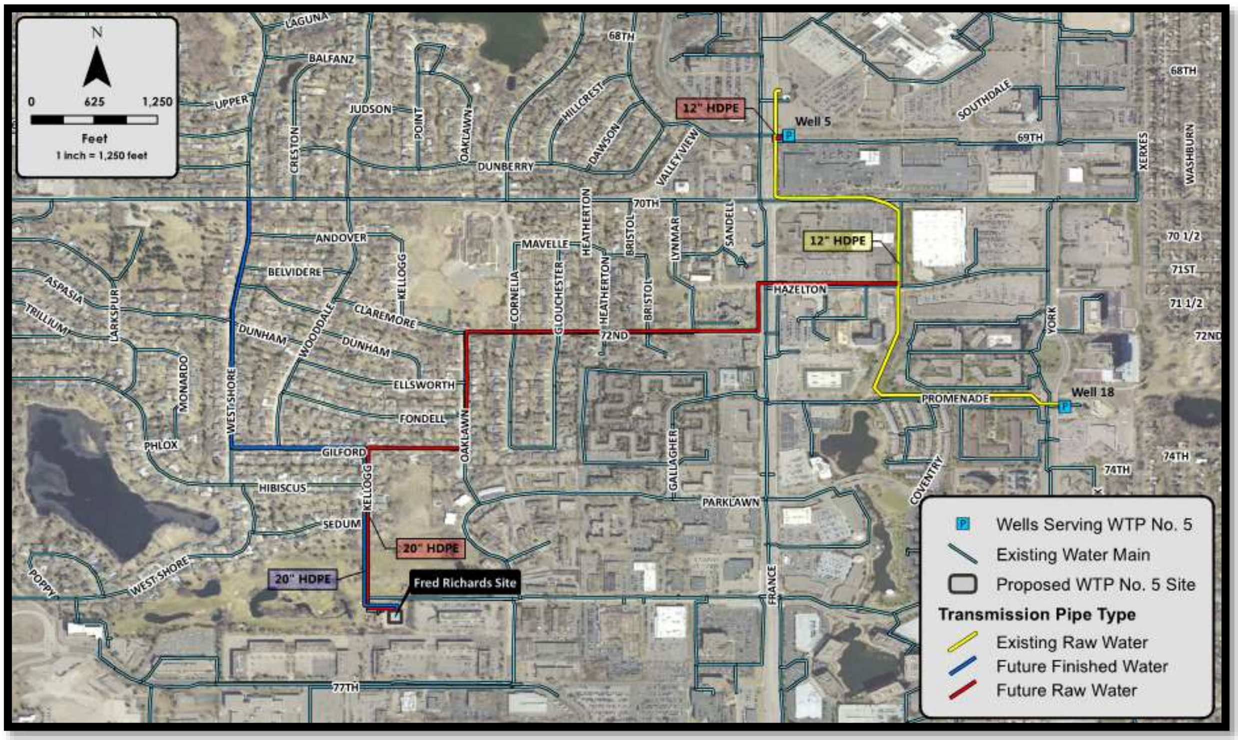

Figure 9.3 Fred Richards Site Raw Water Transmission

9.5 Finished Water Transmission

Once proposed WTP No. 5 treats the water it enters into the distribution system by either the well pumps through a pressure filter system of by the high service pumps located within the WTP. Each site will also pump the finished water into a different portion of the distribution system. The impacts to the distribution system will vary depending on the location of the system connection.

9.5.1 Southdale Site Finished Water Transmission

Construction of the finished water transmission pipeline from the Southdale site is relatively straight forward. As illustrated in Figure 9.4 below, a new 20-inch HDPE pipe will be directionally drilled 150-feet under France Ave from the Southdale site to the 12-inch ductile iron distribution system pipe located on the western side of France Ave. Anticipated costs for this pipeline are $55,000.

Figure 9.4 Southdale Site Finished Water Transmission

9.5.2 Yorktown Site Finished Water Transmission

Connecting the finished water pipeline from the Yorktown site to the distribution system is a relatively straight forward installation. As illustrated in Figure 9.5 below, a new 20-inch HDPE pipe will be directionally drilled 250-feet under York Ave S from the Yorktown site to the 10-inch ductile iron distribution system pipe located in the center of York Ave S. Anticipated costs for this pipeline are $90,000.

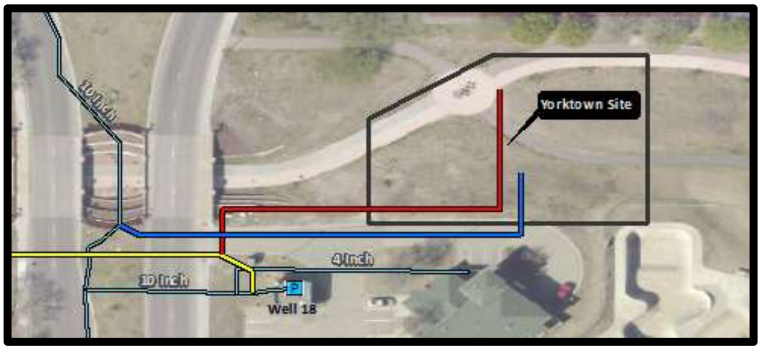

Figure 9.5 Yorktown Site Finished Water Transmission

Edina's water distribution system consultant conducted an Extended Period Simulation (EPS) of Edina's water distribution system to determine the impact of a 3,000 gpm flow from the proposed WTP No. 5. Results from this analysis indicated a large pressure spike along York Ave if the City selects the Yorktown site and constructs the finished water connection. In order to mitigate these pressure spikes, York Ave requires a utility improvement. This improvement consists of replacing the existing 10-inch ductile iron water main with a new 16-inch ductile iron main and reconnecting the existing distribution system piping along this corridor. Anticipated costs for these improvements would be $1,500,000 assuming open cut pipe installation and the project completed in conjunction with other utility improvements along York Ave. Figure 9.6 illustrates the alignment of these proposed improvements.

Figure 9.6 Yorktown Site Distribution System Improvements

9.5.3 Median Site Finished Water Transmission

As illustrated in Figure 9.7, connecting the finished water pipeline from the Median Site to the distribution system would require a 20-inch HDPE main to be directionally drilled 400-feet from the WTP site to the 12-inch ductile iron pipe located on France Avenue. The location of the connection may vary depending on potential utility conflicts within France Ave. It is unlikely that the connection occurs directly west of the median site because of the complicated nature of the intersection at that location. Anticipated costs for this pipeline are $135,000.

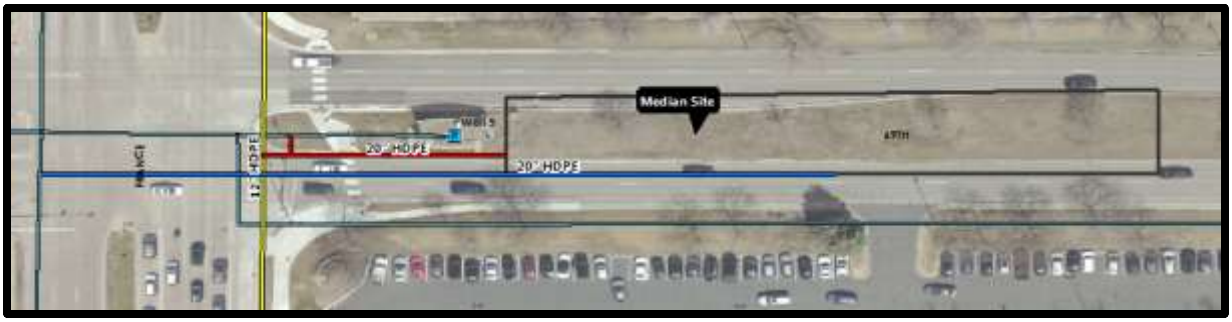

Figure 9.7 Median Site Finished Water Transmission

9.5.4 Fred Richards Site Finished Water Transmission

Raw water supply from the Fred Richards site to the distribution system would require a significant pipeline in order to limit pressure spikes in the distribution system. Connecting the finished water pipeline from the Fred Richards site requires a 20-inch HDPE pipe directionally drilled north along Kellogg Ave, west along Gilford Ave, then north along West Shore Dr., eventually connecting into the 12-inch ductile iron main along West 70th St. This pipeline is approximately 5,300-feet in length. Figure 9.8 illustrates the proposed alignment as the blue line. Anticipated costs for this pipeline are $1,700,000.

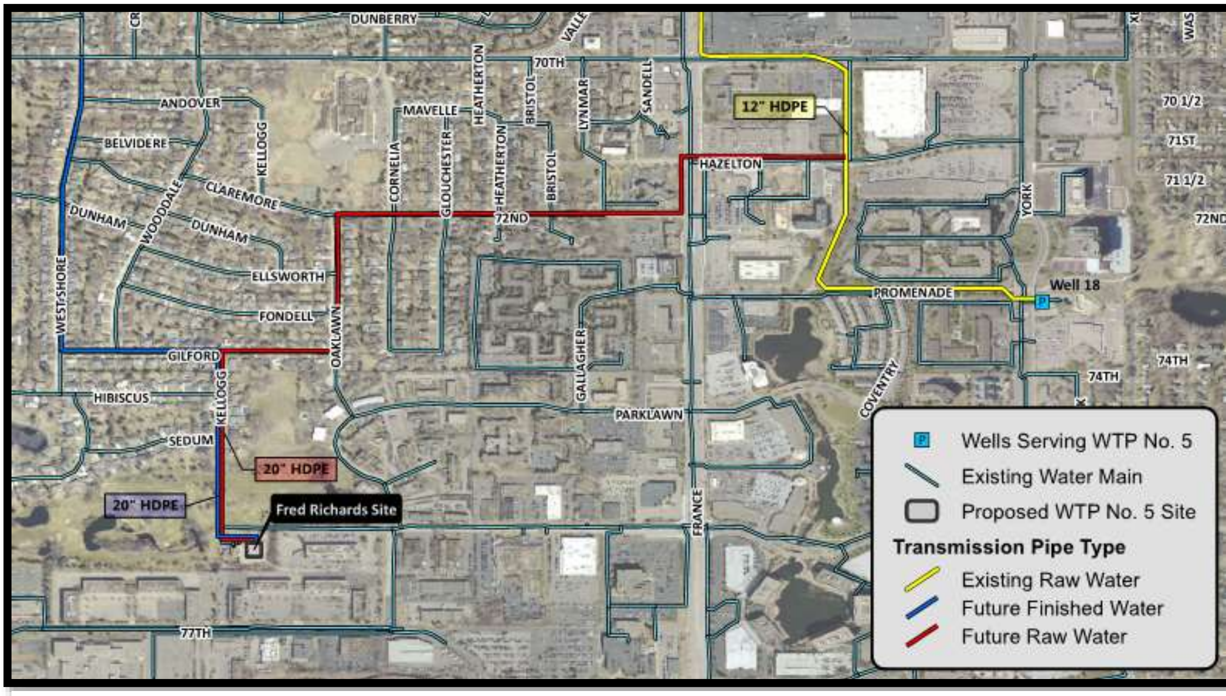

Figure 9.8 Fred Richards Site Finished Water Transmission

9.6 Required Utility Relocations / Improvements

9.6.1 Yorktown Site Utility Modifications

In addition to the distribution system improvements, the Yorktown site poses significant challenges related to sanitary sewer and storm sewer pipelines that run through the proposed site. The Minnesota Department of Health requires a minimum of 50-feet of separation between any below grade water tanks and any storm or sanitary sewers. If 50-feet of separation is not achievable, construction of the sewer mains can be to water main grade standards, which will reduce the separation requirement to 15-feet. Currently, the proposed Yorktown site contains the following sewer mains.

1. 33-inch RCP Sanitary Sewer
2. 60-inch RCP Storm Sewer
3. 16-inch Storm Sewer Force Main
4. Sanitary Sewer Service

5. Site Storm Sewer Tie-Ins

In addition to these existing pipes, recent sanitary sewer feasibility studies conducted throughout the City indicate future requirement of a 12-inch sanitary forcemain or 33-inch RCP sanitary sewer running through the northern boundary of the site. Figure 9.9 illustrates these sewer mains.

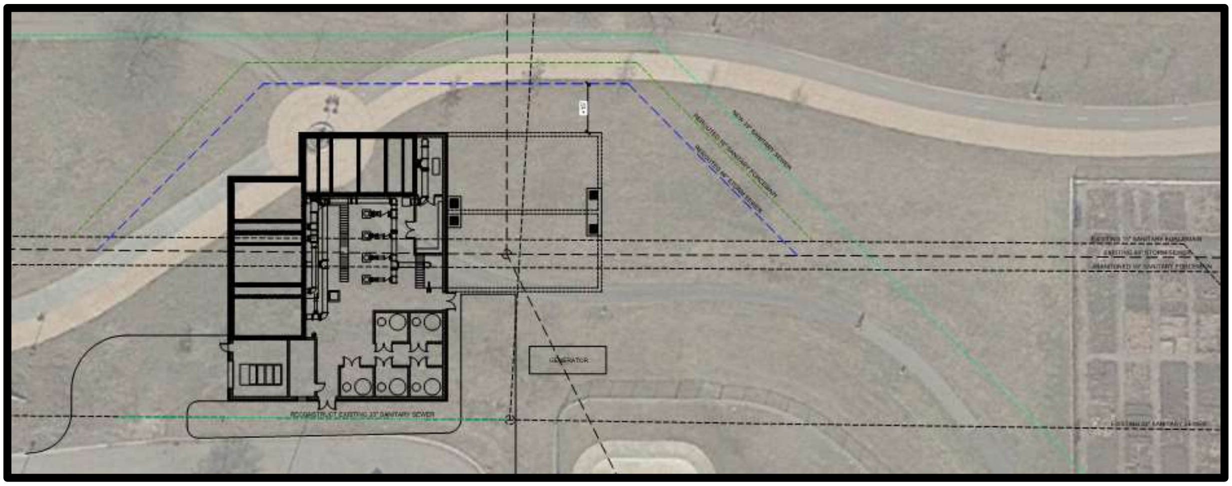

Figure 9.9 Yorktown Sewer Alignments and Proposed Realignments

In order to construct WTP No. 5 on the proposed Yorktown Site, these sewer mains will need to be relocated to maintain a minimum 15-foot separation from the below grade tanks in the WTP. This assumes that construction of the re-located utilities is to water main grade standards. Relocation of these mains is a significant endeavor and requires significant analysis to understand the capacity and flexibility currently available in the existing infrastructure. Re-routing the alignment of these sewer mains will ultimately require additional length added to them. In some instances, it may be possible to add this length and maintain the necessary capacity in the main. In other instances, the additional length may not be feasible. Review of as-built information indicated the flowing characteristics of the major pipelines requiring rerouting.

Table 9.5 Existing Yorktown Sewer Characteristics

System	Existing Grade	Proposed Length	Proposed Grade
33-inch Sanitary Sewer	0.1%	142	0.1%
60-inch RCP Storm Sewer	1.2%	307	1.0%
16-inch Storm Sewer Force Main	-	325	-

After review of these characteristics, relocation of the 60-inch storm sewer and 16-inch forcemain is achievable. This requires a utility easement constructed on the north side of the WTP site. Reconstruction can also include the proposed 33-inch sanitary sewer within this easement and ultimately accept the sanitary sewer service that currently connects to the existing 33-inch sewer on the south side of the site.

Review of the existing 33-inch sanitary sewer on the south side of the proposed site indicates an existing grade of 0.1%. This is extremely low and significantly limits any potential modifications to this main. Ultimately, adding additional length would significantly reduce its capacity. If the City ultimately chooses the Yorktown Site for WTP No. 5, assume that this sanitary sewer must remain in place at the current grade. It may be possible to replace the main in its current location with a water main grade pipe that will ultimately limit the required separation to 15-feet.

