Design of the 2026 91st Avenue Regional Advanced Water Purification Facility

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ENGINEERS

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CENE 486C

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COPPERHEAD
ENGINEERS

Letter of Transmittal

To: Dr. Jeffrey Heiderscheidt

From: Nick Dawson, Moses Marsico, Leyla Still, Samuel Thompson

Date: 05/05/2026

Re: CENE 486C - Final Design Report

To Whom It May Concern,

This Design Report presents the proposed design for the 91st Avenue Advanced Water Purification Facility, to be located in Tolleson, Arizona. The facility will receive effluent from the existing 91st Avenue Wastewater Treatment Facility after it has gone through the Tres Rios Flow Regulating Wetlands.

The treatment process will begin with bar screening, followed by coagulation and flocculation using a high-rate tube settler. The process will then incorporate three stages of physical separation. Subsequently, the water will undergo an advanced oxidation process and ultraviolet disinfection, followed by final chlorination prior to distribution into the Arizona drinking water system. Brine generated during treatment will be recovered and directed to a dedicated brine management system within the facility.

Excluding labor costs, the estimated capital cost to construct the complete treatment train and associated hydraulic infrastructure is approximately $413,000,000. When accounting for labor and additional project variables not fully defined at this stage, the total project cost is anticipated to exceed $1 billion.

Sincerely,

Copperhead Engineering

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Abbreviations

Advanced Oxidation Processes	AOP
Advanced Water Purification Facility	AWPF
Arizona Administrative Code	A.A.C.
Arizona Water Association	AZ Water
City of Phoenix	COP
Disinfection By-Product	DBP
Ductile Iron Pipe	DIP
Equalization	EQ
Granular Activated Carbon	GAC
Gallons per Minute	GPM
High Pressure Reverse Osmosis	HPRO
Microfiltration	MF
Million Gallons Per Day	MGD
Prestressed Concrete Cylinder Pipe	PCCP
Polydiallyldimethylammonium chloride	PolyDADMAC
Reverse Osmosis	RO

Student Design Competition	SDC
Total Dynamic Head	TDH
Total Dissolved Solids	TDS
Total Suspended Solids	TSS
Tres Rios Flow Regulating Wetlands	TRFRW
Ultrafiltration	UF
Ultraviolet	UV
Variable Frequency Drive	VFD
Water Environment Federation	WEF
Wastewater Treatment Plant	WWTP

I. Abstract

Water scarcity is a growing concern around the United States and world. In Arizona, the demand for water continues to grow as the water sources are being used faster than they are being replenished. To combat this, Arizona has turned to Advanced Water Purification as an alternative source of water.

This project is to design the 91st Avenue Advanced Water Purification Facility, which will be the first direct potable reuse facility in Arizona. This is a 30 million gallon per day facility created for the City of Phoenix which will increase the potable water supply while reducing the demand from surface and ground water sources which are limited. This project will include a full design of this facility including but not limited to a physical separation process, an advanced oxidation process, and an ultraviolet system with a dose of at least 300 mJ/cm^2. This project will cost approximately 412 million dollars. The site will be brought to full operation through dry commissioning, wet commissioning, and regulatory approval phasing where the daily operation will be run by a staff of around 25-50 members. This project is estimated to be completed in 2032.

II. Summary of Project Team

Copperhead Engineering is a multidisciplinary team composed of four senior students from the Steve Sanghi College of Engineering at Northern Arizona University. The team brings together complementary expertise, with two students specializing in Environmental Engineering and two in Civil Engineering. This balanced composition allowed the group to approach the project from both process design and infrastructure perspectives, ensuring a well-rounded and practical solution.

The Environmental Engineering students were primarily responsible for the selection and design of the treatment train. Their work involved evaluating various treatment technologies, considering regulatory requirements, and optimizing processes to achieve efficient and sustainable water treatment. This included assessing factors such as contaminant removal efficiency, operational reliability, environmental impact, and cost-effectiveness.

Meanwhile, the Civil Engineering students focused on the hydraulic analysis and physical layout of the facility. Their responsibilities included designing flow paths, sizing pipes and channels, ensuring proper system hydraulics, and developing a site layout that maximized efficiency while adhering to spatial and logistical constraints. They also accounted for constructability, site accessibility, and long-term operational considerations.

III. Acknowledgments

The students could not have completed this competition without the help of Technical Advisor and Grading Instructor Dr. Jeffrey Heiderscheidt. Dr. Heiderscheit held consistent meetings offering a space for students to ask clarifying questions, opinions on research completed and report redlines. The student also communicated with SDC chair sponsor Adias Fostino and AZ Water Representative Kt Stowers about competition deadlines and site visit logistics.

1. Project Description

1.1 Design Problem

The purpose of this project is to develop a conceptual design for an Advanced Water Purification Facility (AWPF) capable of producing and distributing up to 30 million gallons per day (MGD) of purified water using treated effluent from the 91st Avenue Wastewater Treatment Plant in Tolleson, Arizona. The treated wastewater is collected from the Tres Rios Flow Regulating Wetlands (TRFRW) at Outfall 005, located southwest of the wetlands. The proposed facility is intended to support long-term regional water reliability through advanced water reuse while meeting stringent regulatory, operational, and site-specific constraints.

1.2 Project Background

This project will be in Tolleson Arizona, which is in Southwest Phoenix. The facility is located directly south of Interstate 10 and west of Arizona Loop 202; Tolleson Arizona has a population of a little over 7000 people. Figure 1-1 shows the Site map of the surrounding area near and around the project site including parcels owned by the city in red:

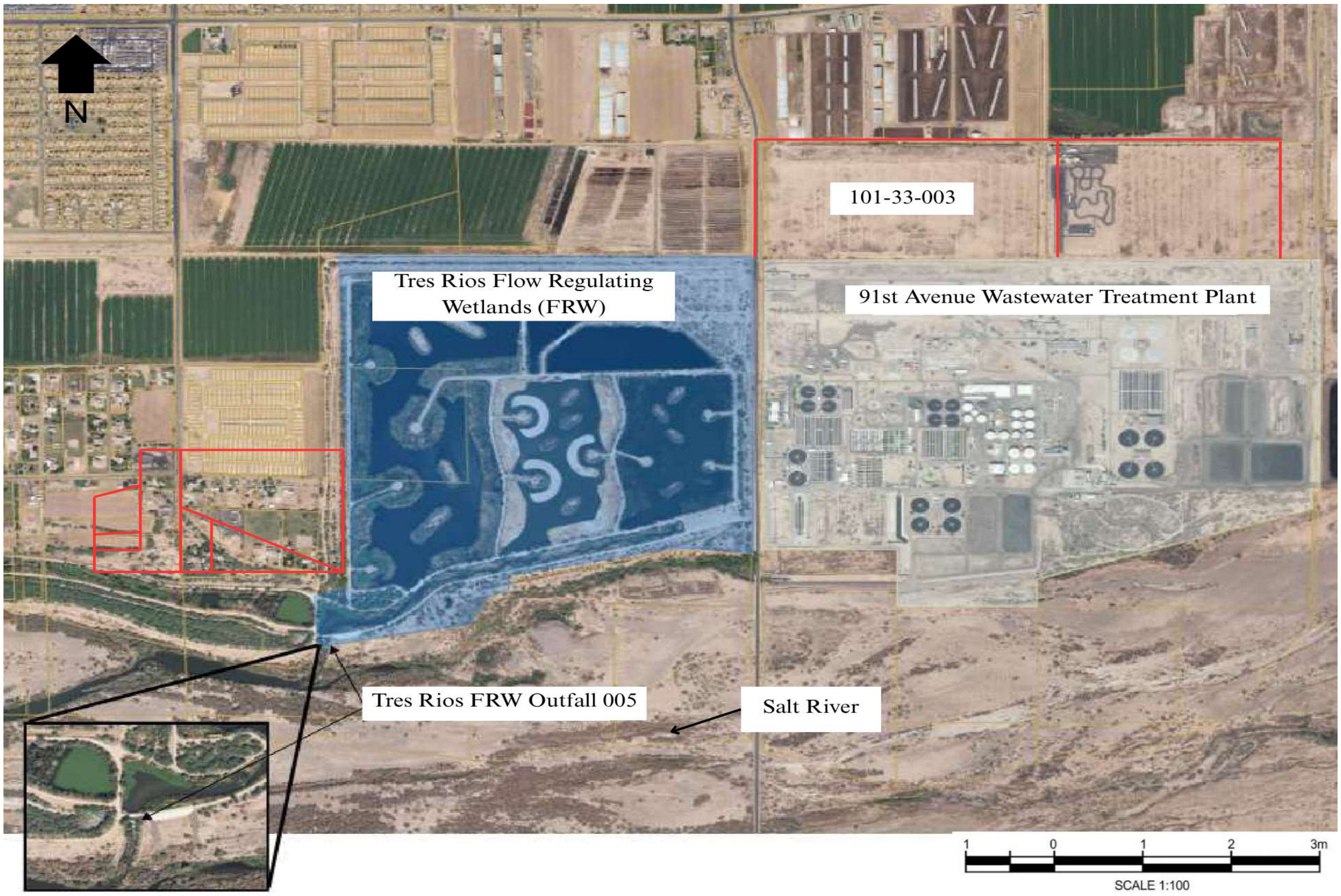

Figure 1-1. Existing Site Map

The effluent from the TRFRW will be collected from Outfall 005, the quality of water is considered average but there are multiple constituents that must be controlled throughout the water treatment process. A table of the existing water quality data can be found in Appendix A. Most importantly, the average Total Dissolved Solids (TDS) is 1142 mg/L with a 95th Percentile of 1335 mg/L, needing to be reduced to less than 750 mg/L due to SDC Prompt guidelines. Additionally, the processes used in the 91st Avenue Wastewater Treatment Plant can be found in the flow diagram below.