Anticipated costs to complete the storm and sanitary sewer improvements noted above is approximately $1,750,000.

9.6.2 Median Site Utility Modifications

In addition to the distribution system improvements, the Median Site poses challenges related to sanitary sewer and storm sewer pipelines that run through the proposed site. As noted previously, the MDH requires a minimum of 50-feet of separation between any below grade water tanks and any storm or sanitary sewers. If 50-feet of separation is not achievable, construction of the sewer mains can be to water main grade standards, which will reduce the separation requirement to 15-feet.

Sewer mains required for rerouting in order to construct WTP No. 5 on the proposed Median Site include a 27-inch arch pipe storm sewer main, with various lateral lines connecting to catch basins along W 69th Street and a 10-inch sanitary sewer pipe that collects from adjacent commercial and retail property. The utilities run directly below the street corridor, requiring significant surface disturbance and high restoration costs.

Relocation of these mains is a significant endeavor and requires analysis to understand the capacity and flexibility currently available in the existing infrastructure. Re-routing the alignment of these sewer mains may ultimately require additional length added to them. In some instances, it may be possible to add this length and maintain the necessary capacity in the main. In other instances, the additional length may not be feasible. Anticipated costs to complete the storm and sanitary sewer improvements noted above is approximately $1,000,000.

9.7 Southdale Site Structural Integrity

The Project Team evaluated the feasibility of constructing a more robust foundation for the standalone Southdale Site, providing the structural integrity necessary for constructing additional commercial or retail space above proposed WTP No. 5 in the future.

Vertical construction atop the new Edina WTP No. 5 at the Southdale Site is feasible and recommended for further consideration and study. This initial review assumed that future construction includes two floor levels added atop the WTP with plan area equal to the WTP footprint, approximately 9000 square feet per floor.

The Project Team assumed the soils at the proposed WTP site are consistent with those identified in a geotechnical investigation for the adjacent Restoration Hardware site. Although quality soils are present, the additional foundation loading from the proposed vertical construction may require subgrade improvement to safely support the load and limit facility settlement.

The primary framing system of the WTP already utilizes cast in place concrete beams, columns, walls and mat foundations. These framing members likely require limited modification to layout, size, and reinforcing to accommodate the proposed vertical construction. The estimated additional construction costs for two floors of vertical construction atop the WTP includes subgrade improvements, foundation and framing modification, and roof adjustments for repurposing the roof as a floor in the future. Anticipated costs to provide a more robust foundation for future construction of up to two additional floors above WTP No. 5 is approximately $500,000 to $700,000. This assumes commercial, retail or office space development above the facility. Other uses, such as parking space or heavy industrial space, or additional floors may trigger additional costs associated with the WTP foundation.

CHAPTER 10 SITE ACCOMMODATIONS EVALUATION

10.1 Introduction

This chapter evaluates each site's ability to accommodate various non-financial criterion related to treatment performance, security and safety, site architecture, constructability, and additional infrastructure considerations. The City, jointly with AE2S and Oertel Architects, provided opinions of favorability to identify the non-financial benefits and tradeoffs of each site. Table 10.1 summarizes the evaluation symbols and descriptions used throughout this chapter.

Table 10.1 Site Accommodations Evaluation Descriptions and Symbols

Evaluation Description	Very Unfavorable	Unfavorable	Neutral	Favorable	Very Favorable
Evaluation Symbol	OO	O	_	X	XX

The Southdale Site evaluation includes two options. These options include one with Simon Properties integrating the water treatment facility into a shared-use building and one with the facility built as standalone public infrastructure. The Project Team and Edina staff had multiple conversations with Simon Properties throughout the preliminary design report process. The conversations led to the decision that Simon Properties has no interest in moving forward with development on the Southdale Site at this time.

10.2 Treatment Performance

10.2.1 Evaluation Criteria

The following evaluation criteria evaluate each sites ability to meet performance objectives, limit operational complexity, and offer future operational flexibility.

10.2.1.1 *Performance Objectives*

This criteria reviews whether the site accommodates all performance goals including, but not limited to, MDH standards, standard industry practices, primary drinking water regulations, and facility treatment goals related to iron, manganese, and radium removal and a consistent disinfection strategy. Consider sites with unfavorable evaluations of this criterion much less favorable than other sites.

10.2.1.2 *Operational Complexity*

Operational complexity relates to the overall range of treatment technologies present within the proposed alternative. This relates to operator familiarity with the proposed technology and ease of operation of the facility compared to existing facilities in the City's system.

10.2.1.3 *Operational Flexibility*

Operational flexibility evaluates whether the site can feasibly modify treatment processes or expand treatment capacities in the future. These expansions relate to treated water capacities, addition of new treatment equipment, and adjustment of chemical feed systems with changes in raw water quality.

10.2.2 Performance Objectives

The gravity and pressure filter options for the Southdale Site, Yorktown Site, and Fred Richards Site all accommodate the required treatment technologies for meeting all MDH standards, primary drinking water regulations, and established treatment goals. This includes chlorine for pre-oxidation and disinfection, permanganate for pre-oxidation and radium removal, HMO for enhanced radium adsorption, a detention basin for 30 minutes of reaction time, filters for iron, manganese, and radium removal, ammonia for supplemental chloramine formation, fluoride for dental hygiene, and an ortho/poly blend for corrosion inhibition. Gravity sites also include a clearwell for storing finished water prior to pumping into the system with high service pumps.

Another item related to meeting the performance objectives is redundancy of the facility systems. Without redundancy, accomplishing treatment goals during maintenance and emergency situations is difficult. The proposed chemical feed systems and pumps incorporate redundancy for every site alternative. Critical chemical feed systems include redundant pumps that provide continuous chemical feed in the event of pump failure or required maintenance. High service pumps are sized to pump the full 3,000 gpm plant capacity with only two (2) of the three (3) pumps online, allowing cycling of online pumps and a back-up during maintenance or emergencies. All filter influent, effluent, and backwash piping is separate, allowing occurrence of backwashes simultaneously with the other filters remaining online. The plant maintains a 2,000 gpm firm capacity during filter backwash.

The pressure filter option proposed for the Median Site does not include a detention basin within the design. The pilot study confirmed that minimal detention time still provides manganese removal below the SMCL of 0.05 mg/L, but permanganate dose optimization is critical and the absence of the extended detention time eliminates the buffer against changes in raw water quality. Another benefit of extended detention resulting during piloting was enhanced radionuclide removal. Table 10.2 summarizes the evaluation of each site related to meeting performance objectives and treatment target goals.

Table 10.2 Performance Objectives Evaluation for All Site Alternatives

Criteria	Standalone Southdale	Integrated Southdale	Yorktown	Median	Fred Richards
Performance Objectives	XX	XX	XX	O	XX

10.2.3 Operational Complexity

The extent of new chemical feed technologies present throughout all the proposed facilities includes permanganate and ammonia chemical feed systems. Selected alternatives for these two systems are both liquid chemical feed systems where a chemical distributor delivers bulk solution to the facility through a bulk connection located outside the building. Plant operator responsibility will be limited to general maintenance of the system and optimization of chemical feed rates. The facility will include instrumentation and controls for automatic adjustment of chemical dose using flow-paced chemical feed and residual concentration monitoring.

In addition to the new chemical feed systems, the gravity filters are a new technology for the City of Edina, but some staff have operated gravity filters in the past. Gravity filters allow visual inspection of the media bed during operation and backwash. No other existing facilities have a detention tank, but O&M of the tank is minimal and similar to traditional backwash reclaim tanks. Detention tanks will incorporate a sludge blowdown and cleaning system, so their presence in the facilities does not increase operational complexity.

Overall, the treatment technologies proposed for WTP No. 5, regardless of site, are not complex. The Median Site is slightly less complex with only pressure filters being feasible and no available footprint for a detention tank. Table 10.3 summarizes the evaluation of each site related to operational complexity.

Table 10.3 Operational Complexity Evaluation for All Site Alternatives

Criteria	Standalone Southdale	Integrated Southdale	Yorktown	Median	Fred Richards
Operational Complexity	—	—	—	X	—

10.2.4 Operational Flexibility

Sizing of chemical feed systems for all site alternatives accommodates operation at a wide range of feed rates with changes in raw water quality. The only exception is the Median Site that includes space for only 150 lb cylinders of chlorine. Based on the raw water quality of future Well No. 21, 150 lb cylinders may limit the available chlorine dose while operating at full plant capacity. These limitations will vary based on raw water contaminant chlorine demand and the City's disinfection strategy at the time. As a general rule of thumb, maximum draw from a single 150 lb chlorine cylinder is 75 pounds per day otherwise cylinders may freeze up. The City has the option to manifold multiple 150 lb cylinders, which provides more chlorine online at a time. Selecting 150 lb cylinders will inevitably increase delivery frequency of chlorine compared to storing ton cylinders.

Online instrumentation will monitor residual concentrations and adjust chemical feed as required. Wells will operate on a variable frequency drive (VFD) for adjustment of plant production based on system demand.

Site size and shape limit the extents of future treatment technology integration for the Southdale and Median sites. For example, if the contamination plume currently treated by WTP No. 6 moves further south over time, the limited site size restricts the potential for adding air strippers or other VOC removal technology to the facility. The Southdale Site integrated into a shared-use facility has less room for expandability because development in direct connection with the site eliminates the space available for future treatment expansion.

Yorktown and Fred Richards have less restriction and accommodate future treatment expansion. With extensive upgrades to the distribution system, Fred Richards has the potential to replace WTP No. 3 with a combined 5,000 gpm facility, accepting raw water from Well No. 5, No. 10, No. 11, No. 18 and future Well No. 21. The City recently developed a Master Plan for redeveloping the Fred Richards golf course into a multi-use park, restaurant, event, and athletic complex. As plans for redevelopment continue at this site, the likelihood of constructing a new water treatment facility at the site reduces.

Table 10.4 summarizes the evaluation of each site related to treatment expandability and flexibility.

Table 10.4 Operational Flexibility Evaluation for All Site Alternatives

Criteria	Standalone Southdale	Integrated Southdale	Yorktown	Median	Fred Richards
Operational Flexibility	—	O	XX	OO	X

10.3 Security and Safety

10.3.1 Evaluation Criteria

The following sections evaluate the safety and security of each proposed facility for plant operators and the public. Factors affecting this criterion include vicinity of hazardous chemicals to public areas or inconvenient site access for operators and chemical deliveries.

10.3.2 Operator Security and Safety

No proposed site alternative includes components that create an unsafe environment for operators. With proper risk management plans established and chemical handling procedures followed, operator safety is not a concern for any of the proposed chemicals or equipment. In the event of a chemical leak or spill, all facilities will incorporate leak detection and emergency shutoff systems for gaseous chemicals and within pump heads.

The largest differentiator related to operator safety between the site alternatives is site access. The Southdale, Yorktown, and the Fred Richards sites have adequate parking and space onsite for operator and chemical delivery truck access. The Southdale Site integrated into a shared-use facility may have less space available for operator and chemical delivery access. The Median Site has very limited space and requires a dual gated access drive within an existing lane of West 69th Street. This access limitation makes the Median Site the least favorable in terms of operator security and safety.

Table 10.5 summarizes the evaluation of each site related to operator security and safety.

Table 10.5 Operator Security and Safety Evaluation for All Site Alternatives

Criteria	Standalone Southdale	Integrated Southdale	Yorktown	Median	Fred Richards
Operator Security and Safety	XX	X	XX	OO	XX

10.3.3 Public Security and Safety

Again, none of the proposed site alternatives incorporates equipment or technology that creates an unsafe environment for the public. This evaluation criterion relates to the public's perception of the safety of the site. For example, the Southdale Site is located within a highly commercial and populated area, making chemical deliveries of hazardous material a potential concern for those in the vicinity. For this reason, high profile sites like the Southdale Site and Median Site incorporate architectural and site security features such as fencing or landscaping that hide the visibility of delivery vehicles. An integrated Southdale Site will increase pedestrian traffic near the facility and chemical delivery areas, which is less desirable in terms of public perception of security and safety.

Similar to the impacts the site accessibility of the Median Site has on operators, the site also influences public safety. The vicinity of the site to the France Avenue and 69th Street intersection makes the site unfavorable. Conversations with MDH staff related to the Median Site indicate concerns about increased potential of vehicular accidents surrounding the building from limited site distances and the potential for vandalism activities with the facility in close proximity to the road. During chemical deliveries or site visits, the gated access way will be periodically open, creating a traffic distraction for the public. Overall, the Fred Richards site is most favorable in terms of operator and public security and safety because its location is not in such a high profile area.

Table 10.6 summarizes the evaluation of each site related to public security and safety.

Table 10.6 Public Security and Safety Evaluation for All Site Alternatives

Criteria	Standalone Southdale	Integrated Southdale	Yorktown	Median	Fred Richards
Public Security and Safety	—	O	X	OO	X

10.4 Site Architecture

10.4.1 Evaluation Criteria

The following evaluation criteria evaluate each sites impact on architectural value, land use consistency with current plans, ability to provide economic and environmental sustainability, and the feasibility of a shared-use facility.

10.4.1.1 *Architectural Value*

Evaluation of the site architectural value aims to identify the potential impacts and benefits development will have on a site, community, and city level. The evaluation criteria includes effective land use, accessibility, site adjacency, and visibility, impact on future development, and community perception and vision.

10.4.1.2 *Sustainability / Resiliency*

Economic and environmental sustainability / resiliency of the proposed facility is an important factor in the selection of a preferred site alternative. This criterion identifies the feasibility of sustainable features for each site including, but not limited to, energy-efficient technology, wastewater reuse, on-site renewable energy, storm water reuse and infiltration, and the potential for LEED certification.

10.4.1.3 *Shared-Use Feasibility*

As identified briefly in Chapter 8, most of the site alternatives offer unique characteristics related to a shared-use facility. Examples of shared-use space range from public works storage space, public amenities such as a regional trail head, and public/private development opportunities such as restaurant and retail space as part of a larger commercial development.

10.4.1.4 *Land Use*

This criterion evaluates whether the proposed site proposes a land use consistent with the City's comprehensive plan or Southdale area plan. This aims to determine whether the water treatment facility fits into the current development plans for Edina.

10.4.2 Architectural Value

The proposed Southdale Site is in a high value area within the city. However, the size of the site and access is limited. The site currently has fair vehicular access and has the ability for use as a pedestrian and bicycle hub per the Greater Southdale Area Planning Framework. The existing and proposed building stock surrounding the site is mid to large scale commercial-type construction. Development of the site will be highly visible, thus necessitating a higher level of fit, finish, and detailing in the overall design.

The Yorktown Site is located adjacent Fire Station No. 2, Southdale YMCA, and Adams Hill Park. The site prominently features a walk/bike trail that is part of the city's public art program. The site has excellent vehicular, pedestrian, and bicycle access. The surrounding building stock is mid-scale commercial type construction and multi-family residential. Development on the site would be low visibility and could integrate into the other existing city infrastructure nearby.

The Median site is located at the center of West 69th Street. The site has poor vehicular access with little or no area for parking, loading, deliveries, etc. The existing and proposed building stock surrounding the site is mid to large scale commercial-type construction. As development within the median is rare, the site poses a challenge as it relates to architectural value. In general, there are a number of limiting factors for construction and use. Development on the site raises some visual concerns as the building might limit or cut-off view corridors and connections along France Ave.

The proposed Fred Richards Site is located at the former site of Fred Richards Golf Course. The site has good vehicular access. The existing and proposed building stock surrounding the site is mid-sized commercial-type construction and single-family residential. The visibility of the site is dependent on the type and scope of future development.

Table 10.7 summarizes the evaluation of each site related to architectural value.

Table 10.7 Architectural Value Evaluation for All Site Alternatives

Criteria	Standalone Southdale	Integrated Southdale	Yorktown	Median	Fred Richards
Architectural Value	X	XX	_	O	X

10.4.3 Sustainability / Resiliency

There are no known limiting factors preventing the integration of sustainable building features at the Southdale, Yorktown, or Fred Richards's sites. Given the size and orientation of the Southdale Site, there are some limitations on the feasible types of site sustainability features and the building layout will have a limited south exposure, negating some solar options. The water tower offers some unique opportunities that the City may explore for viability. There are a number of utilities on-site, which presents a unique opportunity to explore water treatment technologies either in practice or as a demonstration system.

The extreme geometry of the Median Site will influence potential sustainability options within the building and eliminate many site options. The site has good south exposure for passive or active solar.

Appendix P provides sustainability options with potential for inclusion in the design of WTP No. 5 moving forward. This includes detailed information related to the three (3) sustainability design tracts the City has the option of pursuing. These tracts range from baseline sustainability options, related to high efficiency lighting fixtures and building orientation, to sustainability certification through a third-party verification program such as LEED.

Table 10.8 summarizes the evaluation of each site related to sustainability / resiliency.

Table 10.8 Sustainability/Resiliency Evaluation for All Site Alternatives

Criteria	Standalone Southdale	Integrated Southdale	Yorktown	Median	Fred Richards
Sustainability / Resiliency	—	—	X	O	X

10.4.4 Shared-Use Benefit

The shared-use benefit of the Southdale site is dependent on a public/private partnership through a developer. A standalone Southdale site has less likelihood for shared-use benefit than an integrated Southdale Site with adjacent development does. Development of a mixed-use project featuring retail, office, hospitality, etc. relies on an integrated parking structure within the building or on a shared agreement to utilize adjacent surface lot parking. ***Appendix Q*** provides preliminary architectural renderings for an integrated Southdale Site consistent with the shared-use options listed herein.

The shared-use benefit of the Yorktown Site is most congruent to additional city-based public elements. Possible programmatic elements include but are not limited to; park trail head building, public art installation and expansion, fire department utilization, park department utilization, integration with nearby public gardens, or joint-use with the YMCA. ***Appendix R*** depicts preliminary architectural renderings for a Yorktown Site concept consistent with the park trail head shared-use option listed previously.

The shared-use benefit of the Median Site is limited. The exiting pump station on-site is in need of rehabilitation, which may occur in conjunction with the treatment facility project.

Water Treatment Plant No. 3 is currently located on-site at the Fred Richards Site. The city could leverage future rehabilitation needs at the existing plant with a new plant. Overall, the City has a number of options to explore for mixed or shared use development on this site. Depending on the flexibility of the recently approved redevelopment Master Plan of Fred Richards Golf Course, this site may offer additional shared-use opportunities.

Table 10.9 summarizes the evaluation of each site related to sustainability / resiliency.

Table 10.9 Shared-Use Benefit Evaluation for All Site Alternatives

Criteria	Standalone Southdale	Integrated Southdale	Yorktown	Median	Fred Richards
Shared-Use Benefit	—	XX	—	OO	X

10.4.1 Land Use

The Future Land Use Plan, developed as part of the City's 2008 Comprehensive Plan Update, indicated that the Southdale and Median sites are within the Community Activity Center (CAC) land use category and the Yorktown and Fred Richards's sites are within the Open Space and Parks (OSP) category. The City's zoning map identifies the Southdale and Median sites as Planned Commercial Development (PCD) and the Yorktown and Fred Richards's sites as Single Dwelling Units (R-1).

Without a public/private partnership through a developer, which would allow integration of the Southdale Site into multi-use development, building standalone public infrastructure does not fit the planned land use or zoning categories for this site. The Median Site, while considered a CAC land use category, does not have substantial shared-use benefit, as indicated in the previous section. While this sight may not fit the current land use or zoning category, the likelihood of other developed within this site is unlikely.

Placement of the water treatment facility at the Yorktown Site changes the parcel to a public/semi-public land use category, but likely maintains the single dwelling unit zoning classification. As previously indicated, options for shared-use features at this site include the possibility of a trail head feature, incorporating components of an open space land use into the proposed facility.

The existing site of Fred Richards Golf Course recently underwent master planning for conversion into a multi-use recreational, retail, and event space. This requires rezoning of this area from the current zoning category. Depending on the redevelopment timeline of this site, incorporating the water treatment facility into this site may end up fitting the planned land use designation.

Table 10.10 summarizes the evaluation of each site related to sustainability / resiliency.

Table 10.10 Land Use Evaluation for All Site Alternatives

Criteria	Standalone Southdale	Integrated Southdale	Yorktown	Median	Fred Richards
Land Use	O	X	_	_	_

10.5 Constructability

10.5.1 Evaluation Criteria

The following evaluation criteria evaluate each sites initial construction considerations, construction staging and sequencing accommodations, and future maintenance accessibility.

10.5.1.1 *Initial Construction*

The initial construction evaluation provides a generalized review of site accessibility and possible constraints related to items such as noise, material and equipment delivery, and site security.

10.5.1.2 *Construction Staging / Sequencing*

Another criterion related to initial construction of the facility evaluates the sites ability to accommodate construction staging and sequencing. Sites with limited space may complicate construction and preliminary site selection must consider these limitations.

10.5.1.3 *Future Maintenance*

Similar to initial construction, future maintenance of the site is important for design, especially in a pressure filter facility. Without adequate space to access various system components, difficulty of maintenance on equipment increases.

10.5.2 Initial Construction

The Southdale and Median sites provide the greatest constraints for initial construction. The construction of a new Restoration Hardware is underway in the existing parking lot directly south of the Southdale Tower, eliminating stockpiling and equipment or material storage within this lot for both sites. The limited site constraints will also require extensive shoring and sheet piling for excavation of below grade tanks. The Median site may require closing down of 69th Street for periods, increasing traffic control requirements and coordination with private and public commercial businesses in the vicinity. An integrated Southdale Site may lend itself to larger construction extents and areas for stockpiling materials and equipment.

The Yorktown Site construction limitations relate to the shared entrance with Fire Station No. 2 and the extensive utility relocations required for this site. Construction cannot obstruct emergency vehicles from accessing their facility, so site access requires careful coordination.

This site requires a large excavation area for utility relocation, which may include sheet piling along the north and south site boundaries to access existing utilities and limit excavation.

Noise constraints are likely for all sites but Fred Richards due to the high density commercial and residential buildings near the other sites. Areas around the Southdale, Median, and Yorktown sites are highly commercial and residential, and with this comes pedestrian traffic. For this reason, site security measures during construction may need enhancement. Table 10.11 summarizes the evaluation of each site related to initial construction.

Table 10.11 Initial Construction Evaluation for All Site Alternatives

Criteria	Standalone Southdale	Integrated Southdale	Yorktown	Median	Fred Richards
Initial Construction	O	X	O	OO	XX

10.5.3 Construction Staging / Sequencing

Similar to the initial construction evaluation, sites with limited size, such as the standalone Southdale and Median sites are less favorable in terms of construction staging and sequencing. Without adequate space for stockpiling of excavated materials or equipment and material storage, construction staging and sequencing becomes increasingly more complicated. Construction staging and sequencing for Yorktown will also require additional coordination with the adjacent fire station and for the integrated Southdale Site with Simon Properties. Construction of the Fred Richards Site would likely take place prior to completion of redevelopment of the golf course into a multi-use park and event facility, so adequate construction staging and sequencing space is available.

Table 10.12 summarizes the evaluation of each site related to construction staging and sequencing.

Table 10.12 Construction Staging and Sequencing Evaluation for All Site Alternatives

Criteria	Standalone Southdale	Integrated Southdale	Yorktown	Median	Fred Richards
Construction Staging / Sequencing	OO	X	X	OO	XX

10.5.4 Future Maintenance

Again, sites with size limitations do not provide favorable conditions for future maintenance of the facility. This is especially true in pressure filter facilities, if entire pressure filter removal is necessary for replacement or repair. For the standalone Southdale and Median Sites, removal of a pressure filter requires lane closures along either 69th Street or the Southdale Mall frontage road. The Yorktown Site provides favorable access to the proposed facility for future maintenance. An integrated Southdale Site may lend itself to easier future maintenance due to inherently better cooperation with Simon Properties. If redevelopment around the Fred Richards Site proceeds, future maintenance access limitations may surface.

Table 10.13 summarizes the evaluation of each site related to future maintenance of the facility.

Table 10.13 Future Maintenance Evaluation for All Site Alternatives

Criteria	Standalone Southdale	Integrated Southdale	Yorktown	Median	Fred Richards
Future Maintenance	O	X	X	OO	X

10.6 Additional Site Considerations

10.6.1 Evaluation Criteria

A few additional site considerations include the operation of the City's distribution system with the addition of WTP No. 5 and the required raw and finished water transmission piping for each alternative. This criterion does not evaluate the financial implications of the additional distribution system components, but rather considers the additional operator maintenance associated with each site. Section 9.3 details the water distribution system analysis conducted for each site. The sections below discuss the variations between the distribution system operation and the impacts on raw and finished water transmission piping for each site.

10.6.2 Distribution System Operation

The water distribution system analysis indicated that with minimal transmission piping additions and connections, selection of the Southdale Site provides the most favorable distribution system operation in terms of water tower balance, handling full capacity from WTP No. 5, and minimizing increases in system pressure. The Median Site provides similar system and water tower operation, but only with an upsized finished water pipe connection into the existing 12-inch France Ave. water main.

The Yorktown Site requires upgrades to the finished water transmission piping along York Ave. before acceptable facility effluent pressures result. This site provided balanced water tower operation across the system.

The Fred Richards Site results indicate elevated system pressures, unbalanced tower operation, and proposed scenario alternatives with results that still limit plant discharge capacities to 3,900 gpm, or 1,100 gpm short of the desired 5,000 gpm produced from WTP No. 3 and WTP No. 5. The extensive transmission piping upgrades and unbalance tower operation make this site the least desirable in terms of distribution system operation.

Table 10.14 summarizes the evaluation of each site related to distribution system operation.

Table 10.14 Distribution System Operation Evaluation for All Site Alternatives

Criteria	Standalone Southdale	Integrated Southdale	Yorktown	Median	Fred Richards
Distribution System Operation	XX	XX	O	_	OO

10.6.3 Raw Water Transmission Pipeline

The raw water transmission pipeline requirements associated with the Median and Yorktown sites are minimal because the City previously installed a 12" HDPE raw water pipeline between Well No. 5 and No. 18. This also benefits the Southdale Site, but the site still requires approximately 500 feet of directionally drilled piping for connecting Well No. 5 to the site, creating slight maintenance increases related to raw water transmission. For the Southdale and Median sites, this existing 12" pipe will bring Well No. 18 to the facility and for the Yorktown Site; Well No. 5 will connect into this pipe.

Raw water pipeline from Well No. 5 and No. 18 to the Fred Richards site does not exist. Bringing these wells to the site requires approximately 7,200 feet of additional water main installation. With this length of pipe comes maintenance of the pipeline and its associated valves and appurtenances. For this reason, the Fred Richards site is the least favorable in terms of additional non-financial considerations of raw water transmission piping. Table 10.15 summarizes the evaluation of each site related to additional raw water transmission pipeline considerations.

Table 10.15 Raw Water Transmission Pipeline Evaluation for All Site Alternatives

Criteria	Standalone Southdale	Integrated Southdale	Yorktown	Median	Fred Richards
Raw Water Pipeline	X	X	XX	XX	OO

10.6.4 Finished Water Transmission Pipeline

The finished water transmission pipeline requirements associated with the Southdale and Median are minimal because France Avenue has an existing finished water main to tie the facility into without creating adverse impacts to the distribution system in terms of pressure and storage facility operation. The Median Site requires 400 feet of directionally drilled pipe to connect the facility directly into the existing water main along France Avenue, which creates some additional maintenance requirements. The Yorktown Site would connect into the water main along York Avenue that is currently only a 10" pipe. The system capacity analysis indicated that this water main must be upsized to handle the proposed 3,000 gpm plant capacity. The implications of this upsizing relate to the cost of installation of the pipe. Since this requires replacement of an existing pipe, no additional maintenance requirements result. The pipeline maintenance is an existing item for the City.

Fred Richards, on the other hand, requires extensive upsizing and addition of finished water main before the proposed facility will operate at full capacity. This site requires approximately 5,300 feet of additional finished water transmission piping to bring maximum day capacity up to 3,900 gpm, still 1,100 gpm below the available capacity of WTP No. 3 and proposed WTP No. 5. With this length of pipe comes maintenance of the pipeline and its associated valves and appurtenances. For this reason, the Fred Richards site is the least favorable in terms of additional non-financial considerations related to finished water transmission piping. Table 10.16 summarizes the evaluation of each site related to additional finished water transmission pipeline considerations.

Table 10.16 Finished Water Transmission Pipeline Evaluation for All Site Alternatives

Criteria	Standalone Southdale	Integrated Southdale	Yorktown	Median	Fred Richards
Finished Water Pipeline	X	X	O	_	OO

10.7 Site Accommodations Evaluation Summary

Evaluation of the site accommodations provides a thorough review of non-financial factors considered in the selection of the preferred site alternative for future WTP No. 5.

In summary, all four (4) sites meet treatment performance objectives and provide relatively low operational complexity. The Southdale and Median sites provide less treatment expandability due to site size limitations. The Median Site provides the least favorable alternative related to operator and public safety due to the vicinity of the site to the France Avenue and 69th Street intersection. Conversations with MDH staff indicated concerns about increased potential for vehicular accidents and potential for vandalism activities with the facility in close proximity to the road.

Overall, an integrated Southdale Site offers the highest and best use of the site in terms of architectural value, shared-use benefit, and planned land use. Without this partnership, the standalone facility lacks the shared-use benefit. The Yorktown and Fred Richards's sites offer appealing opportunities for shared-use, sustainability, and planned land use. The Median Site has the least benefit in terms of architectural value, sustainability, and shared-use.

Fred Richards and the integrated Southdale sites are the most favorable options when considering initial construction and construction staging and sequencing, and future maintenance. The Yorktown Site has disadvantages when considering the extensive utility relocations required before constructing the water treatment plant. Integrating the Southdale Site with Simon Properties expands the possibilities and cooperation for the site related to access and initial construction.

Finally, additional considerations related to distribution system operation and raw and finished water transmission piping result in the Southdale and Median sites being the most favorable. The Fred Richards Site is the least favorable due to the extensive raw and finished water transmission pipelines added as part of this project to get Well No. 5 and No. 18 to the facility and finished water out into the distribution system. Table 10.17 summarizes all criterion evaluated in the site accommodations analysis. Evaluation descriptions and symbols are provided again for reference below Table 10.17.

Table 10.17 Summary of WTP No. 5 Site Accommodations Evaluation

Evaluation Criteria	Standalone Southdale	Integrated Southdale	Yorktown	Median	Fred Richards
Treatment Performance					
Performance Objectives	XX	XX	XX	O	XX
Operational Complexity	_	_	_	X	_
Operational Flexibility	_	O	XX	OO	X
Security and Safety					
Operator Security and Safety	XX	X	XX	OO	XX
Public Security and Safety	_	O	X	OO	X
Site Architecture					
Architectural Value	X	XX	_	O	X
Sustainability / Resiliency	_	_	X	O	X
Shared-Use Feasibility	_	XX	_	OO	X
Land Use	O	X	_	_	_
Constructability					
Initial Construction	O	X	O	OO	XX
Staging / Sequencing	OO	X	X	OO	XX
Future Maintenance	O	X	X	OO	X
Additional Site Considerations					
Distribution System Operation	XX	XX	O	_	OO
Raw Water Pipeline	X	X	XX	XX	OO
Finished Water Pipeline	X	X	O	_	OO

Evaluation Description	Very Unfavorable	Unfavorable	Neutral	Favorable	Very Favorable
Evaluation Symbol	OO	O	_	X	XX

CHAPTER 11 FINANCIAL CONSIDERATION EVALUATION

The Project Team evaluated eight (8) different base facility options for the four (4) site alternatives. This chapter provides a financial evaluation of the estimated total project cost that includes the capital cost of constructing WTP No. 5 and integrating the facility into the City of Edina's existing water distribution system. Additionally, the chapter identifies selected optional premium costs for treatment alternatives feasible for each site.