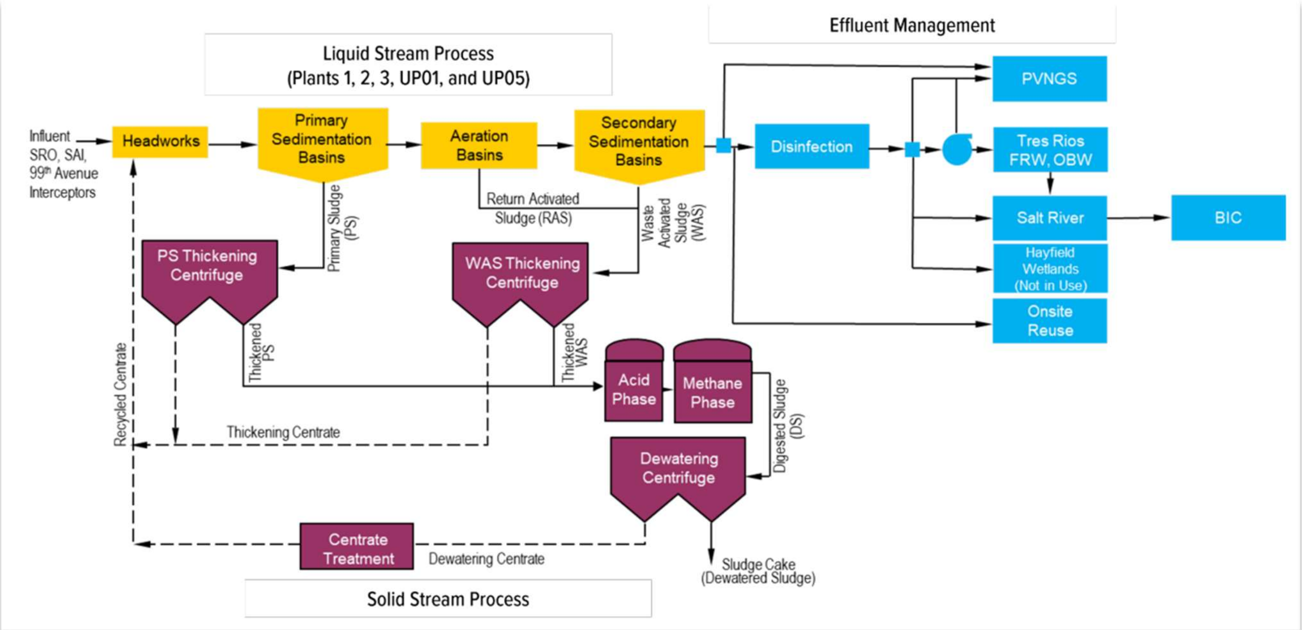

Figure 1-2. 91st Avenue Wastewater Treatment Plant Process Diagram

1.3 Constraints and Limitations

The proposed project location must be located on parcels owned by the City of Phoenix (COP) around the pre-existing WWTP. The quality of water must adhere to the Safe Drinking Water Act (SDWA) and maximum contamination levels (MCL) for the surrounding area. Along with state regulations such as the Arizona Department of Environmental Quality (ADEQ). A list of these requirements can be found in Appendix A.

Additionally, there are constraints set by the Water Environment Federation (WEF) Student Design Competition (SDC) which the facility is required to adhere to. These include pathogen removal and inactivation using a minimum of 3 treatment barriers, and pathogen reduction using standard log removal requirements of 13-log virus and 10-log protozoa (Cryptosporidium and Giardia) per Arizona Administrative Code (A.A.C.) R18-9-E828, TDS reduction to achieve a salinity less than 750 milligrams per liter (mg/L), and adequate brine management to reduce waste from this facility. Additionally, this competition requires that the facility includes ultraviolet (UV) disinfection with a dose of at least 300 mJ per cm^2, an advanced oxidation process (AOP), and a physical separation process. All applicable state and federal regulations can be found in Appendix B.

1.4 Objectives

The objective of this project is to evaluate, select, and design an advanced water purification facility (AWPF) that can reliably provide safe drinking water while fitting within an existing parcel of city-owned land adjacent to the 91st Avenue Wastewater Treatment Plant (WWTP). Because the influent originates at a lower elevation, the design must also address hydraulic grade line limitations, pumping requirements, and energy demands associated with lifting and conveying flow through membrane and advanced oxidation processes (AOPs). In addition to treatment performance, the project emphasizes brine management feasibility, lifecycle cost effectiveness, and constructability. The AWPF must remain practical for municipal implementation, accounting for budget limitations, operational complexity, and the need for phased construction that does not disrupt the ongoing operation of the 91st Avenue WWTP. This project aims to identify an advanced treatment configuration that balances regulatory compliance, hydraulic feasibility, land use efficiency, cost, operational flexibility, and community considerations, while consistently meeting the required 30 MGD treatment and distribution capacity.

2. Treatment Train Evaluation Process

2.1 Parcel Selection Process

The proposed AWPF is required to be located on COP owned land per the SDC requirements and since that is how we will be owning and operating the facility. As seen in figure 1-1, there are about 9 parcels that are located near the facility along with being owned by the city. The land required for a facility of this nature will require many acres, at least 50, including room to expand the facility, if need be, making this one of the more important factors in the parcel decision matrix. The AWPF also needed to be at a distance from the salt river just due to discharge issues, soil stability and other interferences. With these 2 main criteria, the 6 parcels located in the bottom left corner were eliminated due to size and location. The final 3 parcels went through a decision matrix including size, location, plot shape and quality of land (grade, soil). These categories were decided based on competition requirements and general parcel needs during construction. After this process parcel 101-33-003 was chosen at 80 acres, fairly flat grade and located directly above WWTP.

2.2 Treatment Train Selection Process

The initial phase of the treatment train selection process involved identifying the individual treatment steps that comprise an AWPF. To inform this effort, the team conducted a review of existing and proposed facilities with characteristics comparable to the proposed project. Based on this research, a preliminary process flow diagram was developed to represent the proposed treatment train, including bar screening, coagulation/flocculation, three physical separation processes, advanced oxidation, UV disinfection, final chlorination, and brine management.

Decision matrices were developed for bar screening, coagulation/flocculation, the three physical separation processes, advanced oxidation, and brine management. These matrices were informed by a range of sources, including technical publications, textbooks, federal regulations, and other relevant references available to the project team.

Each matrix evaluated alternatives across five criteria: land use, operational flexibility, and operational/capital cost (as required by the 2026 Arizona Water SDC), as well as environmental impact and energy efficiency, which were selected by the project team as additional considerations. The required criteria were each assigned a weight of 25%, while the additional criteria were each weighted at 12.5%. Alternatives were scored on a scale from 1 to 3, where 3 indicated the most favorable performance and 1 indicated the least favorable. Scores were determined through comparative assessment of each process within the respective criteria.

Each decision matrix included three to four candidate processes, from which one or two were selected. The selected processes were then combined to develop three alternative treatment train configurations.

2.2.1 Bar Screening

The purpose of bar screening is to remove large debris and particles from the incoming water before it is pumped to the start of the treatment facility. By intercepting materials such as sticks, plastics, and other coarse solids, the bar screen plays a critical protective role in preventing clogging, excessive wear, or mechanical damage to downstream equipment—especially pumps. This not only improves operational efficiency but also reduces maintenance requirements and prolongs the lifespan of the facility's components. To determine the most suitable option, a decision matrix was developed to compare these alternatives based on criteria such as efficiency, maintenance requirements, cost, and reliability. Based on this evaluation, the vertical bar screen was selected as the preferred option due to its overall effectiveness, simplicity, and compatibility with the facility's operational needs. Below is the decision matrix used in the bar screen selection process.