The recommendations presented in previous chapters created the foundation for estimating each base facility opinion of probable construction cost. In addition, the facility integration components identified for each site in Chapter 9 quantified the required integration construction costs.

Equipment consistent throughout all eight (8) alternatives include three (3), 1,000 gpm filters and chemical feed systems for gaseous chlorine, liquid ammonium sulfate, sodium permanganate, HMO, fluoride, and an ortho/poly blend. With the exception of the Median Site (Option 3A), all base facilities include ton cylinder gaseous chlorine cylinders and a chlorine scrubber. Site size limitations for Option 3A allow for only space for 150 lb cylinders equipped with an automatic shutoff system, instead of a chlorine scrubber. All sites besides the Median Site also have a detention basin for extended pre-oxidant contact time. Option 1C for the Southdale Site includes an above grade plate settler backwash reclaim system, while the other seven (7) options all include traditional backwash settling systems.

In general, the base facilities themselves do not present a significant range of probable construction cost. Significant differences in the costs for integrating the facility into the City's existing distribution system resulted in greater variation in overall estimates of total project cost. The Project Team consolidated the base facility construction cost presented herein into six (6) major categories, which include the following:

1. General Requirements – includes legal and administrative fees, mobilization, temporary facilities and utilities, bonding and insurance, allowances, general demolition, dewatering, and miscellaneous costs.
2. Structural / Architectural – includes costs associated with concrete, masonry, metals, carpentry, thermal and moisture protection, doors and windows, finishes, specialties, and furnishings.
3. Mechanical – includes costs associated with fire protection, plumbing, and mechanical equipment
4. Electrical – includes costs associated with electrical components.
5. Site Work – includes earthwork, exterior improvement, and utility costs.

6. Process Equipment and Integration – includes costs associated with process integration, instrumentation and controls, gas and liquid handling equipment, and water and wastewater equipment.

In addition to these six (6) categories, required integration costs include estimates, when applicable, for raw and finished water transmission pipeline addition, utility relocation, distribution system improvements, and Well No. 5 and No. 18 rehabilitation. A major assumption used throughout each option includes the construction and integration of future Well No. 21 as a separate project, but the facility design capacity includes the 1,000 gpm Well No. 21 will provide. Additionally, rehabilitation estimates for Well No. 5 assume that the City downsizes or upsizes the pump, depending on each site's needs as discussed in Section 9.1. The cost estimations include an optional premium cost for reconstruction Well 5 into a pitless, submersible pump for feasible sites.

11.1 Option 1 – Southdale Site

The first site evaluated is the Southdale Site with three (3) different base facility options. These include Option 1A, Option 1B, and Option 1C previously introduced in Section 8.2.

11.1.1 Option 1A – Southdale Site with Gravity Filters

Option 1A consists of gravity filtration with a traditional backwash reclamation system. ***Appendix G*** provides a preliminary site layout and plan views of the upper and lower levels of the facility. The building is approximately 96-feet east to west by 80-feet north to south, with the buried backwash tanks extending the building an additional 56-feet north. Building height will be approximately 30-feet high at the peak to accommodate the gravity filters and an upper level process area. The structural and architectural estimate incorporates enhanced architectural features to ensure the facility is aesthetically similar to buildings adjacent to the site.

The gravity filters provide a dual purpose: major process equipment and exterior walls for the building. This filter type requires higher concrete costs but reduced process equipment costs. The shoring system associated with construction of the deep excavation backwash reclaim tanks increases the site work costs significantly.

Costs for integrating the facility into the City's existing distribution system are relatively low for Option 1A. This option requires minor raw water and finished water pipeline improvements to connect Well No. 5 into the facility and tie the finished water pipeline into the existing 12-inch main along France Ave. Base integration costs related to the well rehabilitation assume that the City maintains the existing well houses and downsizes the pumps, motors, VFD's, and electrical connections. The Project Team considered reconstruction of Well No. 5 to a submersible pump an optional premium cost.

Table 11.1 provides a combined summary of the facility construction and integration costs. This summary also includes 15-percent contingencies and 15-percent for engineering design and construction phase services. ***Appendix S*** provides a detailed opinion of probable total construction cost for Option 1A.

Table 11.1 Option 1A Construction Cost Summary

Facility Construction Cost		
1	General Requirements	$ 797,000
2	Structural / Architectural	$ 2,766,000
3	Mechanical	$ 490,000
4	Electrical	$ 1,257,000
5	Site Work	$ 1,390,000
6	Process Equipment and Integration	$ 1,908,000
	Facility Construction Subtotal	**$ 8,608,000**
Facility Integration Cost		
1	Raw Water Pipeline	$ 145,000
2	Finished Water Pipeline	$ 55,000
3	Utility Relocation	-
4	Distribution System Improvements	-
5	Well 5 Rehabilitation	$ 100,000
6	Well 18 Rehabilitation	$ 100,000
	Facility Integration Subtotal	**$ 400,000**
	Construction Cost Subtotal	**$ 9,008,000**
	Contingencies (15%)	$ 1,351,000
	Preliminary Opinion of Probable Total Construction Costs	**$ 10,359,000**
	Engineering Design Phase Services (10%)	$ 1,036,000
	Construction Phase Services (5%)	$ 518,000
	Total Project Costs	**$ 11,913,000**

Table 11.2 summarizes the optional premium costs of components feasible for inclusion in Option 1A. If the City selects these components for inclusion in the facility, add these costs to the construction cost subtotal. The costs do not include the contingencies, engineering design phase services, or construction phase service fees.

The current base facility includes a gaseous chlorine system with ton cylinders and a chlorine scrubber. An onsite hypochlorite generation system could replace the gaseous system if desired by the City and operations staff. Addition of a forced draft aerator above the proposed detention basin is another feasible treatment technology that may enhance non-chemical pre-oxidation for the facility. The City also has the option of improving the structural integrity of the Southdale Site options to plan for future commercial or residential development above the facility. Finally, conversion of Well No. 5 to a submersible pump is feasible for this option. This premium cost assumes VFD location within the facility electrical room and demolition of the existing wellhouse. This cost is in addition to the $100,000 well rehabilitation costs for Well No. 5 previously assumed in Table 11.1.

Table 11.2 Optional Premium Costs for Option 1A

Facility Construction Cost		
1	Onsite Hypochlorite Generation	$ 379,400
2	Forced Draft Aeration	$ 350,000
3	Improved Structural Integrity	$ 500,000
4	Conversion of Well No. 5 to Submersible Pump	$ 100,000

11.1.2 Option 1B – Southdale Site with Pressure Filters

Option 1B consists of pressure filtration with a traditional backwash reclamation system. ***Appendix H*** provides a preliminary site layout and plan views of the upper and lower levels of the facility. The building is approximately 90-feet east to west by 108-feet north to south, with the buried backwash tanks extending the building an additional 10-feet north. Building height will be approximately 14-feet to 16-feet high to accommodate the pressure filters and main operating level. The structural and architectural estimate incorporates a premium for enhanced architectural features to ensure the facility is aesthetically similar to adjacent buildings.

The use of pressure filters allows sliding of the backwash reclaim tanks below the main operating level, reducing concrete costs associated with the cover slab of the reclaim tank. The overall smaller size of the facility also decreases the structural and architectural related costs. This option shifts the detention basin below grade, in addition to the backwash tanks, which balances out with the excavation requirements of Option 1A. The shoring system associated with construction of the deep excavation areas increases the site work costs. The pressure filters elevate the process equipment and integration costs compared to gravity filter options.

Costs for integrating the facility into the City's existing distribution system are relatively low for Option 1B. This option requires minor raw water and finished water pipeline improvements to connect Well No. 5 into the facility and tie the finished water pipeline into the existing 12-inch main along France Ave. Base integration costs related to the well rehabilitation assume that the City maintains the existing well houses and downsizes the pumps, motors, VFD's, and electrical connections. The Project Team considered reconstruction of Well No. 5 to a submersible pump an optional premium cost.

Table 11.3 provides a combined summary of the facility construction and integration costs. This summary also includes 15-percent contingencies and 15-percent for engineering design and construction phase services. ***Appendix T*** provides a detailed opinion of probable total construction cost for Option 1B.

Table 11.3 Option 1B Construction Cost Summary

Facility Construction Cost		
1	General Requirements	$ 804,000
2	Structural / Architectural	$ 2,170,000
3	Mechanical	$ 490,000
4	Electrical	$ 1,215,000
5	Site Work	$ 1,390,000
6	Process Equipment and Integration	$ 2,659,000
	Facility Construction Subtotal	**$ 8,728,000**
Facility Integration Cost		
1	Raw Water Pipeline	$ 145,000
2	Finished Water Pipeline	$ 55,000
3	Utility Relocation	-
4	Distribution System Improvements	-
5	Well 5 Rehabilitation	$ 100,000
6	Well 18 Rehabilitation	$ 100,000
	Facility Integration Subtotal	**$ 400,000**
	Construction Cost Subtotal	**$ 9,128,000**
	Contingencies (15%)	$ 1,369,000
	Preliminary Opinion of Probable Total Construction Costs	**$ 10,498,000**
	Engineering Design Phase Services (10%)	$ 1,050,000
	Construction Phase Services (5%)	$ 525,000
	Total Project Costs	**$ 12,072,000**

Table 11.4 summarizes the optional premium costs of components feasible for inclusion in Option 1B. If the City selects these components for inclusion in the facility, add these costs to

the construction cost subtotal. The costs do not include the contingencies, engineering design phase services, or construction phase service fees.

The current base facility includes a gaseous chlorine system with ton cylinders and a chlorine scrubber. An onsite hypochlorite generation system could replace the gaseous system, if desired by the City and operations staff. Addition of a forced draft aerator above the proposed detention basin is another feasible treatment technology that may enhance non-chemical pre-oxidation for the facility. The City also has the option of improving the structural integrity of the Southdale Site options to plan for future commercial or residential development above the facility. Finally, conversion of Well No. 5 to a submersible pump is feasible for this option. This premium cost assumes VFD location within the facility electrical room and demolition of the existing wellhouse. This cost is in addition to the $100,000 well rehabilitation costs for Well No. 5 previously assumed in Table 11.3.

Table 11.4 Optional Premium Costs for Option 1B

Facility Construction Cost		
1	Onsite Hypochlorite Generation	$ 379,400
2	Forced Draft Aeration	$ 350,000
3	Improved Structural Integrity	$ 500,000
4	Conversion of Well No. 5 to Submersible Pump	$ 100,000

11.1.3 Option 1C – Southdale Site with Gravity Filters and Plate Settler

Option 1C consists of gravity filtration with an above grade plate settler backwash reclamation system. ***Appendix I*** provides a preliminary site layout and plan views of the upper and lower levels of the facility. The building is approximately 80-feet east to west by 136-feet north to south. Building height will be approximately 30-feet high at the peak to accommodate the gravity filters and an upper level process area. The structural and architectural estimate incorporates a premium for enhanced architectural features to ensure the facility is aesthetically similar to buildings adjacent to the site.

The gravity filters provide a dual purpose: major process equipment and exterior walls for the building. This filter type requires higher concrete costs but reduced process equipment costs. The above grade plate settler system requires a smaller backwash reclaim tank footprint, providing shallower and smaller excavation areas with this option, reducing site work costs significantly compared to the other options. The smaller reclaim basin also reduces structural costs, but the addition of the above grade plate settler increases the process equipment costs comparted to Option 1A. Overall, this base facility option better utilizes the confined site space and reduces site work costs associated with excavation.

Costs for integrating the facility into the City's existing distribution system are relatively low for Option 1C. This option requires minor raw water and finished water pipeline improvements to connect Well No. 5 into the facility and tie the finished water pipeline into the existing 12-inch main along France Ave. Base integration costs related to the well rehabilitation assume that the City maintains the existing well houses and downsizes the pumps, motors, VFD's, and electrical connections. The Project Team considered reconstruction of Well No. 5 to a submersible pump an optional premium cost.

Table 11.5 provides a combined summary of the facility construction and integration costs. This summary also includes 15-percent contingencies and 15-percent for engineering design and construction phase services. ***Appendix U*** provides a detailed opinion of probable total construction cost for Option 1C.

Table 11.5 Option 1C Construction Cost Summary

Facility Construction Cost		
1	General Requirements	$ 740,000
2	Structural / Architectural	$ 2,346,000
3	Mechanical	$ 490,000
4	Electrical	$ 1,257,000
5	Site Work	$ 660,000
6	Process Equipment and Integration	$ 2,098,000
	Facility Construction Subtotal	**$ 7,591,000**
Facility Integration Cost		
1	Raw Water Pipeline	$ 145,000
2	Finished Water Pipeline	$ 55,000
3	Utility Relocation	-
4	Distribution System Improvements	-
5	Well 5 Rehabilitation	$ 100,000
6	Well 18 Rehabilitation	$ 100,000
	Facility Integration Subtotal	**$ 400,000**
	Construction Cost Subtotal	**$ 7,991,000**
	Contingencies (15%)	$ 1,199,000
	Preliminary Opinion of Probable Total Construction Costs	**$ 9,189,000**
	Engineering Design Phase Services (10%)	$ 911,000
	Construction Phase Services (5%)	$ 459,000
	Total Project Costs	**$ 10,560,000**

Table 11.6 summarizes the optional premium costs of components feasible for inclusion in Option 1C. If the City selects these components for inclusion in the facility, add these costs to the construction cost subtotal. The costs do not include the contingencies, engineering design phase services, or construction phase service fees.

The current base facility includes a gaseous chlorine system with ton cylinders and a chlorine scrubber. An onsite hypochlorite generation system could replace the gaseous system if desired by the City and operations staff. Addition of a forced draft aerator above the proposed detention basin is another feasible treatment technology that may enhance non-chemical pre-oxidation for the facility. The City also has the option of improving the structural integrity of the Southdale Site options to plan for future commercial or residential development above the facility. Finally, conversion of Well No. 5 to a submersible pump is feasible for this option. This premium cost assumes VFD location within the facility electrical room and demolition of the existing wellhouse. This cost is in addition to the $100,000 well rehabilitation costs for Well No. 5 previously assumed in Table 11.5.

Table 11.6 Optional Premium Costs for Option 1C

Facility Construction Cost		
1	Onsite Hypochlorite Generation	$ 379,400
2	Forced Draft Aeration	$ 350,000
3	Improved Structural Integrity	$ 500,000
4	Conversion of Well No. 5 to Submersible Pump	$ 100,000

11.2 Option 2 – Yorktown Site

The second site evaluated is the Yorktown Site with two (2) different base facility options. These include Option 2A and Option 2B, previously introduced in Section 8.3.

11.2.1 Option 2A – Yorktown Site with Gravity Filters

Option 2A consists of gravity filtration with a traditional backwash reclamation system. ***Appendix J*** provides a preliminary site layout and plan views of the upper and lower levels of the facility. The building is approximately 80-feet east to west by 98-feet north to south, with the buried backwash tanks extending the building an additional 56-feet east. Building height will be approximately 30-feet high at the peak to accommodate the gravity filters and an upper level process area. The structural and architectural estimate incorporates architectural features that ensure the facility is aesthetically similar to buildings adjacent to the site.

The gravity filters provide a dual purpose: major process equipment and exterior walls for the building. This filter type requires higher concrete costs but reduced process equipment costs. The Yorktown Site is larger than the Southdale Site, which reduces the excess shoring system costs associated with construction of the backwash reclaim tanks. This is the only differentiator between Option 1A and Option 2A in terms of the facility construction.

Costs for integrating the facility into the City's existing distribution system are significant for Option 2A due to the extensive utility relocation requirements associated with this site. Utility relocations include, but are not limited to, rerouting a 60-inch to 66-inch gravity storm sewer, rerouting a 16-inch storm forcemain, reconstructing a 33-inch sanitary sewer, and other miscellaneous pipes and underdrains the construction limits. All utilities located near buried water holding tanks must maintain a 15-foot offset from the basin, with all pipe constructed to water main grade standards. If not constructed to this standard, the offset increases to a 50-foot minimum. In addition to the major utility relocations present for this site, water distribution system analysis indicated that the adjacent distribution main along York Ave would not handle the 3,000 gpm addition, indicating distribution system improvements are necessary for this site. This option requires minor raw water and finished water pipeline improvements to connect Well No. 5 and No. 18 into the facility and tie the finished water pipeline into the existing main along York Ave. Base integration costs related to the well rehabilitation assume that the City maintains the existing well houses and downsizes the pumps, motors, VFD's, and electrical connections.

Table 11.7 provides a combined summary of the facility construction and integration costs. This summary also includes 15-percent contingencies and 15-percent for engineering design and construction phase services. ***Appendix V*** provides a detailed opinion of probable total construction cost for Option 2A.

Table 11.7 Option 2A Construction Cost Summary

Facility Construction Cost		
1	General Requirements	$ 760,000
2	Structural / Architectural	$ 2,766,000
3	Mechanical	$ 490,000
4	Electrical	$ 1,257,000
5	Site Work	$ 760,000
6	Process Equipment and Integration	$ 1,908,000
	Facility Construction Subtotal	**$ 7,941,000**
Facility Integration Cost		
1	Raw Water Pipeline	$ 65,000
2	Finished Water Pipeline	$ 190,000
3	Utility Relocation	$ 1,750,000
4	Distribution System Improvements	$ 1,500,000
5	Well 5 Rehabilitation	$ 100,000
6	Well 18 Rehabilitation	$ 100,000
	Facility Integration Subtotal	**$ 3,605,000**
	Construction Cost Subtotal	**$ 11,546,000**
	Contingencies (15%)	$ 1,732,000
	Preliminary Opinion of Probable Total Construction Costs	**$ 13,277,000**
	Engineering Design Phase Services (10%)	$ 1,328,000
	Construction Phase Services (5%)	$ 664,000
	Total Project Costs	**$ 15,268,000**

Table 11.8 summarizes the optional premium costs of components feasible for inclusion in Option 2A. If the City selects these components for inclusion in the facility, add these costs to the construction cost subtotal. The costs do not include the contingencies, engineering design phase services, or construction phase service fees.

The current base facility includes a gaseous chlorine system with ton cylinders and a chlorine scrubber. An onsite hypochlorite generation system could replace the gaseous system if desired by the City and operations staff. Addition of a forced draft aerator above the proposed detention basin is another feasible treatment technology that may enhance non-chemical pre-oxidation for the facility.

Table 11.8 Optional Premium Costs for Option 2A

Facility Construction Cost		
1	Onsite Hypochlorite Generation	$ 379,400
2	Forced Draft Aeration	$ 350,000

11.2.2 Option 2B – Yorktown Site with Pressure Filters

Option 2B consists of pressure filtration with a traditional backwash reclamation system. ***Appendix K*** provides a preliminary site layout and plan views of the upper and lower levels of the facility. The building is approximately 108-feet east to west by 90-feet north to south, with the buried backwash tanks extending the building an additional 10-feet east. Building height will be approximately 14-feet to 16-feet high to accommodate the pressure filters and main operating level.

The use of pressure filters allows sliding of the backwash reclaim tanks below the main operating level, reducing concrete costs associated with the cover slab of the reclaim tank. The overall smaller size of the facility also decreases the structural and architectural related costs. This option shifts the detention basin below grade, in addition to the backwash tanks, which balances out with the excavation requirements of Option 2A. The pressure filters elevate the process equipment and integration costs compared to gravity filter options.

Costs for integrating the facility into the City's existing distribution system are significant for Option 2B, similar to Option 2A, due to the extensive utility relocation requirements associated with this site. This option requires minor raw water and finished water pipeline improvements to connect Well No. 5 and No. 18 into the facility and tie the finished water pipeline into the existing main along York Ave. Base integration costs related to the well rehabilitation assume that the City maintains the existing well houses and downsizes the pumps, motors, VFD's, and electrical connections.

Table 11.9 provides a combined summary of the facility construction and integration costs. This summary also includes 15-percent contingencies and 15-percent for engineering design and construction phase services. ***Appendix W*** provides a detailed opinion of probable total construction cost for Option 2B.

Table 11.9 Option 2B Construction Cost Summary

Facility Construction Cost		
1	General Requirements	$ 766,000
2	Structural / Architectural	$ 2,170,000
3	Mechanical	$ 490,000
4	Electrical	$ 1,215,000
5	Site Work	$ 760,000
6	Process Equipment and Integration	$ 2,659,000
	Facility Construction Subtotal	**$ 8,060,000**
Facility Integration Cost		
1	Raw Water Pipeline	$ 65,000
2	Finished Water Pipeline	$ 90,000
3	Utility Relocation	$ 1,750,000
4	Distribution System Improvements	$ 1,500,000
5	Well 5 Rehabilitation	$ 100,000
6	Well 18 Rehabilitation	$ 100,000
	Facility Integration Subtotal	**$ 3,605,000**
	Construction Cost Subtotal	**$ 11,665,000**
	Contingencies (15%)	$ 1,750,000
	Preliminary Opinion of Probable Total Construction Costs	**$ 13,416,000**
	Engineering Design Phase Services (10%)	$ 1,342,000
	Construction Phase Services (5%)	$ 671,000
	Total Project Costs	**$ 15,428,000**

Table 11.10 summarizes the optional premium costs of components feasible for inclusion in Option 2B. If the City selects these components for inclusion in the facility, add these costs to the construction cost subtotal. The costs do not include the contingencies, engineering design phase services, or construction phase service fees.

The current base facility includes a gaseous chlorine system with ton cylinders and a chlorine scrubber. An onsite hypochlorite generation system could replace the gaseous system if desired by the City and operations staff. Addition of a forced draft aerator above the proposed detention basin is another feasible treatment technology that may enhance non-chemical pre-oxidation for the facility.

Table 11.10 Optional Premium Costs for Option 2B

Facility Construction Cost		
1	Onsite Hypochlorite Generation	$ 379,400
2	Forced Draft Aeration	$ 350,000

11.3 Option 3 – Median Site

The third site evaluated is the Median Site with one base facility option, called Option 3A, previously introduced in Section 8.4.

11.3.1 Option 3A – Median Site with Pressure Filters

Option 3A consists of pressure filtration with a traditional backwash reclamation system. ***Appendix L*** provides a preliminary site layout and plan views of the facility. The process equipment building is approximately 132-feet east to west by 36-feet north to south, with the below grade backwash tanks buried east of the process building, which are 122-feet east to west and 34-feet north to south. Process equipment building height will be approximately 14-feet to 16-feet high to accommodate the pressure filters and main operating level.

The elongated site does not accommodate a detention basin, which also eliminates the pumping system required downstream of detention prior to pressure filtration. This reduces costs associated with structural, architectural, electrical, and process equipment components of the facility. The site constrains construction boundaries and requires an extensive shoring system for construction of the pipe chase and backwash reclaim basins, which significantly increases costs associated with site work for this option. The pressure filters elevate the process equipment and integration costs compared to other gravity filter options.

Costs for integrating the facility into the City's existing distribution system are slightly elevated for Option 3A. This option requires minor raw water and finished water pipeline improvements to connect Well No. 5 into the facility and tie the finished water pipeline into the existing 12-inch main along France Ave. The existing main along W 69th Street is not adequately sized to handle the 3,000 gpm finished water capacity and requires upsizing if chosen as the facility effluent tie-in location. This adds distribution system improvement costs if the City selects this approach. Utility relocations associated with this site include rerouting sanitary and storm sewers around the site, maintaining a 50-foot offset. The utilities travel beneath W 69th Street, further increasing the reconstruction and rehabilitation costs associated with the utility relocations. Base integration costs related to the well rehabilitation assume that the City maintains the existing well houses and downsizes the pumps, motors, VFD's, and electrical connections. The Project Team considered reconstruction of Well No. 5 to a submersible pump an optional premium cost.

Table 11.11 provides a combined summary of the facility construction and integration costs. This summary also includes 15-percent contingencies and 15-percent for engineering design and construction phase services. ***Appendix X*** provides a detailed opinion of probable total construction cost for Option 3A.

Table 11.11 Option 3A Construction Cost Summary

Facility Construction Cost		
1	General Requirements	$ 786,000
2	Structural / Architectural	$ 2,135,000
3	Mechanical	$ 490,000
4	Electrical	$ 1,125,000
5	Site Work	$ 1,910,000
6	Process Equipment and Integration	$ 1,971,000
	Facility Construction Subtotal	**$ 8,417,000**
Facility Integration Cost		
1	Raw Water Pipeline	$ 35,000
2	Finished Water Pipeline	$ 135,000
3	Utility Relocation	$ 1,000,000
4	Distribution System Improvements	-
5	Well 5 Rehabilitation	$ 100,000
6	Well 18 Rehabilitation	$ 100,000
	Facility Integration Subtotal	**$ 1,370,000**
	Construction Cost Subtotal	**$ 9,787,000**
	Contingencies (15%)	$ 1,468,000
	Preliminary Opinion of Probable Total Construction Costs	**$ 11,255,000**
	Engineering Design Phase Services (10%)	$ 1,126,000
	Construction Phase Services (5%)	$ 563,000
	Total Project Costs	**$ 12,943,000**

Table 11.12 summarizes the optional premium costs of components feasible for inclusion in Option 3A. If the City selects these components for inclusion in the facility, add these costs to the construction cost subtotal. The costs do not include the contingencies, engineering design phase services, or construction phase service fees.

The current base facility includes a gaseous chlorine system with 150 pound cylinders and an automatic shutoff system, instead of a chlorine scrubber. For an additional cost, the City has the option to install a chlorine scrubber for the site. Site size limitations make an onsite hypochlorite generation system or addition of a forced draft aerator infeasible for this site. Conversion of Well No. 5 to a submersible pump is feasible for this option. This premium cost assumes VFD location within the facility electrical room and demolition of the existing wellhouse. This cost is in addition to the $100,000 well rehabilitation costs for Well No. 5 previously assumed in Table 11.3.

Table 11.12 Optional Premium Costs for Option 3A

Facility Construction Cost		
1	150 lb Cylinder Chlorine Scrubber	$ 90,000
2	Conversion of Well No. 5 to Submersible Pump	$ 100,000

11.4 Option 4 – Fred Richards Site

The fourth site evaluated is the Fred Richards Site with two (2) different base facility options. These include Option 4A and Option 4B, previously introduced in Section 8.5.

11.4.1 Option 2A – Fred Richards Site with Gravity Filters

Option 4A consists of gravity filtration with a traditional backwash reclamation system. ***Appendix M*** provides a preliminary site layout and plan views of the upper and lower levels of the facility. The building is approximately 72-feet east to west by 130-feet north to south, with the buried clearwell extending the building an additional 48-feet east. Building height will be approximately 30-feet high at the peak to accommodate the gravity filters and an upper level process area. The structural and architectural estimate incorporates architectural features that ensure the facility is aesthetically similar to buildings adjacent to the site.

The gravity filters provide a dual purpose: major process equipment and exterior walls for the building. This filter type requires higher concrete costs but reduced process equipment costs. The Fred Richards Site is larger than the Southdale Site, which reduces the excess shoring system costs associated with construction of the backwash reclaim tanks. This option also incorporates a clearwell for storing finished water upstream of the high service pumping chamber, which increases excavation costs slightly compared to Option 2A.

Costs for integrating the facility into the City's existing distribution system are significant for Option 4A due to the extensive raw water and finished water transmission piping upgrades required to tie Well No. 5 and No. 18 into the site and get the 3,000 gpm plant capacity into the system. This includes approximately 7,200 feet of 20-inch HDPE raw water piping and 5,300 feet of 20-inch HDPE finished water piping, which still limits the allowable treated capacity. Base integration costs related to the well rehabilitation assume that the City maintains the existing well houses and downsizes the pumps, motors, VFD's, and electrical connections.

Table 11.13 provides a combined summary of the facility construction and integration costs. This summary also includes 15-percent contingencies and 15-percent for engineering design and construction phase services. ***Appendix Y*** provides a detailed opinion of probable total construction cost for Option 4A.

Table 11.13 Option 4A Construction Cost Summary

Facility Construction Cost		
1	General Requirements	$ 766,000
2	Structural / Architectural	$ 2,766,000
3	Mechanical	$ 490,000
4	Electrical	$ 1,257,000
5	Site Work	$ 870,000
6	Process Equipment and Integration	$ 1,908,000
	Facility Construction Subtotal	**$ 8,057,000**
Facility Integration Cost		
1	Raw Water Pipeline	$ 2,300,000
2	Finished Water Pipeline	$ 1,700,000
3	Utility Relocation	-
4	Distribution System Improvements	-
5	Well 5 Rehabilitation	$ 100,000
6	Well 18 Rehabilitation	$ 100,000
	Facility Integration Subtotal	**$ 4,200,000**
	Construction Cost Subtotal	**$ 12,257,000**
	Contingencies (15%)	$ 1,839,000
	Preliminary Opinion of Probable Total Construction Costs	**$ 14,095,000**
	Engineering Design Phase Services (10%)	$ 1,410,000
	Construction Phase Services (5%)	$ 705,000
	Total Project Costs	**$ 16,209,000**

Table 11.14 summarizes the optional premium costs of components feasible for inclusion in Option 4A. If the City selects these components for inclusion in the facility, add these costs to the construction cost subtotal. The costs do not include the contingencies, engineering design phase services, or construction phase service fees.

The current base facility includes a gaseous chlorine system with ton cylinders and a chlorine scrubber. An onsite hypochlorite generation system could replace the gaseous system if desired by the City and operations staff. Addition of a forced draft aerator above the proposed detention basin is another feasible treatment technology that may enhance non-chemical pre-oxidation for the facility.

Table 11.14 Optional Premium Costs for Option 4A

Facility Construction Cost		
1	Onsite Hypochlorite Generation	$ 379,400
2	Forced Draft Aeration	$ 350,000

11.4.2 Option 2B – Fred Richards Site with Pressure Filters

Option 4B consists of pressure filtration with a traditional backwash reclamation system. ***Appendix N*** provides a preliminary site layout and plan views of the upper and lower levels of the facility. The building is approximately 90-feet east to west by 108-feet north to south, with the buried backwash tanks extending the building an additional 10-feet north. Building height will be approximately 14-feet to 16-feet high to accommodate the pressure filters and main operating level.

The use of pressure filters allows sliding of the backwash reclaim tanks below the main operating level, reducing concrete costs associated with the cover slab of the reclaim tank. The overall smaller size of the facility also decreases the structural and architectural related costs. This option shifts the detention basin below grade, in addition to the backwash tanks, which balances out with the excavation requirements of Option 4A. The pressure filters elevate the process equipment and integration costs compared to gravity filter options.

Costs for integrating the facility into the City's existing distribution system are significant for Option 4A due to the extensive raw water and finished water transmission piping upgrades required to tie Well No. 5 and No. 18 into the site and get the 3,000 gpm plant capacity into the system. This includes approximately 7,200 feet of 20-inch HDPE raw water piping and 5,300 feet of 20-inch HDPE finished water piping, which still limits the allowable treated capacity. Base integration costs related to the well rehabilitation assume that the City maintains the existing well houses and downsizes the pumps, motors, VFD's, and electrical connections.