Table 2-1: Bar Screening Decision Matrix

Bar Screening						
Alternative	**Environmental impact (0.125)**	**Energy Efficiency (0.125)**	**Operational and Capital Cost (0.25)**	**Operational Flexibility (0.25)**	**Land Use (0.25)**	**Total Score**
Manual Screen	1	2	3	1	1	1.625
Vertical Screen	2	2	2	3	2	2.25
Curved Screen	2	2	2	1	3	2

2.2.2 Coagulation and Flocculation

The purpose of coagulation and flocculation is to cause the Total Suspended Solids(TSS) to stick together (coagulation) then settle out of the water (flocculation). The first part of this process, which was considered, was coagulation. The different types of coagulants which were compared included Alum, Ferric Chloride, and Polydiallyldimethylammonium Chloride (PolyDADMAC). The flocculants are polymers and the different types being considered are Cationic, anionic, and Nonionic

Alum is aluminum sulfate; a compound frequently used in the water treatment process for coagulation. This compound can be consistently used with a large amount of water quality parameter fluctuations. Another coagulant considered was Ferric Chloride. This compound is also frequently used in water treatment as a coagulant. The final coagulant considered for this process was PolyDADMAC. PolyDADMAC is a cationic polyelectrode which is used for drinking water treatment (Richard, 2024). This compound is not biodegradable and produces disinfection by products (DBPs). Alum and Ferric Chloride are being considered as alternatives. Below is the decision matrix for the coagulants options:

Table 2-2: Coagulation Decision Matrix

Coagulation						
Alternative	**Environmental impact (0.125)**	**Energy Efficiency (0.125)**	**Operational and Capital Cost (0.25)**	**Operational Flexibility (0.25)**	**Land Use (0.25)**	**Total Score**
Alum	3	2	2	3	2	2.375
Ferric Chloride	2	2	3	3	2	2.5
PolyDADMAC	1	2	2	2	2	1.875

The polymers only differ in charge and in their ability to polymerize specific types of monomers present in the system. Cationic polymers are more effective at polymerizing electron-donating species, while anionic polymers are better suited for electron-withdrawing compounds. In contrast, nonionic polymers are more versatile and can interact with a wider range of particle types regardless of their charge characteristics. This broader applicability makes nonionic polymers particularly effective in systems with variable water quality, where the composition of suspended and dissolved materials may fluctuate. Additionally, nonionic polymers tend to produce more stable flocs under a range of conditions, improving overall treatment performance. For these reasons, nonionic polymers were chosen for the treatment train. Below is the decision matrix used to support this selection.

Table 2-3: Flocculation Decision Matrix

Flocculation						
Alternative	**Environmental impact (0.125)**	**Energy Efficiency (0.125)**	**Operational and Capital Cost (0.25)**	**Operational Flexibility (0.25)**	**Land Use (0.25)**	**Total Score**
Cationic Polymers	2	2	2	2	2	2
Anionic Polymers	2	2	2	2	2	2
Nonionic Polymers	3	2	2	3	2	2.375

After coagulation and flocculation, a sedimentation basin is required to remove the total suspended solids (TSS) formed during these processes. This basin allows the aggregated particles (floc) to settle out of the water, producing a clarified effluent for downstream treatment. Due to their widespread use and high efficiency with relatively limited land requirements, a high-rate sedimentation basin with tube settlers was selected. Tube settlers increase the effective settling area, allowing for improved particle removal within a smaller footprint compared to conventional basins. A decision matrix was not necessary for this selection, as the high-rate settler met all design criteria for this stage, including efficiency, space constraints, and operational reliability.

2.2.3 Physical Separation

The first group of physical separation processes included slow sand filters, rapid media filters, and cartridge filters. This group of physical separation focuses on removing Total Suspended Solids (TSS) from the water.

Slow sand, rapid media, and cartridge filters differ mainly in how they physically remove particles and are operated. Slow sand filters use a biological layer that forms on top of the sand to naturally treat water, and they operate at low flow rates without the need for frequent maintenance. Rapid media filters push water through sand or other media at higher rates and require regular backwashing to clean the filter. Cartridge filters pass water through replaceable filter cartridges that trap particles, and instead of being cleaned, the cartridges are periodically replaced. Overall, slow sand relies on biological treatment, rapid media relies on mechanical filtration with cleaning, and cartridge filters rely on disposable media. With this assessment, a rapid media filter was decided on as the best physical separation process for all the alternatives.

Table 2-4: Physical Separation Decision Matrix

Physical Separation 1						
Alternative	**Environmental impact (0.125)**	**Energy Efficiency (0.125)**	**Operational and Capital Cost (0.25)**	**Operational Flexibility (0.25)**	**Land Use (0.25)**	**Total Score**
Slow Sand Filter	3	2	2	1	1	1.625
Rapid Media Filter	2	2	2	3	2	2.25
Cartridge filter	1	2	1	3	3	2.125

For the second physical separation process microfiltration (MF), ultrafiltration (UF), and adsorption (GAC/PAC) are considered. MF removes larger total suspended solids (TSS) and some bacteria, while UF, with its smaller pore size, can remove smaller particles, including viruses and finer colloids. GAC/PAC can reduce the dissolved particles in the system such as organics and metals. Due to the quality of water being produced, UF and GAC/PAC were the best options across the treatment alternatives.

Table 2-5: Physical Separation 2 Decision Matrix

Physical Separation 2						
Alternative	**Environmental impact (0.125)**	**Energy Efficiency (0.125)**	**Operational and Capital Cost (0.25)**	**Operational Flexibility (0.25)**	**Land Use (0.25)**	**Total Score**
Microfiltration	3	1	2	2	3	2.25
Ultrafiltration	3	2	2	2	3	2.375
Adsorption (GAC/PAC)	3	1	2	2	2	2

For the final physical separation process, the different treatment processes have different results. Ion exchange is selected as the preferred method for dissolved ion (TDS) removal due to its balance of performance, cost, and efficiency. While reverse osmosis (RO) provides the highest level of removal—including monovalent ions, dissolved organics, and pathogens—it requires high operating pressures and produces a significant volume of brine waste, making it the most energy-intensive and expensive option. Nanofiltration (NF) operates at lower pressures and is more energy-efficient than RO but primarily removes divalent ions and larger organics, resulting in lower overall treatment performance for TDS removal. Below is the decision matrix showing Ion Exchange and Nanofiltration were the best option for the treatment train.

Table 2-6: Physical Separation 3 Decision Matrix

Physical Separation 3						
Alternative	**Environmental impact (0.125)**	**Energy Efficiency (0.125)**	**Operational and Capital Cost (0.25)**	**Operational Flexibility (0.25)**	**Land Use (0.25)**	**Total Score**
Reverse Osmosis	1	1	1	3	2	1.75
Nanofiltration	3	3	2	1	2	2
Membrane Bioreactor	2	1	1	2	1	1.375
Ion Exchange	2	2	2	3	3	2.5

2.2.4 Advanced Oxidation Processes

Advanced Oxidation Processes (AOPs) are treatment methods that use highly reactive oxidizing agents, such as hydroxyl radicals (•OH), to break down organic contaminants into simpler compounds like water and carbon dioxide (Kujawska, Kielkowska, Atisha, Yanful, & Kujawski, 2022). These radicals are extremely effective at degrading a wide range of persistent organic pollutants that are difficult to remove through conventional treatment methods. The design of the 91st Avenue AWPF requires the implementation of an AOP as part of the treatment process. For these treatment trains, several AOP combinations were considered, involving chlorine, ozone, hydrogen peroxide, and UV radiation. It is important to note that neither UV nor hydrogen peroxide alone qualifies as an AOP, as they cannot independently generate hydroxyl radicals. Instead, these processes must be used in combination—such as UV/H_2O_2 or ozone/H_2O_2—to effectively produce •OH radicals. For this stage, the hydrogen peroxide–based AOP was selected as the most suitable option. The following decision matrix was used to evaluate and compare the alternatives.

Table 2-7: Advanced Oxidation Process Decision Matrix

AOP						
Alternative	**Environmental impact (0.125)**	**Energy Efficiency (0.125)**	**Operational and Capital Cost (0.25)**	**Operational Flexibility (0.25)**	**Land Use (0.25)**	**Total Score**
Ozone/UV	3	1	1	3	2	2
Chlorination/UV	2	3	2	3	3	2.625
H2O2/UV	3	3	2	3	3	2.75

2.2.5 Final Chlorination

This is the final stage of treatment the water undergoes before being distributed into the drinking water supply. It is a critical step for inactivating remaining pathogens and preventing microbial regrowth throughout the distribution system. This process ensures a residual chlorine concentration of 0.2 mg/L is maintained after a contact time of 4 hours, meeting the requirements of the Surface Water Treatment Rule established by AZDEQ and helping to safeguard public health.

2.2.6 Brine Management

To reduce the amount of waste from the facility and to maximize the amount of water produced, the facility will need to concentrate its brine—a process commonly referred to as brine management. The different brine management processes considered for this facility include brine concentration/crystallization, membrane distillation, and high-pressure reverse osmosis.

The first brine management technique evaluated was brine concentration/crystallization. These techniques are frequently used in large-scale industrial applications, because of their ability to achieve near-zero liquid discharge. In this process, water is evaporated from the brine, leaving behind solid salts as the final waste product.

Membrane distillation was also considered as an alternative brine management approach. This is a thermally driven separation process in which a hydrophobic membrane allows water vapor to pass through while rejecting dissolved salts and other contaminants. The process typically operates at lower temperatures and pressures compared to conventional thermal evaporation systems, making it potentially more energy-efficient when low-grade or waste heat is available. Membrane distillation can achieve high levels of brine concentration and is less sensitive to high salinity levels than traditional membrane processes.

The third technique considered was high-pressure reverse osmosis (HPRO). This process is an extension of conventional reverse osmosis but operates at significantly higher pressures to treat highly concentrated brines that standard systems cannot handle. HPRO can recover additional water from brine streams, thereby reducing overall waste volume and improving system efficiency. Compared to thermal methods, it generally has lower energy consumption, but the required pressures lead to increased mechanical stress on system components, necessitating specialized equipment and higher maintenance requirements.

Table 2-8: Brine Management Decision Matrix

Brine Management						
Alternative	**Environmental impact (0.125)**	**Energy Efficiency (0.125)**	**Operational and Capital Cost (0.25)**	**Operational Flexibility (0.25)**	**Land Use (0.25)**	**Total Score**
Concentration and crystallization	3	2	3	2	1	2.125
Membrane Distillation	1	2	1	2	2	1.625
High-Pressure Reverse Osmosis	3	2	1	1	1	1.375

2.3 Treatment Train Alternatives

The next step to designing the 91st Avenue Advanced Water Purification Facility was to select from three different treatment train alternatives. These different treatment train alternatives are different possible processes which would be used at the advanced water purification facility (AWPF). Each of these treatments of trains must meet requirements set by the competition as well as local and federal codes and regulations. Each of these trains included bar screening, coagulation and flocculation, physical treatments, AOPs, UV disinfection, and final chlorination. The final treatment train process is given in Table 2.8.

Table 2-9: Treatment Train Alternatives

Process	Treatment train 1	Treatment train 2	Treatment train 3
Bar Screening	Vertical Screen	Vertical Screen	Vertical Screen
Coagulant	Alum	Ferric Chloride	Ferric Chloride
Flocculant	Nonionic Polymers	Nonionic Polymers	Nonionic Polymers
Physical Treatment 1	Rapid Media Filter	Rapid Media Filter	Rapid Media Filter
Physical Treatment 2	Carbon Adsorption	Ultrafiltration	Ultrafiltration
Physical Treatment 3	Nanofiltration	Ion Exchange	Ion Exchange
Advanced Oxidation Process	Hydrogen Peroxide	Ozone	Hydrogen Peroxide
Brine Management	Crystallization	Crystallization	Crystallization
UV			
Final Chlorination			

Each process in the decision matrix had a purpose when purifying the effluent water, depending on the chemical or process a certain amount of TDS, TSS, pathogens or organics are removed. For coagulation, the third physical separation and brine management there were 2 options that could be used from the decision matrix. The option that scored higher out of the 2 was put into 2 alternatives while the other option was only put into one. The categories stayed the same and the values from each treatment process were combined for each alternative. Creating a possible score out of 24, since there were 8 processes with a possible score of 3 each. With this, treatment train 3 scored the highest with a score of 18.875 out of 24 and was chosen as the final treatment train. The scores for all three treatment alternatives can be found in Table 2-9.

Table 2-10: Treatment Train Decision Matrix

Treatment Train Scores						
Alternative	**Environmental Impact (0.125)**	**Energy Efficiency (0.125)**	**Operational and Capital Cost (0.25)**	**Operational Flexibility (0.25)**	**Land Use (0.25)**	**Total Score**
Treatment Train 1	22	18	17	21	16	18.5
Treatment Train 2	20	16	17	21	17	18.25
Treatment train 3	20	17	17	22	18	18.875

3. Design of Selected Treatment Train

3.1 Final Treatment Train

After the completion of multiple treatment train decision matrices, Alternative 3 was chosen as the best treatment train for removing TDS and TSS, protection against pathogens and bacteria such as Giardia and Cryptosporidium along with providing a reliable production of 30 MGD. Figure 1-2 shows the final treatment train flow diagram.

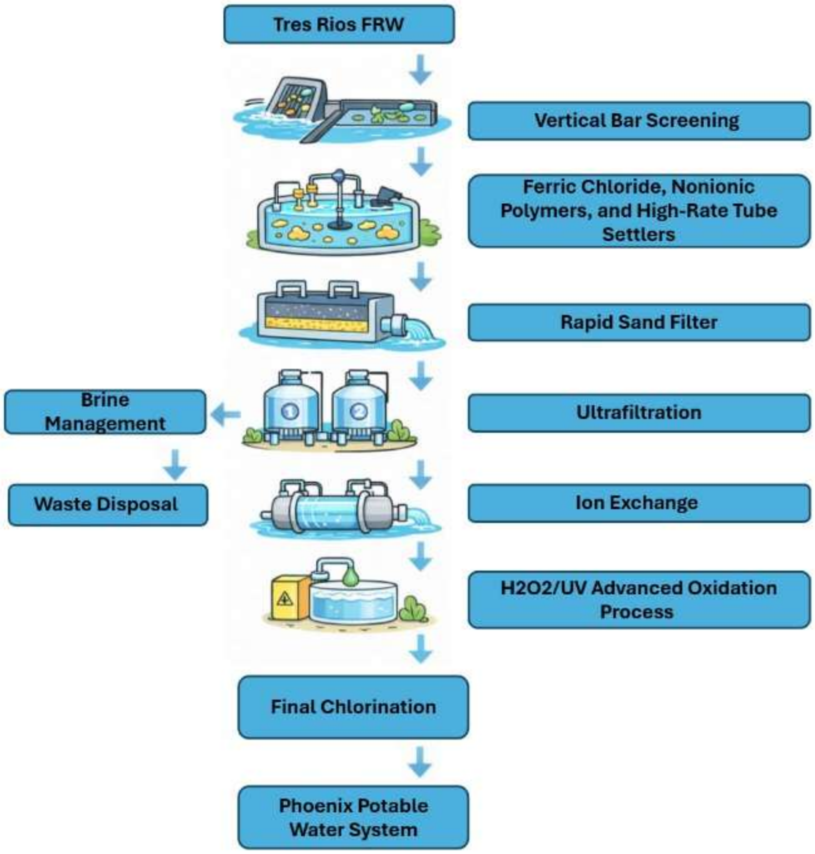

Figure 3-1: Treatment Train Flow Diagram

3.2 Coagulation and Flocculation

The team designed a High-Rate Sedimentation Basin required for Coagulation and Flocculation. The High-Rate Sedimentation Basin Area was calculated using equations from Water and Wastewater Engineering Design Principles and Practice (Davis, 2020). With the incoming flow of 40 MGD, the basin needs to be around 340 m^2. After calculating the size of the basin, the dosing of Ferric Chloride and Nonionic polymers needs to be calculated. The exact dosing cannot be determined without a Jar Test; a Jar Test was not completed due to inability to obtain a sample from the TRFRW.

Currently the TSS sits at 21 mg/L in the 95th percentile, to reduce this the TSS in the system a dose between 2-100 mg/L of Ferric Chloride is required. For this system a dose of 15 mg/L was selected due to the given water quality and the gpm leaving the facility. This value was calculated with a 42% ferric chloride solution in mind (Australian Drinking Water Guidelines , 2011). With this a daily mass rang of 668 lbs/day to 33,381 lbs/day was calculated with an estimated daily mass of about 5,000 lbs/day. For flocculation a typical nonionic polymer range is anywhere between 0.05-0.5 g/m^3, the team decided on 0.1 g/m^3 of nonionic polymers per day. With this a low dose of 16.7 lbs/day and a high dose of 167 lbs/day is calculated with an estimated dose of 33.4 lbs/day is needed for the system. A typical nonionic polymer range is anywhere between 0.05-0.5 g/m^3, the team decided on 0.1 g/m^3 of nonionic polymers per day. With this a low dose of 16.7 lbs/day and a high dose of 167 lbs/day is calculated with an estimated dose of 33.4 lbs/day is needed for the system.

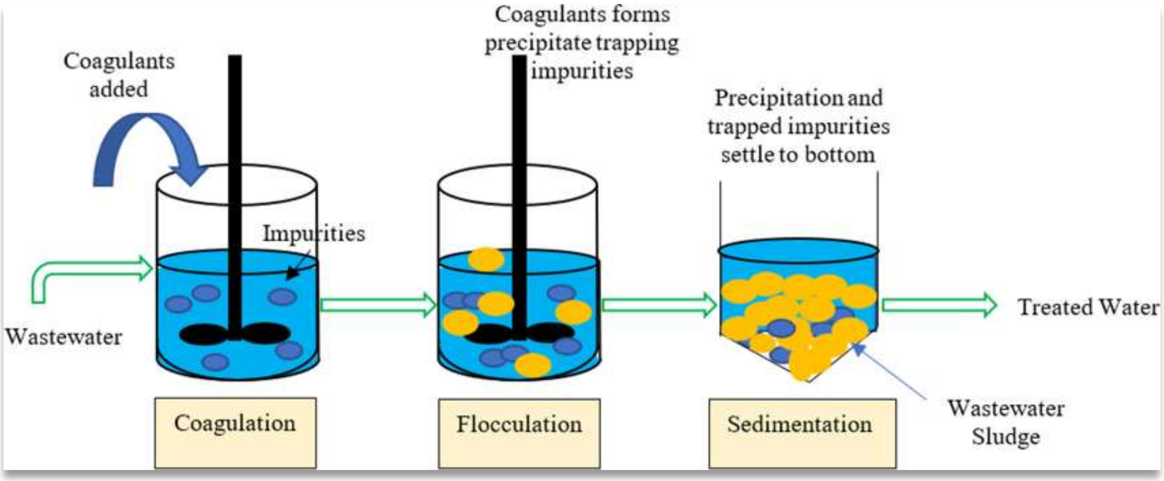

Figure 3-2: Coagulation and Flocculation Process

3.3 Filtration

The Rapid Media filter used a combination of sand and GAC, due to the filtration properties held by both types of media. The GAC is meant to remove extra organics that may not typically be removed during this stage. The basin required for a filter of this type was calculated using equations from Water and Wastewater Engineering Design Principles and Practice (Davis, 2020). The tank surface area will need to be 650 m^2, using 9 beds for this process to be able to process the 40 MGD entering. With this the filtration rate will be 293 m/d.