Table 11.15 provides a combined summary of the facility construction and integration costs. This summary also includes 15-percent contingencies and 15-percent for engineering design and construction phase services. ***Appendix Z*** provides a detailed opinion of probable total construction cost for Option 4B.

Table 11.15 Option 4B Construction Cost Summary

Facility Construction Cost		
1	General Requirements	$ 775,000
2	Structural / Architectural	$ 2,170,000
3	Mechanical	$ 490,000
4	Electrical	$ 1,257,000
5	Site Work	$ 860,000
6	Process Equipment and Integration	$ 2,659,000
	Facility Construction Subtotal	**$ 8,211,000**
Facility Integration Cost		
1	Raw Water Pipeline	$ 2,300,000
2	Finished Water Pipeline	$ 1,700,000
3	Utility Relocation	-
4	Distribution System Improvements	-
5	Well 5 Rehabilitation	$ 100,000
6	Well 18 Rehabilitation	$ 100,000
	Facility Integration Subtotal	**$ 4,200,000**
	Construction Cost Subtotal	**$ 12,411,000**
	Contingencies (15%)	$ 1,862,000
	Preliminary Opinion of Probable Total Construction Costs	**$ 14,273,000**
	Engineering Design Phase Services (10%)	$ 1,427,000
	Construction Phase Services (5%)	$ 714,000
	Total Project Costs	**$ 16,414,000**

Table 11.16 summarizes the optional premium costs of components feasible for inclusion in Option 4B. If the City selects these components for inclusion in the facility, add these costs to the construction cost subtotal. The costs do not include the contingencies, engineering design phase services, or construction phase service fees.

The current base facility includes a gaseous chlorine system with ton cylinders and a chlorine scrubber. An onsite hypochlorite generation system could replace the gaseous system if desired by the City and operations staff. Addition of a forced draft aerator above the proposed detention basin is another feasible treatment technology that may enhance non-chemical pre-oxidation for the facility.

Table 11.16 Optional Premium Costs for Option 4B

Facility Construction Cost		
1	Onsite Hypochlorite Generation	$ 379,400
2	Forced Draft Aeration	$ 350,000

11.5 Capital Cost Evaluation Summary

This chapter presented eight (8) base facility options for the four (4) available site alternatives. These eight (8) options present conceptual design of base facilities that adequately accomplish the treatment goals of WTP No. 5. Based on the facility construction and facility integration costs associated with the evaluated alternatives, the Project Team estimates a cost between $10.56M and $16.41M (2017 dollars). Table 11.17 summarizes the total construction costs of each of the eight (8) options evaluated. ***Appendix AA*** provides a detailed opinion of probable total construction cost, required integration costs, and options premium costs for all eight (8) options.

Table 11.17 Summary of Opinion of Total Construction Costs for WTP No. 5

Site	**Option**	**Facility Construction**	**Facility Integration**	**Contingencies (15%)**	**Engineering & Construction Phases (15%)**	**Total Construction Cost**
Southdale Site	Option 1A	$ 8,608,000	$ 400,000	$ 1,351,000	$ 1,554,000	**$ 11,913,000**
	Option 1B	$ 8,728,000	$ 400,000	$ 1,369,000	$ 1,575,000	**$ 12,072,000**
	Option 1C	$ 7,591,000	$ 400,000	$ 1,199,000	$ 1,370,000	**$ 10,560,000**
Yorktown Site	Option 2A	$ 7,941,000	$ 3,605,000	$ 1,732,000	$ 1,992,000	**$ 15,268,000**
	Option 2B	$ 8,060,000	$ 3,605,000	$ 1,750,000	$ 2,013,000	**$ 15,428,000**
Median Site	Option 3A	$ 8,417,000	$ 1,370,000	$ 1,468,000	$ 1,689,000	**$ 12,943,000**
Fred Richards Site	Option 4A	$ 8,057,000	$ 4,200,000	$ 1,839,000	$ 2,115,000	**$ 16,209,000**
	Option 4B	$ 8,211,000	$ 4,200,000	$ 1,862,000	$ 2,141,000	**$ 16,414,000**

The integration of the facility into the City of Edina's existing distribution system is the largest differentiator in the alternative selection. With preliminary level optimization of the facility at the Southdale Site, facility construction cost reductions resulted. If the Project Team applied a similar treatment technology optimization approach to the Yorktown and Fred Richards sites, facility construction costs may decrease. The extensive facility integration costs associated with these sites outweigh the facility construction cost savings, making these sites remain the least cost effective.

11.6 Life Cycle Cost Considerations

The Project Team did not complete a life cycle cost comparison for the eight (8) base facility alternatives because the treatment technologies present within each facility are relatively the same. Chapter 7 provided life cycle cost comparisons for chemical feed system alternatives, filtration system alternatives, and backwash reclamation alternatives. The list below briefly identifies all treatment technologies previously evaluated in terms of O&M costs over a 30-year planning period in Chapter 7.

1. Chlorine System Alternatives - including gas chlorination, bulk sodium hypochlorite, and onsite hypochlorite generation systems.

2. Ammonia Alternatives – including anhydrous ammonia, liquid ammonium sulfate, and dry ammonium sulfate feed systems.

3. Additional Treatment Chemicals – including sodium permanganate, HMO, fluoride, and an orthophosphate / polyphosphate blend.

4. Pre-Oxidation Alternatives – comparing forced draft aeration to additional chlorine dose based on pilot study results.

5. Filtration System Alternatives – comparing pressure and gravity filtration.

6. Backwash Reclaim System Alternatives – comparing a traditional backwash reclaim system to an above grade plate settler system.

CHAPTER 12 SELECTION OF PREFERRED ALTERNATIVE

12.1 Summary of Alternative Selection Process

This preliminary design report used a multifaceted approach to selection of a preferred site and base facility alternative for WTP No. 5. The following paragraphs define this approach.

Chapter 4 defines the treatment objectives and goals of WTP No. 5. These include standard engineering design criteria, primary drinking water regulations, and treatment goals related to both primary and secondary drinking water regulations based on the known characteristics of the facility's source water. Chapter 5 identifies the treatment process technologies available to meet these objectives and goals. Here the Project Team provides preliminary recommendations of feasible technologies for consideration in the facility.

The Project Team developed pilot study examination protocol, completed preliminary bench scale testing, and conducted a twelve (12) day pilot study. Chapter 6 summarizes all methods, data, results, and conclusions drawn from this analysis. The pilot study confirmed the ability of the chosen treatment technologies to meet treatment objectives and goals with the facility's source water. Further recommendations for treatment technologies included in the base facility resulted from this analysis.

Chapter 7 evaluates the alternatives available for each selected treatment technology. This includes life cycle cost comparisons and additional evaluation factors related to operation and maintenance of each system. Section 12.1.2 summarizes the recommendations identified within this chapter.

Chapter 8 first introduces the eight (8) facility options for the four (4) site alternatives, after determination of the major treatment technologies required for inclusion in the base facilities. Chapter 9 discusses items related to integration of the base facilities into the City's existing distribution system. This evaluates source water well rehabilitation, water tower operation, required improvements to raw and finished water transmission piping, utility relocation needs, and opportunities for increasing the structural integrity of the Southdale Site options for future development above the proposed facility. Section 12.1.3 discusses the major takeaways of the facility integration analysis.

The Project Team evaluated each site's ability to accommodate various non-financial criterion related to treatment performance, security and safety, site architecture, constructability, and additional infrastructure considerations in Chapter 10. The City, jointly with AE2S and Oertel Architects, provided opinions of favorability to identify the non-financial benefits and tradeoffs of each site. Section 12.1.4 provides site selection recommendations based on this evaluation.

Finally, Chapter 11 provides a financial consideration evaluation of the total construction cost of each base facility, including the facility construction and additional facility integration costs associated with each option. Section 12.1.5 discusses the financial tradeoffs identified in this chapter.

From these evaluations, Section 12.6 provides a recommended preferred site alternative.

12.2 Treatment Technology Evaluation

For many of the treatment technologies evaluated there are multiple treatment alternatives available to meet the same goal. Chapter 7 presented life cycle cost analyses for the alternatives feasible in future WTP No. 5.

Chemical alternatives selected for the base facilities include gaseous chlorine, liquid ammonium sulfate, sodium permanganate, HMO, fluoride and an ortho / poly blend. Chapter 7 indicates a gaseous chlorine feed system based on life cycle cost comparisons, operator familiarity, non-degradation of the chemical, and safety enhancements with inclusion of a chlorine scrubber as part of the system. Another feasible option, identified in Chapter 11 as a premium technology, is an onsite hypochlorite generation system.

The Project Team based liquid ammonium sulfate selection on ease of operation, maintenance, and non-degradation of the chemical. With the current source water quality, required ammonia dose is relatively low. It is important to note that future Well No. 21 may influence the required ammonia feed. If the raw water is high in ammonia, WTP No. 5 ammonia feed requirements will decrease, and if raw water ammonia is low, will increase ammonia feed.

The treatment technology evaluation also reviewed pre-oxidation alternatives, comparing the addition of forced draft aeration to additional chlorine dose. The pilot study concluded that aeration reduces the chlorine demand of Well No. 18 raw water by approximately 0.1 mg/L FAC. Without aeration, a slight increase in chlorine dose accomplishes pre-oxidation. This result and the life cycle cost comparison indicate that addition of an aerator is not necessary with the current source water quality.

The evaluation also compared the 30-year O&M life cycle costs for gravity versus pressure filtration. The gravity filtration option has less O&M costs over the analysis period. Both filtration alternatives remained in the analysis, resulting in gravity and pressure filter options for all but the Median Site. A gravity filter option at the Median Site is not feasible due to the long, narrow, site constraints.

Comparing life cycle costs for traditional and above grade plate settler backwash reclaim systems indicates that the more feasible option for WTP No. 5 is an above grade plate settler system. The majority of the base facilities include traditional backwash reclaim systems because the site has the space available to install traditional reclaim tanks. For sites with small footprints, the above grade plate settler system may realize additional cost savings by minimizing excavation and the required shoring system to accommodate steep slopes.

12.3 Facility Integration Evaluation

Integration with the existing infrastructure is a critical part of the PDR process. The new WTP must operate seamlessly with the existing distribution system and the Project Team must consider the facility's impacts on existing infrastructure.

Rehabilitation of Well No. 5 and No. 18 is necessary in some capacity for each of the four (4) site alternatives. For the Southdale, Yorktown, and Fred Richards's sites, pump rehabilitation includes downsizing the pump impeller, motor, VFD, and electrical connection. The pumps currently tie directly into a 90 to 100 psi distribution system pressure, but in the future will tie directly into the treatment facility and enter the detention basin, which requires significantly less pressure. The exception is the Median Site, which maintains system pressure through the treatment facility. The treatment train increases head loss, requiring a larger pump for overcoming system pressures. The City also has the option of converting Well No. 5 to a submersible pump if the future facility is on the Southdale or Median Sites.

The only site with accommodating space for onsite storage is the Fred Richards Site. The gravity filter options include clearwells providing approximately 100 minutes of maximum hour demand (MHD) assuming the facility operates at the full 3,000 gpm capacity. The storage analysis conducted as part of Section 3.4 indicated that the City's current storage is adequate; assuming that the Dublin reservoir pump capacity meets MHD typically seen in Edina. If this assumption is invalid, Edina may consider making pumping improvements to the pump station or construct additional elevated storage.

The City's water distribution system consultant completed an analysis on the impacts of the proposed WTP No. 5 at the various sites and under multiple scenarios. The analysis determined that overall, the Southdale Site results in the most desirable operation of water towers and minimizes impacts on the distribution system in terms of pressure increases and pipe velocities, requiring no upgrades to existing infrastructure aside from tying in the proposed facility to the 12-inch main along France Ave. The Median Site produced similar results if tied into the same 12-inch water main along France Ave. The analyzed scenarios determined the existing 8-inch main along W 69th Street would not handle the 3,000 gpm capacity. The Yorktown Site requires extensive infrastructure upsizing along York Ave. before eliminating any pressure related or finished treatment capacity concerns. Similarly, the Fred Richards site scenarios resulted in unbalanced tower operation and pressure increases in excess of 35 psi. This site requires substantial improvements to finished water transmission piping before becoming a viable option.

In terms of raw and finished water transmission piping, the Southdale, Median, and Yorktown sites require relatively limited improvements. In addition to the finished water main requirements of the Fred Richards Site, the site also requires over 7,200 feet of raw water transmission piping to connect the two existing wells into the proposed facility. The Median and Yorktown Sites also require extensive utility relocations of both storm and sanitary sewer pipelines.

With all infrastructure integration components considered, the Southdale Site is the most favorable option for future WTP No. 5.

12.4 Site Accommodations Evaluation

Evaluation of the site accommodations provides a thorough review of non-financial factors considered in the selection of the preferred site alternative for future WTP No. 5.

All four (4) sites meet treatment performance objectives and provide relatively low operational complexity. The Southdale and Median sites provide less future treatment expandability due to site size limitations. The Median Site provides the least favorable alternative related to operator and public safety. This is not because the treatment facility lacks safety or security; this is solely due to site access limitations and public perception of hazardous chemical delivery in a highly commercial and populated area.

Overall, an integrated Southdale Site offers the highest and best use of the site in terms of architectural value, shared-use benefit, and planned land use. Without this partnership, the standalone facility lacks the shared-use benefit. The Yorktown and Fred Richards's sites offer appealing opportunities for shared-use, sustainability, and planned land use. The Median Site has the least benefit in terms of architectural value, sustainability, and shared-use.

Fred Richards and the integrated Southdale sites are the most favorable options when considering initial construction and construction staging and sequencing, and future maintenance. The Yorktown Site has disadvantages when considering the extensive utility relocations required before constructing the water treatment plant. Integrating the Southdale Site with a public/private partnership expands the possibilities and cooperation for the site related to access and initial construction.

Finally, additional considerations related to distribution system operation and raw and finished water transmission piping result in the Southdale and Median sites being the most favorable. The Fred Richards Site is the least favorable due to the extensive raw and finished water transmission pipelines added as part of this project to get Well No. 5 and No. 18 to the facility and finished water out into the distribution system.

In summary, the integrated Southdale Site is the most favorable in terms of non-financial site accommodations. Based on multiple conversations with Simon Properties, the likelihood of this public/private partnership is low. With this in mind, the standalone Southdale Site is the second most favorable alternative, with accommodations limitations related to shared-use benefit and constructability.

12.5 Financial Consideration Evaluation

Chapter 11 provides opinions of total construction costs for the eight (8) evaluated facility options. The Project Team estimates a cost between $10.56M and $16.41M (2017 dollars) for WTP No. 5, including facility construction, facility integration into the distribution system, 15-percent contingencies and 15-percent for engineering design and construction phase services.

Facility construction costs ranged from $7.59M to $8.73M, which is a relatively small range when compared to the total construction cost estimates. This indicates that the biggest differentiator between the sites is the facility integration costs, with the Yorktown and Fred Richards's sites requiring the most extensive improvements. Based on total construction cost, Option 1C of the Southdale Site, which includes gravity filtration with an above grade plate settler backwash reclaim system, is most favorable. This option limits facility integration costs and reduces site work costs associated with excavation and a shoring system with the addition of the above grade plate settler system, rather than a tradition backwash reclaim system.

12.6 Recommended Alternative

Option 1C of the Southdale Site meets all desired treatment objectives and goals set by the Project Team for WTP No. 5. Distribution system modeling expects selection of this site produces minimal increases in pressure in the system and actually helps with the currently lagging Southdale Tower operation.