3.4 Ultrafiltration

With a typical pore size range of 0.01 to 0.1 microns, ultrafiltration effectively removes microorganisms such as *Giardia* and *Cryptosporidium*, significantly improving water quality and pathogen removal. The UF-0880-LP UF

Membrane Module from imemflo requires approximately 50 units per rack to accommodate a facility flow of 181,844 m³/d. This number of units per rack is based on manufacturer specifications.

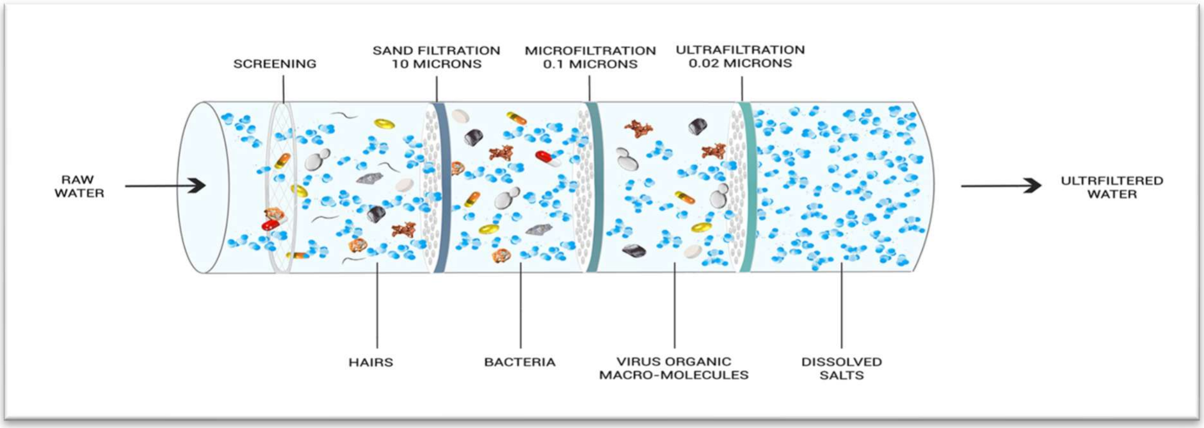

Figure 3-3: Ultrafiltration Diagram

3.5 Ion Exchange

Ion exchange systems require a significant physical footprint, as a typical vessel can treat approximately 1,000 gpm. To meet the facility's required flow rate, a total of 30 vessels are needed, with 28 operating continuously and 2 reserved for redundancy to allow for maintenance and operational flexibility. This configuration ensures consistent treatment performance while minimizing downtime. With this arrangement, the system achieves an approximate retention time of 8 hours, which is sufficient for effective ion exchange processes, and supports a total production capacity of 40 MGD.

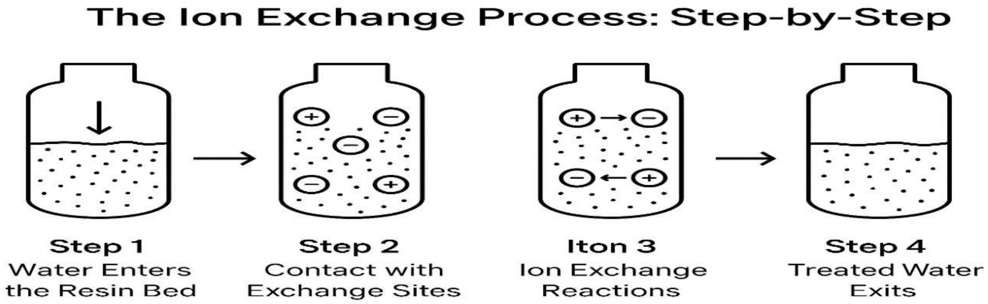

Figure 3-4: Ion Exchange Diagram

3.6 Advanced Oxidation Process and Ultraviolet

The AOP for this design is one that utilized the combination of hydrogen peroxide and UV. For this WAP plant, it is recommended that the TrojanUVFlexAOP system be installed. This system will apply a UV dose of 300 mJ/cm^2 and incorporates hydrogen peroxide injection in the system. Each of these vessels have a flow rate of between 1000 to 2000 GPM (TROJAN technologies , 2026). It is recommended that a total of 13 units at a flow rate of 1750 GPM be installed, one of these units being for redundancy in case of a unit failure.

3.7 Final Chlorination

To ensure that the effluent water meets the required chlorine residual levels set by the ADEQ Surface Water Treatment Rule, chlorine must be added before the water enters the potable water system. This rule states that the system must maintain a chlorine residual of 0.2 mg/L for four hours after leaving the treatment plant (Arizona Department of Environmental Quality , 2025). The recommended chlorine dose for this facility is 0.8 mg/L, however once the water is distributed the concentration bay decreases slightly.

3.8 Brine Management

Crystallization and Concentration offer several advantages, including lower energy consumption compared to more mechanically intensive recovery methods, as well as the ability to produce a higher-purity solid product through controlled concentration and crystallization. For this project, a total of six basins will be required to manage and store the brine generated throughout the treatment process, ensuring adequate capacity for continuous operation.

4. Hydraulic Analysis

The hydraulic analysis for this project includes the analysis of the site layout, parcel selection, pump station and system, pump selection, and pipe design. Any proposed decision matrices for these designs can be found in Appendix D, and any calculations for these designs can be found in Appendix E. Furthermore, this section will go over the actual decision matrices that were used for the hydraulic analysis.

4.1 Site Layout and Parcel Selection Overview

The proposed facility layout utilizes 73 acres out of the 80 acres available on Parcel 101-33-003 for process units and hydraulic infrastructure, while maintaining additional space for operational flexibility and long-term expansion. The hydraulic design presented in this section is directly governed by the proposed site configuration, which dictates flow routing, elevation changes, and pumping requirements. Further site layout information can be found in Appendix F and Appendix G.

The contours in Figure 4-1 show elevation levels on the site that correlate with our topography map in Appendix K. A legend is also provided in the figure showing the location of each treatment process location. These locations will not be at the same elevations or completely level, and this can be seen in the hydraulic profile provided in Figure 4-2 in the pump station and system overview section. The treatment train processes will use a gravity flow system and this can also be seen in the hydraulic profile provided in Figure 4-2.

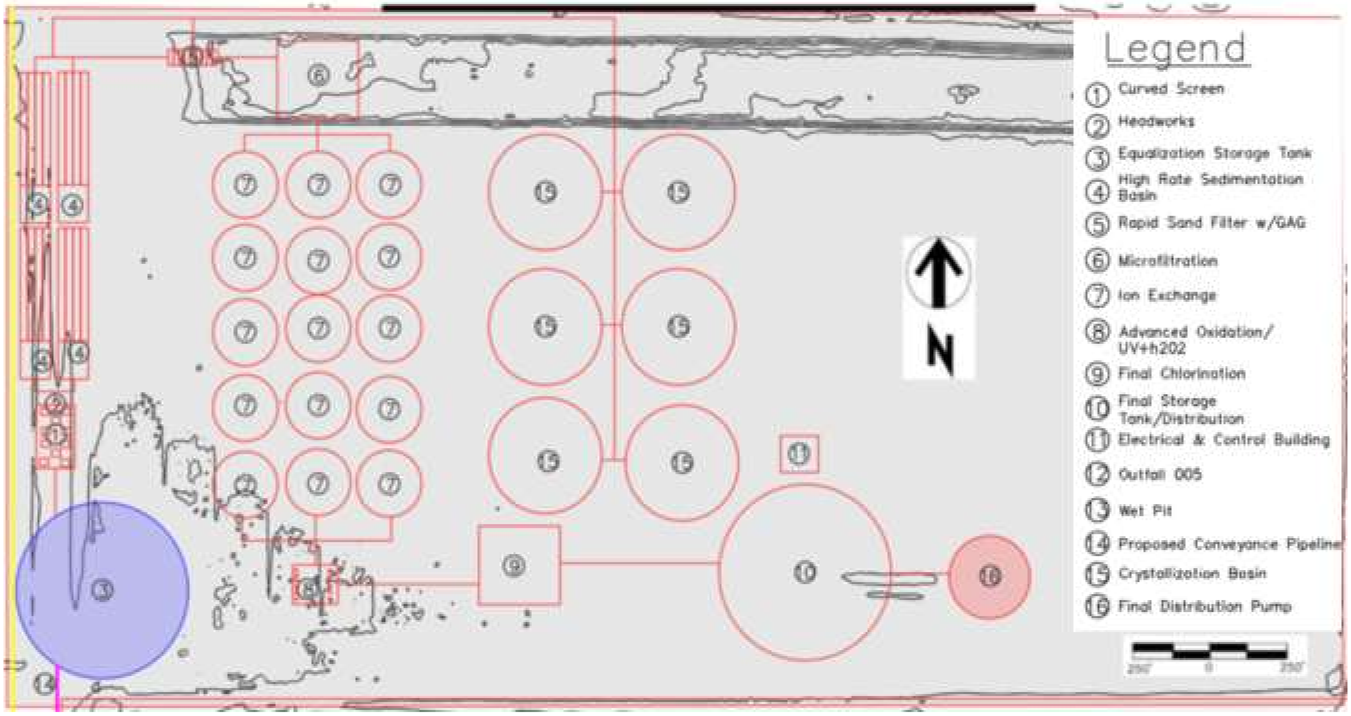

Figure 4-1: Propose Site Layout

The proposed pump station is located at Outfall 005 of the Tres Rios Flow Regulating Wetlands and serves as the hydraulic starting point for conveying flow to the Advanced Water Purification Facility (AWPF). The system is designed to deliver a total flow of 40 MGD, consisting of 30 MGD of finished potable water and 10 MGD allocated for reject and brine streams.

The pump station utilizes a wet pit (open sump) configuration with four vertical turbine pumps operating in parallel and one standby unit (4 duty + 1 standby) to ensure reliability during peak demand and maintenance conditions. Each pump delivers approximately 6,944 gpm and is equipped with a variable frequency drive (VFD) for flow control and energy efficiency (Davis, 2020). Refer to Appendix J for wet well configuration and pump arrangement. Wet well sizing was determined using a weighted decision matrix evaluating footprint, effective volume control, constructability, and operational stability. The 35-ft diameter alternative achieved the highest score and was selected as the optimal design, balancing hydraulic performance and constructability. Flow equalization is provided by an on-site equalization (EQ) tank, which regulates influent variability and maintains stable pump operation. Individual pump discharges connect to a common 42-inch header, equipped with isolation and check valves to allow independent operation and system control (Davis, 2020).

Table 4-1: Wet Well Diameter Decision Matrix

Wet Well Diameter					
Alternative	**Footprint (0.20)**	**Effective Volume Control (0.30)**	**Constructability/Excavation (0.25)**	**Safety/Stability (0.25)**	**Total Score**
25'	5	2	3	3	3.1
30'	4	4	4	4	4
35'	3	5	5	5	4.5
40'	2	5	3	4	3.6

The use of a common 42-inch discharge header ensures efficient conveyance of total system flow while maintaining stable hydraulic conditions. The system operates at a total dynamic head (TDH) of 62.1 ft. Refer to Appendix E for detailed hydraulic calculations. A summary of key pump system design parameters is provided in Table 4-2.

Table 4-2: Pump Station Parameters

Parameter	**Value**
Design Flow	40 MGD
Flow per Pump	6,944 gpm
Number of Pumps	4 working duty + 1 standby
Pump Type	Vertical Turbine
Total Dynamic Head (TDH)	62.1 ft
Static Head	24.61 ft

The conveyance pipeline consists of a 42-inch ductile iron pipe (DIP) extending approximately 10,826.8 ft. DIP was selected over prestressed concrete cylinder pipe (PCCP) based on constructability, flexibility, and compatibility with the pump station configuration, while maintaining adequate structural strength and hydraulic performance. Refer to Appendix L for pipeline alignment and length. (Davis, 2020).

A simplified hydraulic profile was developed to represent system energy conditions from the wet pit through the treatment train to final storage. Gravity-driven flow was assumed for most treatment processes, with a design allowance of 1 ft of head-loss per process unit to represent interconnecting piping and transitions.

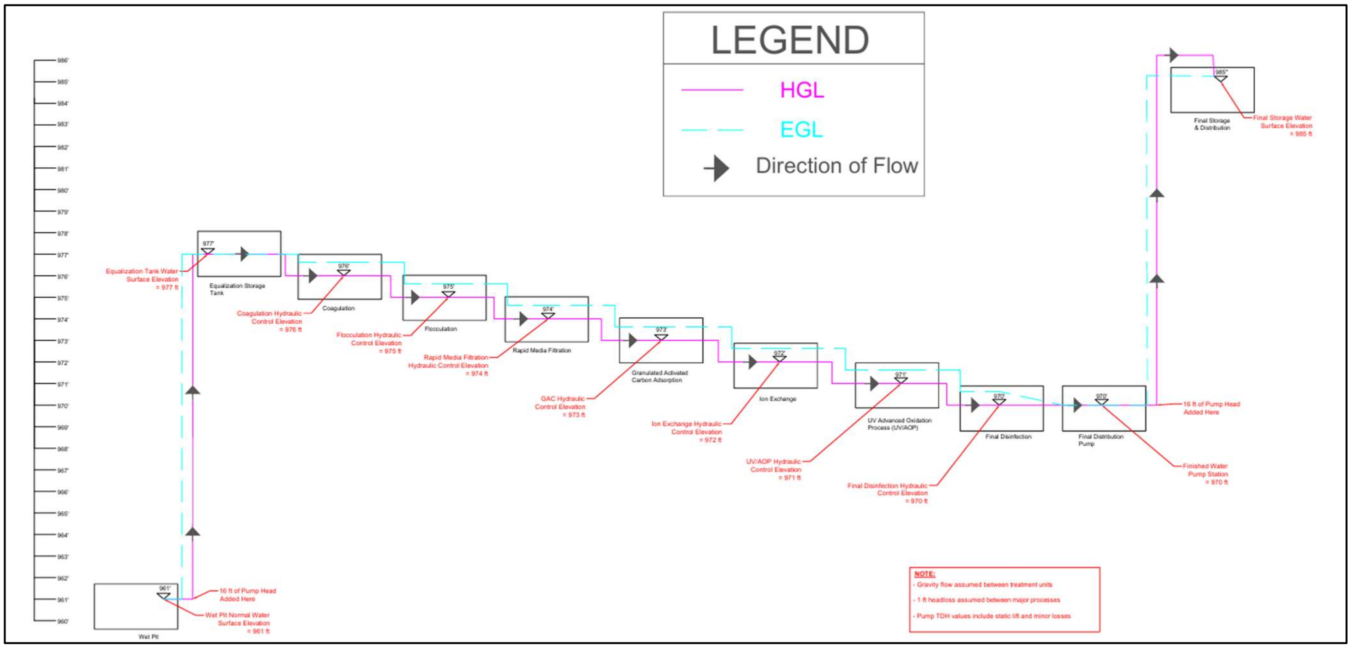

Figure 4-2: Hydraulic Profile

The system curve was developed and overlaid with the manufacturer's pump curve to determine the operating point. The intersection confirms the pumps operate at the best efficiency point. Refer to Appendix L and Appendix K for system and pump curve development. (Davis, 2020) **Section 4.3** will go more into depth on the pump and system curve development.

Table 4-3: Final Simplified Hydraulic Profile

Hydraulic Profile Summary				
Location	**Elevation (ft)**	**HGL (ft)**	**EGL (ft)**	**Description**
Wet Pit (Start)	961	961.00	961.64	System starting point
Pump Discharge (to EQ Tank)	977	977.00	977.64	Initial pump lift (~16 ft added head)
End of Treatment Train	970	970.00	970.64	Gravity-driven flow with 1 ft of head-loss per process
Final Storage & Distribution	985	985.00	985.00	Final pump lift to storage

4.2 Pump Selection

The system curve was developed by calculating total dynamic head (TDH) across a range of flow rates using the hydraulic assumptions and loss equations applied to the conveyance system. Refer to Appendices E and F for detailed calculations and governing equations. The system curve was overlaid on the manufacturer's pump performance curve to determine the operating point. The selected operating point occurs at approximately 6,944 gpm per pump and 62.1 ft TDH, near the pump's best efficiency point (BEP), confirming efficient operation under design conditions. Refer to Appendices K and L for pump and system curve development. A summary of pump performance at the selected operating point is provided in Table 4-4.

Table 4-4: Pump Performance at Selected Operating Point

Parameter	Value
Flow per Pump	6,944 gpm
Total Dynamic Head (TDH)	62.1 ft
Pump Efficiency	~87.3%
Power Required	124.9 hp
NPSHr	21.8 ft
NPSHa	> 21.8 ft
Operating Region	Near Best Efficiency Point (BEP)

The Net Positive Suction Head available (NPSHa) in the wet pit configuration exceeds the Net Positive Suction Head required (NPSHr), confirming that cavitation will not occur and that the selected pump is hydraulically suitable for the system. (Davis, 2020) Pump selection was based on the developed system curve and required operating conditions described in Sections 6.1 and 6.2. Refer to Table 4 and Appendices E and K for supporting hydraulic calculations and pump selection parameters. The details for the wet pit can be found in Appendix I. (Davis, 2020)(Xylem, 2026)

A Goulds VIT vertical turbine pump (Model VIC/VIT, Size 20GHO) operating at 1180 RPM with a single stage was selected to meet the required flow and head conditions. At the operating point, the pump achieves approximately 87.3% bowl efficiency with a non-overloading power requirement of 124.9 hp. The required Net Positive Suction Head (NPSHr) of 21.8 ft is satisfied under the selected wet pit configuration. The wet well configuration can be found in Appendix H, the pump curve can be found in Appendix K, and the operating point verification can be found in Appendix L.