The site may present some challenges with constructability due to small construction extents and lack of staging or stockpiling area. The site also lacks a shared-use benefit due to the denial of a public/private partnership with Simon Properties after multiple attempts to date. The City has the option of building a more robust foundation for WTP No. 5, providing opportunities for commercial or retail space above the facility in the future. Preliminary estimation of this premium cost is $500,000.

Option 1C offers the best use of the City's financial resources while including both $1.20M for contingencies and $1.37M for engineering, administrative, and legal fees associated with the project. ***Appendix AB*** provides the architectural renderings developed for Option 1C on the Southdale Site.

CHAPTER 13 CONCLUSIONS AND RECOMMENDATIONS FOR IMPLEMENTATION

The City of Edina's Water Utility primarily consists of eighteen (18) wells, with seventeen (17) currently active, four (4) regional WTPs, approximately 200 miles of water main, and five (5) finished water storage structures.

Providing an adequate supply of high-quality finished water to both current and future customers at a reasonable cost is a primary goal of the City of Edina, which requires ongoing investment in the City's water system infrastructure. It is important for the City to address issues such as maintaining an adequate raw water supply, adjustments to changing raw water quality, optimization of existing systems, and replacement of aging infrastructure to ensure the sustained success of the City's Water Utility.

13.1 Overview of Existing Water Supply, Treatment, and Distribution System

13.1.1 Existing Raw Water Supply

The City of Edina has eighteen (18) active groundwater appropriation permits authorized by the Minnesota Department of Natural Resources (MnDNR). The cumulative appropriations provide for an instantaneous withdrawal rate of 17,650 gpm (approximately 25.42 MGD) and a total annual withdrawal volume of 3,000 MG/year (which equates to an average daily withdrawal of 8.22 MGD). Each of the eighteen (18) raw water supply wells is unique in its location and production capabilities. Table 2.1 presents the pumping rate for each well.

Table 13.1 Well Characteristics and Pumping Rates

Well Name	Unique Well No.	Well Use Status	Well Depth (ft.)	Source Aquifer[2]	Pumping Rate (gpm)
No. 2	208399	Active	448	OPDCCJDN	850
No. 3	240630	Active	496	OPDCCJDN	1,000
No. 4	200561	Active	500	OPDCCJDN	1,000
No. 5	206377	Active	443	OPDCCJDN	1,000
No. 6	200564	Active	505	OPDCCJDN	1,000
No. 7	206474	Active	547	OPDCCJDN	1,000
No. 8	204884	Active	472	OPDCCJDN	1,000
No. 9	206588	Inactive[1]	1,130	CMTS	1,000
No. 10	206184	Active	1,001	CMTS	800

Well Name	Unique Well No.	Well Use Status	Well Depth (ft.)	Source Aquifer[2]	Pumping Rate (gpm)
No. 11	206183	Active	403	CJDN	1,000
No. 12	203614	Active	1,080	CMTS	1,000
No. 13	203613	Active	495	CJDN	1,000
No. 14	200913	Inactive[1]	420	CJDN	1,000
No. 15	207674	Active	475	OPDCCJDN	1,000
No. 16	203101	Active	381	OPDCCJDN	1,000
No. 17	200914	Active	461	CJDN	1,000
No. 18	200918	Active	446	CJDN	1,000
No. 19	505626	Active	520	CJDN	1,000
No. 20	686286	Active	467	CJDN	850
				Total	**17,650** **(25.42 MGD)**

[1] Inactive due to high levels of radium in the raw water. Well 9 inactive since 2010.
[2] OPDCCJDN: Prairie du Chien – Jordan, CMTS: Mt. Simon, CJDN: Jordan.

13.1.2 Wells to Service WTP No. 5

Well No. 5 is located south of the Southdale Tower within the median of W 69th Street on the east side of the France Ave S and W 69th St intersection. Original drilling of the well occurred in 1950. Bergerson-Caswell reconstructed the well in 2002. The water level during pumping is approximately 90 feet below the surface. During reconstruction of the well, test pumping indicated that the well has a specific capacity of approximately 24.0 gpm/ft. The existing pump is a 100 horsepower (Hp) J-Line vertical turbine pump designed to pump 1,000 gpm at an estimated total dynamic head (TDH) of 310 feet.

Well No. 18 is located along York Ave S in the parking lot of Edina Fire Station No. 2. Keys Well Drilling Company drilled the well back in 1973. The water level during pumping is approximately 90 feet below the surface. The well has a specific capacity of approximately 83.3 gpm/ft. The existing pump is a 125 horsepower (Hp) Peerless vertical turbine pump designed to pump 1,000 gpm at an estimated total dynamic head (TDH) of 320 feet.

The location of the third well, future Well No. 21, is currently unknown, but this preliminary design report assumes sizing of base facilities based on this well providing an additional 1,000 gpm of flow to WTP No. 5, increasing the facilities treated capacity to 3,000 gpm.

13.1.3 Existing Treatment System

The existing water treatment plants treat ten (10) of the City's wells within the four (4) existing WTPs where the primary treatment goal is removal of iron and manganese through oxidation and granular filtration. The newest of the facilities, WTP No. 6, includes air-stripping towers to remove vinyl chloride and provisions to install the equipment to feed HMO within the Well No. 6 well house to remove the well's high radium levels. WTPs No. 3 and No. 4 already include the HMO feed equipment to remove high levels of radium chemically from the raw water.

Table 13.2 summarizes the source water wells, well pumping capacities, and design and current capacities of the four (4) existing facilities.

Table 13.2 Summary of Existing Water Treatment Plant Capacities

Treatment Facility	Well ID	Pumping Capacity (gpm)	Combined Plant Capacity (gpm) Design/Current	Combined Plant Capacity (MGD) Design/Current
WTP No. 2	No. 4	850	2,850	4.10
	No. 6	1,000		
	No. 17	1,000		
WTP No. 3	No. 10	1,000	2,000	2.88
	No. 11	1,000		
WTP No. 4	No. 12	1,000	2,000	2.88
	No. 13	1,000		
WTP No. 6	No. 2	1,000	4,000 / 2,500[2]	5.76 / 3.60
	No. 7	1,000		
	No. 9[1]	1,000		
	No. 15	1,000		
Currently Unfiltered	No. 3	1,000		
	No. 5	1,000		
	No. 8	800		
	No. 16	1,000		
	No. 18	1,000		
	No. 19	1,000		
	No. 20	1,000		
Existing Filter Water Capacity			**10,850 / 9,350**	**15.6 / 13.5**

[1] Well No. 9 currently inactive.

[2] Combined plant capacity is limited by the facility effluent piping. Distribution pressure is too high with all wells operating at full pumping capacity.

All four (4) existing facilities have dual pressure filters with silica sand filtration media originally designed to remove raw water iron and manganese. The exception to the silica sand media is in WTP No. 6, which has conventional dual sand and anthracite media. Pre-chlorine feed at the well sites oxidizes the iron and manganese prior to filtration. All facilities are equipped with chemical feed systems to dose chlorine for oxidation and disinfection, fluoride for public health and wellness, and an orthophosphate / polyphosphate blend to inhibit pipe corrosion. Post-chlorination equipment exists, but the City does not use it for current operation.

Every facility includes a backwash reclaim system that reclaims backwash water to the front of the system and wastes sludge to sanitary. Current operations base filter backwash frequency on an iron breakthrough concentration of 0.1 to 0.15 mg/L, depending on the facility.

13.1.4 Existing Distribution System and Storage Capacity

The City's distribution system totals approximately 200 miles of water main ranging from four (4) to sixteen (16) inches in diameter. In general, the distribution system is well looped throughout the City, but a few un-looped areas still exist. The City continuously pursues maintenance of old, unlined cast-iron mains and it is typically in the form of full pipe replacement or restoration by pipe lining. Providing the water main looping also ensures adequate water supply for fire flow. The Edina System includes areas with average day pressures ranging from 40 psi up to over 100 psi. In general, the southern third of the City has high pressures above 90 psi in the highly commercial areas.

The City has five (5) finished water storage structures located throughout the distribution system. The Dublin Reservoir and the Gleason Road Tower are located in the southwest quadrant of the City, the Community Center Tower is directly adjacent to WTP No. 2, the Van Valkenburg is near WTP No. 4, and the Southdale Tower is by Well No. 5. Table 13.3 summarizes the type of structure and storage capacity for each of the existing storage facilities in Edina. Conversations with operations staff identified that tower levels control well operation.

Table 13.3 Edina Storage Structure Summary

Storage Structure Name	Year Constructed	Type	Storage Capacity (MG)
Dublin Reservoir	1960	Ground	4.0
Gleason Road Tower	1970	Elevated	1.0
Community Center Tower	1955	Elevated	0.5
Van Valkenburg Tower	1989	Elevated	1.0
Southdale Tower	1956	Elevated	0.5
		Total	**7.0**

Source: 2008 Edina Comprehensive Plan

13.2 Overview of Population and Water Demand

According to the Metropolitan Council's Thrive 2040 population forecasts, Edina is likely to experience moderate growth through 2040. Edina's 2000 population was 47,425 and its projected 2040 population is 54,400. In addition, Edina is currently conducting additional water supply analysis that provides a more detailed review of the potential water demands and commercial development in various portions of the City. These projections may further inform policy makers of the overall water supply needs. For the purposes of this report, the Project Team utilized the Metropolitan Council population projections and the historical water use data to determine the adequacy of the water supply system for Edina.

Since 2000, the average gallon per capita per day water usage has been approximately 147.12 gpcd and has shown to be decreasing over the past 5 to 10 years. The future water use projections use a 158.62 gpcd average day water demand as a conservative estimation of average occurrences in the past 10 years. Based on this assumption, the estimated average day demand for the City of Edina will be approximately 8.63 MGD in 2040.

The Project Team also projects an approximate maximum day demand of 25.89 MGD for 2040. The maximum day demand is critical to the long term planning of a utility because it determines the highest demand likely experienced by the water utility. In 2016, the City of Edina experienced a peaking factor of approximately 2.15. Previous Edina water demand analysis utilized a peaking factor of 3.0. Peaking factors vary on an annual basis for a number of reasons. The 3.0 peaking factor applied from 2000 through 2008 and 2017 through 2040 estimates the anticipated past and future maximum day demands.

The review of the historical water use and future projections indicates that WTP No. 5 will likely provide the City of Edina with additional treatment capacity that will further eliminate the need to utilize the unfiltered wells during peak day scenarios in the summer months. WTP No. 5 will also provide additional treatment redundancy to allow City staff more operational flexibility during maintenance or emergencies.

As illustrated in Figure 13.1, Edina's current firm well capacity is 21.58 MGD, which is below the projected maximum day demands in 2040 of 25.89 MGD. The firm capacity was determined by removing one of the many 1,000 gpm wells from the system, which considers the largest well for the City of Edina. Additional firm capacity assumptions include limiting the WTP No. 6 plant production from the remaining available wells (Well No. 2, No. 7, and No. 15) to 2,500 gpm due to current distribution system limitations. The reconstruction of Well No. 9 is currently underway, so the firm capacity assumes the well is active.

This graph shows that the current firm well capacity is below the projected maximum day demand through 2040, indicating that additional wells may be necessary in the future to meet system demands. The firm well and well capacities represented in the figure do not include the 1,000 gpm additional capacity planned for future Well No. 21, anticipated for inclusion in the treated capacity of WTP No. 5. Recall that this projection uses a total annual gallon per capita per day of 158.62 gpcd for calculation of average day demand based on population projections and a 3.0 peaking factor for determining maximum day demands.

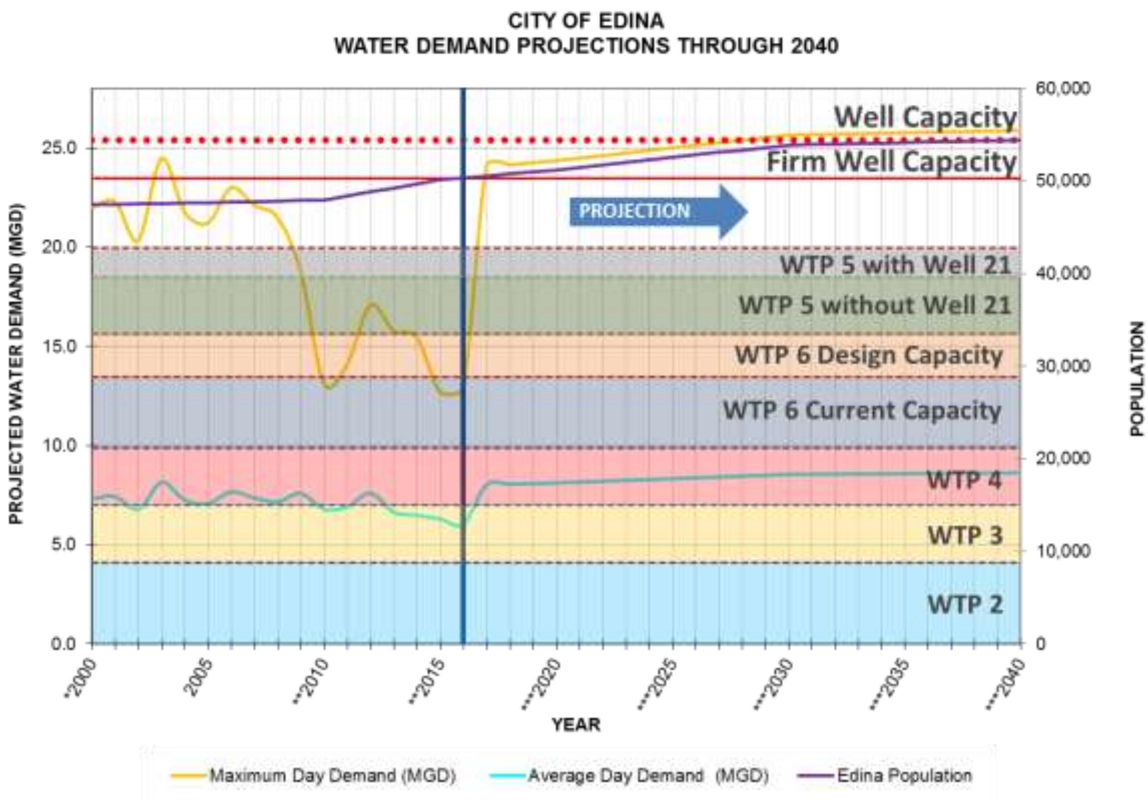

Figure 13.1 Projected Water Use Analysis

Based on Figure 13.1, it is prudent for Edina to investigate the addition of another well to meet projected maximum day demands. This well would likely be Well No. 21, which would provide WTP No. 5 with the final 1,000 gpm of planned capacity. If Well No. 21 produces 1,000 gpm, the need for a well in addition to Well No. 21 is likely to provide a firm well capacity equivalent to the projected 2040 maximum Day demand.

13.3 Overview of Water Storage Considerations

For the purposes of this analysis, the Project Team assumed that the total storage needed is equalization and the greater of either fire flow storage or emergency storage. Given the developed demographic of Edina and the current buildings within the city, fire storage controls for this analysis. Table 13.4 shows a preliminary assessment of storage volumes required based on an initial water demand and fire flow assessment of the water system. Based on this analysis, Edina has a prudent amount of storage volume within the City. The Project Team suggests that the City conduct a more specific pressure zone evaluation to analyze zone specific storage requirements and fire flow demands. Additionally, the Project Team suggests evaluation of the Dublin ground storage reservoir pumping capacity to confirm that it can meet the maximum hourly demand for its service area. If it is unable to meet the projected maximum hourly demand, we suggest removal of the 4.0 MG from the total 7.0 MG total available storage, leaving 3.0 MG remaining available storage.