4.3 Pipe Design

The conveyance pipeline was designed for a flow of 40 MGD. A 42-inch ductile iron pipe (DIP) was selected with a total pipeline length of 10,826.8 ft. Refer to Appendix F for pipeline alignment and length. (Davis, 2020). The pipe diameter was selected to maintain appropriate flow velocities while minimizing head loss and ensuring efficient hydraulic performance. A summary of key conveyance pipeline hydraulic parameters is provided in Table 4-5, with detailed calculations included in Appendix E. (Davis, 2020)

Table 4-5: Conveyance Pipeline Hydraulic Performance Summary

Flow (MGD)	Flow (gpm)	Velocity (ft/s)	Friction Loss (ft)	Minor Loss (ft)	Static Head (ft)	TDH (ft)
40	27,778	6.43	33.64	3.86	24.61	62.11

Ductile iron pipe (DIP) was selected over prestressed concrete cylinder pipe (PCCP) based on constructability, flexibility, and compatibility with the pump station and site layout. DIP allows for easier alignment adjustments and improved integration with mechanical components such as valves and pump connections. A summary of the pipe material selection criteria is provided in Table 4-4 through 4-6. The final selection was based on a weighted evaluation of hydraulic performance, structural strength, constructability, sustainability, and corrosion resistance. The hydraulic performance analysis considered the velocity, flow rate, and pressure drop. This allowed for the design system to deliver a required capacity that does not have excessive operational costs. The pipe size and layout were fit to the most optimal to reduce friction and lower pumping costs. In addition. The structural (Davis, 2020)

Table 4-6: Pipe Material Selection Summary

Alternative	**Hydraulic Performance**	**Structural Strength**	**Constructability**	**Sustainable**	**Corrosion Resistance**	**Weighted Score**
	(20%)	**(15%)**	**(30%)**	**(20%)**	**(15%)**	**(100%)**
PCCP	4	5	3	4	3	3.85
DIP	4	4	5	4	4	4.25
Steel	4	4	4	3	3	3.65
HDPE	3	2	3	3	5	3

5. Final Design Recommendations

The final design will follow the previously stated treatment train from section 3. This treatment train is as follows: bar screening, coagulation and flocculation, high rate sedimentation, sand filtration, ultrafiltration, ion exchange, a hydrogen peroxide and UV AOP, final chlorination, and brine management by crystallization. All calculations which are discussed in this final design recommendation can be found in. The first step in this process is bar screening. It is recommended that this process uses a total of 10 bar screens with 8 being used at a time and with two for redundancy.

Again, the exact dosing for coagulation and flocculation cannot be determined without a jar test. It is predicted that the coagulants be injected at a dose of 15 mg/L. The recommended flocculant for this design is nonionic polymers. It is predicted that the polymers be injected at a dose of 30 mg/L (Johnson , et al., 2008), however the actual ferric chloride dose may fall between the range of 2 mg/L to 100 mg/L (Australian Drinking Water Guidelines , n.d.). These recommended doses will remove a majority of TSS from the system.

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Table 5-1: Calculated Sedimentation Basin Dimensions

Parameter	Units	Values
Tank Surface Area	m^2	336.75
Selected Width	m	4.8
Selected Length	m	70.2
Tank Length	m	93.5
% of tank covered by settler	%	75
Side Water Depth	m	4
Depth of Sludge Zone	m	2
Overflow Rate	m/d	180
Tube Angle	deg	60
Tube Diameter	m	0.07

Next, a rapid sand filter is recommended for this design. The recommended sand filter uses sand and granular activated carbon to remove solids from the water which were not removed during sedimentation. The following table shows the recommended design for the sand filter.

Table 5-2: Recommended Sand Filter Design

Parameter	Units	Values
Numbers of Beds	-	9
Bed Area	m^2	68.96
Cell Width	m	3.5
Bed Length	m	9.85
Gullet Width	m	0.6
Number of Launders	-	5
Trough Spacing	m	1.97
Trough Width	m	0.53
Trough Sidewall Height	m	0.375
Trough Freeboard	m	0.05
Trough Depth	m	0.64
Trough Elevation	m	1
Backwash Tank Volume	m^3	1379.2

Additionally, the ultrafiltration for this treatment system was designed. For this design, it is recommended that a Conva STARK ultrafiltration system is used. This treatment system has a flow rate of 160 m^3/hr per skid (Conva STARK , n.d.). This equates to a recommended 53 skids with 48 in use and 5 for redundancy.

The Ion Exchange systems have a wide range of flow rates, ranging from 10-1200 GPM (Industrial Water Solutions , 2026). It is recommended that the facility install 30 of these units, each with a flow rate of 1000 GPM, with 28 in operation at a time and 2 for redundancy. Following UF, it is recommended that ion exchange systems be implemented. For this project, Industrial Water Solutions ion exchange units were researched. These systems have a wide range of flow rates, ranging from 10-1200 GPM (Industrial Water Solutions , 2026). It is recommended that the facility install 30 of these units, each with a flow rate of 1000 GPM, with 28 in operation at a time and 2 for redundancy. This system is capable of applying a UV dose of 300 mJ/cm^2 and incorporates hydrogen peroxide injection in the system. Each of these vessels have a flow rate of between 1000 to 2000 GPM (TROJAN technologies , 2026). It is recommended that a total of 13 units at a flow rate of 1750 GPM be installed.

The recommended chlorine dose for this facility is 0.8 mg/L, however due to unknown distribution system conditions, the actual chlorine dose may change once it enters the Arizona Drinking System. To ensure that the effluent water meets the required chlorine residual levels set by the ADEQ Surface Water Treatment Rule, chlorine must be added before the water enters the potable water system. This rule states system must maintain a chlorine residual of 0.2 mg/L for four hours after leaving the treatment plant ￼.It is recommended that a total of 6 units be installed for this facility with 5 in use a time and one for redundancy.

6. Design Cost Analysis

The total project cost for the proposed Advanced Water Purification Facility (AWPF) is summarized in Table 6-1 and includes major capital components such as the lift station, treatment facilities, site development, and engineering services. The estimated construction cost represents an Opinion of Probable Construction Cost (OPCC), which is based on conceptual design assumptions, unit cost estimates, and historical cost data from RSMeans and comparable infrastructure projects (RSMeans, 2026; Labor, 2026).

The OPCC is estimated at approximately $329.28 million, which includes both the lift station and the primary treatment and site development components. The lift station cost of $13.47 million includes construction of the wet well, vertical turbine pumps, discharge piping, valves, electrical systems, and associated structural components required to convey flow through the facility. The remaining $315.81 million represents treatment facilities and site development, including process units such as coagulation, sedimentation, filtration, ion exchange, advanced oxidation (UV/H_2O_2), disinfection systems, site grading, structural construction, and installation of mechanical and electrical systems.

Engineering services are estimated at $82.9 million, or approximately 20% of the construction cost, and include planning, detailed design, permitting, project management, construction administration, and inspection services. These services are necessary to ensure the project is designed in accordance with regulatory requirements and constructed according to specifications.

Table 6-1: Summary of Cost Analysis

Category	**Cost ($)**
Lift Station	13,469,781
Proposed Site	315,809,193
Opinion of Probable Construction Cost (OPCC)	329,278,974
Engineering Services	82,900,000
Total Project Cost	412,178,974
Estimated Annual O&M Cost (4% of construction cost)	13,170,000

The total project cost is estimated at approximately $412.18 million, which represents the combined cost of construction and engineering services. In addition to capital costs, the estimated annual operation and maintenance (O&M) cost is approximately $13.17 million per year, based on 4% of the construction cost. This includes ongoing expenses such as energy consumption, chemical usage, labor, maintenance, and replacement of consumable materials required to operate the facility.

A more detailed breakdown of cost assumptions, unit pricing, and lifecycle considerations is provided in Appendix M:.

7. Construction Phasing

The construction of the 91st Avenue Advanced Water Purification Facility (AWPF) will be completed in phases so the existing 91st Avenue Wastewater Treatment Plant (WWTP) and Tres Rios Flow Regulating Wetlands (FRW) can continue operating during construction. This is important because the existing wastewater treatment and wetland systems cannot simply be shut down while new facilities are being built. The phasing plan allows construction crews to build the new treatment facility, pump station, and conveyance piping while maintaining the movement of water through the existing system.

The first phase will include site preparation, clearing, grading, excavation, and temporary construction access. This phase prepares the site for major structures by creating stable work areas, establishing erosion control, and setting up temporary utilities and staging areas. During this phase, construction sequencing must also protect existing nearby infrastructure and maintain safe access for plant operators, construction crews, and maintenance vehicles.

The next phase will include installation of underground utilities and the 42-inch conveyance pipeline. This pipeline is a major part of the project because it carries water from the wetland area toward the AWPF. Since the FRW must remain in operation, pipeline tie-ins should be completed using phased connections, temporary bypass pumping, or short-duration shutdown windows when flows are lowest. These methods allow water to continue moving through the system while the new piping is connected.

The pump station will be constructed early in the project because it provides the hydraulic lift needed to move water through the proposed system. This work includes construction of the wet well, installation of vertical turbine pumps, discharge piping, check valves, isolation valves, electrical equipment, and control systems. The proposed pump station includes four duty pumps and one standby pump, which provides redundancy if one pump is offline for maintenance or repair. This helps improve reliability and reduces the risk of service interruption.

After the pump station and conveyance system are established, the treatment processes will be constructed in the direction of flow. This includes initial treatment processes such as coagulation and flocculation, followed by sedimentation, filtration, ion exchange, advanced oxidation using ultraviolet light and hydrogen peroxide, and final chlorination. Building the treatment processes in the direction of flow helps organize construction and allows individual systems to be tested in sequence before the entire facility is placed into operation.