Table 13.4 Water Storage Volume Requirements

Equalization Storage	Based on 20 percent of the 2040 MDD for Edina.	5.17 MG
Fire Storage	Based on 3,000 gpm fire demand for 3 hr duration	0.5 MG
	Total	**5.67 MG**

13.4 Overview of Treatment Options

This preliminary design report evaluates eight (8) base facilities that include the treatment technologies necessary for achieving the treatment objectives and goals of WTP No. 5. Technologies included within all eight (8) facilities are chemical feed systems for chlorine, permanganate, ammonia, HMO, fluoride, and an ortho/polyphosphate blend, filtration systems, and backwash reclaim systems.

Chapter 7 evaluated options for a few of these treatment processes. For the chlorine system, the Project Team recommends implementation of a gaseous chlorination system in the new WTP employing the use of a wet scrubber at this time based on operator familiarity, non-degradation of the chemical, and life cycle costs. For the ammonia feed system, the Project Team recommends implementation of a liquid ammonium sulfate system in the new WTP based on ease of operation, maintenance, and non-degradation of the chemical. The long term annual O&M expenses are less desirable than other ammonia alternatives, but the lower initial capital investment and smaller fixed O&M costs make this system a favorable option for WTP No. 5. It is important to note that future Well No. 21 may influence the required ammonia feed. If the raw water is high in ammonia, WTP No. 5 ammonia feed requirements will decrease, and if raw water ammonia is low, will increase ammonia feed.

The following sections review additional treatment options evaluated in Chapters 5, 6, and 7.

13.4.1 Pre-Oxidation

Removal of dissolved iron and manganese is primarily achieved by oxidizing the soluble, reduced forms (Fe^{+2}, Mn^{+2}) to the oxidized forms (Fe^{+3}, Mn^{+4}). The oxidized ions precipitate and form particles that the filters then remove. Water treatment facilities commonly provide thirty (30) minutes of reaction time in a detention tank after oxidation and prior to filtration, depending on the relative concentrations of iron and manganese and the type of media used for filtration.

Oxidation processes occur by reaction with a chemical oxidant dosed to the water, or on the surface of an oxidizing filter media, such as manganese greensand or pyrolusite. Pre-oxidation processes utilize a chemical to oxidize the iron and manganese prior to filtration. Candidate chemical oxidants include oxygen, chlorine, and permanganate. Oxygen is typically added to water through an aeration process, whereas chlorine and permanganate are typical dosed to the water via a chemical feed system. As noted previously, the Project Team selected chlorine and permanganate addition for WTP No. 5.

13.4.2 Filtration Processes

Water treatment facilities typically use filtration as a polishing step for the removal of suspended solids and particles from water. For ground water sources, oxidation of iron and manganese, coagulation, and lime softening often precedes filtration. Excluding membrane filtration technology, there are four general classes of filters including rapid rate gravity filters, rapid rate pressure filters, diatomaceous earth filters, and slow sand filters. Based on industry trends, treatment facility footprint considerations, and operator convenience, the Project Team deemed gravity filters and pressure filters most appropriate in the treatment concepts developed for this report.

13.4.3 Chlorination

The Project Team evaluated three options for chlorine addition at the future WTP No. 5. The chlorination processes selected for evaluation included:

1. The City's current chlorination process, gaseous chlorination;
2. Bulk delivery of sodium hypochlorite; and
3. Onsite generation of sodium hypochlorite.

Evaluation included considerations of initial capital including building footprint, operation and maintenance (O&M) costs, and the advantages and disadvantages of implementation. The purpose of providing such information is in an effort to develop a preliminary recommendation related to a preferred chlorination process for the proposed facility.

13.4.4 Radium Removal

Installation of manganese greensand media with permanganate and preformed HMO addition is common. This treatment train provides a pre-oxidation and filtration system that removes iron, manganese, and radium simultaneously is common. Permanganate oxidizes the iron and manganese and provides continuous regeneration of the filter media and radium adheres to the HMO particles. Additionally, the manganese greensand media aids in radium removal by adsorption of the radium particles on to the media coating. In recent years, facilities with this type of treatment system have had problems with the disposal of the filter media once it reaches its usable lifespan. Radium accumulation occurs over time and results in a media classified as radioactive waste. Based on this knowledge, the Project Team does not recommend manganese greensand media use for the proposed facility.

13.4.5 Backwash Recovery / Recycle Processes

During the treatment process, filters regularly undergo backwashes for removal of built up particulates. This backwash water is then routed to either the sanitary sewer or some sort of backwash reclamation facility. As water resources in the area become more and more scarce, the use and re-use of water will become a more important topic for large water producers and individual water consumers. While a financial investment is required for the re-use of backwash water, providing good stewardship of the state's resources is a primary concern for the City of Edina. Facilities achieve backwash reclamation through traditional settling basins or through a treatment and recycle process utilizing plate settlers.

13.5 Development of Alternatives

As part of this preliminary design report, the Project Team developed eight (8) alternative base facilities for the four (4) available site alternatives. The Team then used a multifaceted approach to selection of a preferred site and base facility alternative for WTP No. 5, which included thorough investigation of viable treatment technologies, requirements for integrating the facility into the existing distribution system, non-financial tradeoffs of each site, and financial evaluations of all eight (8) alternatives.

13.5.1 Site Alternatives

Siting and planning for WTP No. 5 began over a decade ago. Since then, the City, with their consultants, completed a feasibility study in 2007, conducted a water system demand and capacity analysis in 2013, and developed various architectural concepts over the years, all related to WTP No. 5. The City also secured easements and partially extended raw water pipes to the preferred site property adjacent the Southdale Tower, known as the Southdale Site.

With economic development and environmental sustainability in mind, the City determined alternative sites for consideration in the preliminary design report. Originally, these sites included the Yorktown Site, located near the YMCA and Fire Station No. 2, and the Median Site, located along West 69th Street, directly east of the wellhouse of Well No. 5. Throughout the PDR process the City added a fourth site, the Fred Richards Site, located immediately adjacent to the existing WTP No. 3, which would potentially take the place of WTP No. 3 in the future.

The Southdale Site offers a unique location for a water treatment facility in a highly commercial area planned for extensive re-development in the near future. Initial visions of the City and the design team included a shared-use facility integrating the water treatment plant into a mixed use commercial and residential building. As the PDR process progressed, response from Simon Properties indicated low likelihood of the public/private partnership for a shared-use facility. The Project Team developed three (3) options for this site, including Option 1A with gravity filtration and traditional backwash reclaim, Option 1B with pressure filtration and traditional backwash reclaim, and Option 1C with gravity filtration and an above grade plate settler backwash reclaim system.

The second site alternative is the Yorktown Site located along York Avenue just north of the Southdale YMCA and Fire Station No. 2. The site, currently called Yorktown Park, is an open green space with trails and walking paths connected to other major parks within the Southdale Area, including the Edina Promenade, Centennial Lakes Park, and Adams Hill Park of Richfield, MN. The Project Team developed two (2) options for this site; including Option 2A with gravity filtration and traditional backwash reclaim and Option 2B with pressure filtration and traditional backwash reclaim system.

The third site alternative is the Median Site located within the median of South 69th Street, directly east of Well No. 5. This site offers the opportunity to integrate a water treatment facility into an area typically deemed undevelopable in a similar road corridor. This site is compact compared to the others, limiting the layout, size, and treatment flexibility of the facility. The Project Team developed Option 3A for this site, which includes a pressure filtration and traditional backwash reclaim system.

The fourth, and final, site alternative is the Fred Richards Site located near the golf course facilities that closed down back in 2014. The site includes WTP No. 3, which treats raw water from wells No. 10 and No. 11. This site became an alternative throughout the PDR process after the City began planning the re-development of the golf course into multi-use infrastructure including athletic fields, parks, restaurants, event space, and more. A similar vision of integrating the water treatment plant into the shared-use development, along with the aging infrastructure of WTP No. 3, made this site a feasible alternative for WTP No. 5. The Project Team developed two (2) options for this site; including Option 4A with gravity filtration and traditional backwash reclaim and Option 4B with pressure filtration and traditional backwash reclaim system.

13.5.2 Alternative Facility Integration Evaluation

Integration with the existing infrastructure is a critical part of the PDR process. The new WTP must operate seamlessly with the existing distribution system and the Project Team must consider the facility's impacts on existing infrastructure.

Rehabilitation of Well No. 5 and No. 18 is necessary in some capacity for each of the four (4) site alternatives. For the Southdale, Yorktown, and Fred Richards's sites, pump rehabilitation includes downsizing the pump impeller, motor, VFD, and electrical connection. The City also has the option of converting Well No. 5 to a submersible pump if the future facility is on the Southdale or Median Sites.

The only site with accommodating space for onsite storage is the Fred Richards Site. The gravity filter options include clearwells providing approximately 100 minutes of maximum hour demand (MHD) assuming the facility operates at the full 3,000 gpm capacity. The storage analysis conducted as part of Section 3.4 indicated that the City's current storage is adequate; assuming that the Dublin reservoir pump capacity meets MHD typically seen in Edina. If this assumption is invalid, Edina may consider making pumping improvements to the pump station or construct additional elevated storage.

The distribution system analysis determined that overall the Southdale Site results in the most desirable operation of water towers and minimizes impacts on the distribution system in terms of pressure increases and pipe velocities. The Median Site produced similar results if tied into the same 12-inch water main along France Ave. The Yorktown Site requires extensive infrastructure upsizing along York Ave. before eliminating any pressure related or finished treatment capacity concerns. This Fred Richards Site requires substantial improvements to finished water transmission piping before becoming a viable option. In addition to the finished water main requirements of the Fred Richards Site, the site also requires substantial raw water transmission piping to connect the two existing wells into the proposed facility. The Median and Yorktown Sites also require extensive utility relocations of both storm and sanitary sewer pipelines.

With all infrastructure integration components considered, the Southdale Site is the most favorable option for future WTP No. 5.

13.5.3 Alternative Site Accommodations Evaluation

Evaluation of the site accommodations provides a thorough review of non-financial factors considered in the selection of the preferred site alternative for future WTP No. 5.

In summary, all four (4) sites meet treatment performance objectives and provide relatively low operational complexity. The Southdale and Median sites provide less treatment expandability due to site size limitations. The Median Site provides the least favorable alternative related to operator and public safety due to the vicinity of the site to the France Avenue and 69th Street intersection. Conversations with MDH staff indicated concerns about increased potential for vehicular accidents and potential for vandalism activities with the facility in close proximity to the road.

Overall, an integrated Southdale Site offers the highest and best use of the site in terms of architectural value, shared-use benefit, and planned land use. Without this partnership, the standalone facility lacks the shared-use benefit. The Yorktown and Fred Richards's sites offer appealing opportunities for shared-use, sustainability, and planned land use. The Median Site has the least benefit in terms of architectural value, sustainability, and shared-use.

Fred Richards and the integrated Southdale sites are the most favorable options when considering initial construction and construction staging and sequencing, and future maintenance. The Yorktown Site has disadvantages when considering the extensive utility relocations required before constructing the water treatment plant. Integrating the Southdale Site with a public/private partnership expands the possibilities and cooperation for the site related to access and initial construction.

Finally, additional considerations related to distribution system operation and raw and finished water transmission piping result in the Southdale and Median sites being the most favorable. The Fred Richards Site is the least favorable due to the extensive raw and finished water transmission pipelines added as part of this project to get Well No. 5 and No. 18 to the facility and finished water out into the distribution system.

In summary, the integrated Southdale Site is the most favorable in terms of non-financial site accommodations. Based on multiple conversations with Simon Properties, the likelihood of this public/private partnership is low. With this in mind, the standalone Southdale Site is the second most favorable alternative, with accommodations limitations related to shared-use benefit and constructability.

13.5.4 Alternative Financial Evaluation

The Project Team developed eight (8) base facility options for the four (4) available site alternatives. These eight (8) options present conceptual design of base facilities that adequately accomplish the treatment goals of WTP No. 5. Based on the facility construction and facility integration costs associated with the evaluated alternatives, the Project Team estimates a cost between $10.56M and $16.41M (2017 dollars). Table 13.5 summarizes the total construction costs of each of the eight (8) options evaluated.

Table 13.5 Summary of Opinion of Total Construction Costs for WTP No. 5

Site	Option	Facility Construction	Facility Integration	Contingencies (15%)	Engineering & Construction Phases (15%)	Total Construction Cost
Southdale Site	Option 1A	$ 8,608,000	$ 400,000	$ 1,351,000	$ 1,554,000	**$ 11,913,000**
	Option 1B	$ 8,728,000	$ 400,000	$ 1,369,000	$ 1,575,000	**$ 12,072,000**
	Option 1C	$ 7,591,000	$ 400,000	$ 1,199,000	$ 1,370,000	**$ 10,560,000**
Yorktown Site	Option 2A	$ 7,941,000	$ 3,605,000	$ 1,732,000	$ 1,992,000	**$ 15,268,000**
	Option 2B	$ 8,060,000	$ 3,605,000	$ 1,750,000	$ 2,013,000	**$ 15,428,000**
Median Site	Option 3A	$ 8,417,000	$ 1,370,000	$ 1,468,000	$ 1,689,000	**$ 12,943,000**
Fred Richards Site	Option 4A	$ 8,057,000	$ 4,200,000	$ 1,839,000	$ 2,115,000	**$ 16,209,000**
	Option 4B	$ 8,211,000	$ 4,200,000	$ 1,862,000	$ 2,141,000	**$ 16,414,000**

The integration of the facility into the City of Edina's existing distribution system is the largest differentiator in the alternative selection. With preliminary level optimization of the facility at the Southdale Site, facility construction cost reductions resulted. If the Project Team applied a similar treatment technology optimization approach to the Yorktown and Fred Richards sites, facility construction costs may decrease. The extensive facility integration costs associated with these sites outweigh the facility construction cost savings, making these sites remain the least cost effective.

13.5.5 Selection of Preferred Alternative

Option 1C of the Southdale Site meets all desired treatment objectives and goals set by the Project Team for WTP No. 5. Distribution system modeling expects selection of this site produces minimal increases in pressure in the system and actually helps with the currently lagging Southdale Tower operation.

The site may present some challenges with constructability due to small construction extents and lack of staging or stockpiling area. The site also lacks a shared-use benefit due to the denial of a public/private partnership with Simon Properties after multiple attempts to date. The City has the option of building a more robust foundation for WTP No. 5, providing opportunities for commercial or retail space above the facility in the future. Preliminary estimation of this premium cost is $500,000.

Option 1C offers the best use of the City's financial resources while including both $1.18M for contingencies and $1.36M for engineering, administrative, and legal fees associated with the project.

13.6 Recommendation for Implementation

The City of Edina should consider the following items for preparing for implementation of the planned WTP No. 5:

- The City should proceed, generally with the design and implementation planning for Option 1C as identified in this WTP Preliminary Design Report.
- The existing water supply wells will need minor improvements in association with the planned WTP project. Replacement/refurbishment of existing values, improved metering capabilities, and pump improvements may all be required to compliment the planned WTP. The City also has the option of reconstruction Well No. 5 as a submersible pump in association with selection of the Southdale Site as the facility location.
- The preliminary design phase should consider investigation of chlorine alternatives, including gaseous chlorine versus onsite hypochlorite generation (OSHG). OSHG would provide an inherently safer chlorine alternative for the selected site alternative, but increases operations and maintenance due to the systems complexity. This should include site visits and discussions with other facilities that have an OSHG system.
- The City should continue making plans for future Well No. 21 location and schedule for construction. The base facility currently assumes initial construction of the plant designed at the full 3,000 gpm capacity.

This WTP Preliminary Design Report, which is the result of the cumulative efforts put forth by the City staff and the Project Team over the past several months, not only to satisfies various planning objectives, but also fosters a dynamic planning process. Throughout the planning period and project implementation process, the Edina should expect many uncertainties and changes, which they can best manage the impacts of these changes through the continuation of the proactive planning process between City staff and the Project Team.