Brine management, electrical systems, instrumentation, and control systems will be completed during later phases. These systems are critical because they support daily plant operation, monitor water quality, control pumps and valves, and manage waste streams produced by treatment. Parallel construction activities may be used where possible to reduce the overall schedule, but critical tie-ins and testing must be carefully coordinated to avoid disruptions. The total construction duration is estimated at approximately 36 to 48 months, depending on permitting, equipment procurement, construction sequencing, and final commissioning requirements.

7.1 Commissioning & Startup

After construction is completed, the AWPF will go through commissioning before full operation begins. Commissioning is the process of testing the facility to confirm that equipment, controls, pipes, pumps, and treatment systems work as intended. This step is necessary because the facility must consistently produce high-quality purified water before it can operate at full capacity.

The first step is dry commissioning. During dry commissioning, equipment is tested before water is introduced into the system. This includes checking electrical connections, pump rotation, valve operation, chemical feed systems, sensors, alarms, and control panels. Dry commissioning helps identify mechanical or electrical issues early, before the system is exposed to full hydraulic loading.

The second step is wet commissioning. During wet commissioning, water is gradually introduced into the facility, so operators can test hydraulic performance and treatment effectiveness. This includes checking flow rates, water levels, pressure, head-loss, pump performance, chemical dosing, filtration performance, and disinfection effectiveness. Operators also verify that each treatment process can meet the required performance standards before the system is placed into continuous service.

Final startup occurs after the facility demonstrates reliable operation and meets regulatory requirements. During this period, operators monitor water quality frequently and make adjustments to chemical dosing, flow control, and treatment settings. Full operation is expected to be reached within approximately 3 to 6 months after construction is completed. Commissioning is considered a one-time startup activity and should not be included as part of the annual O&M cost.

7.2 Annual Operation & Maintenance Cost

The estimated annual operation and maintenance cost for the AWPF is $13,170,000 per year, based on approximately 4% of the construction cost. This annual cost represents the money needed to operate the facility after construction is complete. Unlike construction costs, which are paid once to build the project, O&M costs occur every year and include energy, chemicals, labor, equipment maintenance, replacement materials, and residuals management.

Energy is one of the major annual costs because the facility requires power to move and treat water. Electricity is needed to operate the lift station pumps, conveyance equipment, membrane systems, ultraviolet disinfection units, chemical feed systems, control panels, and other mechanical equipment. Pumping is especially important because water must be lifted and moved through the treatment process at the required flow rate.

Chemical costs are also included in the annual O&M estimate. Ferric chloride and polymers may be used during the initial treatment stage to help particles clump together so they can be removed more easily. Chlorine is used for final disinfection to protect water quality before distribution or storage. Hydrogen peroxide is used with ultraviolet light during the advanced oxidation process to help break down trace organic compounds. These chemicals must be purchased regularly and dosed carefully based on flow rate and water quality.

Consumable materials are another major part of annual O&M. Granular activated carbon may require periodic replacement or regeneration because its ability to adsorb contaminants decreases over time. Membranes used in filtration processes must be cleaned, maintained, and eventually replaced as performance declines. Ion exchange resin also requires regeneration or replacement because it becomes exhausted after removing target constituents from the water. These replacement cycles are necessary to keep the treatment process reliable.

The facility is expected to operate continuously and will require staffing to support 24-hour operations. A typical staffing configuration for a facility of this size includes approximately 6 to 8 certified operators, distributed across three shifts to ensure continuous monitoring and control of treatment processes. Each shift would generally include one to two operators responsible for system monitoring, process adjustments, and responding to operational alarms.

Additional personnel would include maintenance staff (approximately 4 to 6 individuals), laboratory staff (2 to 3 individuals), and management and supervisory personnel (3 to 5 individuals). This staffing structure supports reliable operation, routine maintenance, water quality monitoring, and regulatory compliance.

Labor costs include the staff needed to operate, monitor, and maintain the AWPF. This includes certified operators who run the facility, maintenance staff who inspect and repair equipment, laboratory staff who test water quality, and management staff who oversee operations, reporting, safety, and regulatory compliance.

Maintenance and repair costs include routine inspections, pump servicing, valve replacement, instrumentation calibration, electrical repairs, membrane cleaning, and replacement of worn mechanical parts. These costs are important because advanced treatment facilities depend on reliable equipment. Regular maintenance helps prevent unexpected shutdowns and extends the service life of major equipment.

Residuals and brine management are also included in the annual O&M cost. Treatment processes can produce waste streams such as concentrate, spent media, sludge, or other residuals that must be handled, treated, hauled, or disposed of properly. Brine management can be a significant cost because concentrate streams often require additional treatment or disposal methods. Including these costs gives a more realistic estimate of what the facility will require after construction.

8. Project Impacts

The proposed 91st Avenue AWPF will impact the environment, surrounding community, and overall performance of Phoenix’s drinking water system. Environmentally, the project introduces considerations related to resource use, energy demand, and management of treatment residuals. From a community standpoint, the facility may influence water supply reliability and public perception. In terms of system performance, the AWPF will affect the reliability, flexibility, and long-term sustainability of the existing water infrastructure.

8.1 Public Health, Safety and Welfare Impacts

The facility provides several benefits, including increased water supply reliability through the reuse of treated wastewater. By converting effluent into potable water, the system reduces dependence on traditional freshwater sources and supports long-term water sustainability. Advanced treatment processes also improve water quality by removing contaminants such as Giardia and Cryptosporidium prior to distribution, significantly reducing the risk of waterborne disease and protecting public health.

By following the requirements set by the 2025 Arizona AWP Rule, this facility ensures safe potable water production through rigorous monitoring, multiple treatment barriers, and continuous system validation. These safeguards minimize the risk of contaminants entering the drinking water supply and provide rapid detection and response in the event of system irregularities. Regular water quality testing before distribution ensures that all regulatory standards are consistently met, reinforcing public confidence in the safety of the water.

From a public health and safety perspective, the facility enhances community resilience by providing a drought-resistant water source that reduces vulnerability to water shortages and supply disruptions. This reliability is critical for maintaining sanitation, supporting emergency services, and protecting vulnerable populations during extreme conditions. Additionally, the project promotes overall public welfare by ensuring equitable access to clean, safe drinking water, supporting economic stability, and contributing to a sustainable and secure water future for the community.

8.2 Social Impacts

From a social perspective, the success of the facility depends heavily on public perception, trust, and community engagement. Because direct potable reuse can initially face skepticism, transparent communication and education are essential. Public outreach efforts—such as community meetings, facility tours, and school partnerships—help demystify the treatment process and allow residents to better understand the science and safety behind the system. As familiarity increases, public acceptance tends to grow, reducing resistance and fostering a sense of shared responsibility for sustainable water use.

Equity is another key social factor. The facility supports consistent and reliable access to clean drinking water across all communities, including underserved or vulnerable populations that are often most affected during droughts or water shortages. By stabilizing supply, the system helps prevent disparities in water availability and affordability, contributing to a more equitable distribution of resources.

In addition, the facility creates opportunities for community involvement and workforce development. Educational partnerships with local schools and universities can introduce students to careers in water treatment and environmental engineering, helping to build a skilled workforce for the future. Public access initiatives, such as internships, training programs, and local hiring efforts, can further integrate the facility into the community. These opportunities not only support economic mobility but also strengthen the connection between the public and the infrastructure that serves them.

From a global social perspective, the implementation of direct potable reuse contributes to a growing shift in how communities perceive and manage water. Around the world, regions facing water stress are adopting similar technologies, and projects like this help normalize water reuse as a safe and viable solution. This can influence public acceptance beyond the local level, contributing to wider societal adoption of sustainable practices. Additionally, by ensuring a reliable water supply, the facility supports population stability, public health, and quality of life—factors that are increasingly recognized as essential to social resilience in water-scarce regions globally.

8.3 Environmental Impacts

The proposed withdrawal of approximately 40 MGD from the Tres Rios Flow Regulating Wetlands (TRFRW) will alter the existing flow regime, as this water is typically discharged to the Salt River. While this diversion may reduce downstream flows, the river will continue to receive water from the wetlands and other contributing sources. However, even modest flow reductions can influence local ecosystems by affecting habitat availability, water temperature, and dissolved oxygen levels, which are critical for aquatic species. Changes in flow patterns may also impact riparian vegetation that depends on consistent surface or shallow groundwater conditions.

The facility will also require significant energy to operate advanced treatment processes such as UV disinfection, reverse osmosis, and advanced oxidation. This energy demand can contribute to indirect environmental impacts, particularly greenhouse gas emissions if sourced from non-renewable energy. Incorporating renewable energy sources, such as on-site solar generation or renewable energy credits, can help offset these impacts and improve the overall sustainability of the facility. Additionally, optimizing system efficiency through energy recovery devices and process design can further reduce the carbon footprint of the facility.

Another important environmental consideration is concentrate (brine) management from reverse osmosis. Improper disposal of this high-salinity waste stream could negatively affect soil and water quality. Implementing responsible brine management strategies—such as controlled discharge, evaporation ponds, or advanced zero-liquid discharge systems—can minimize ecological harm. Furthermore, the facility can provide environmental benefits by reducing the need for groundwater extraction and limiting the discharge of partially treated effluent into natural waterways, ultimately supporting long-term watershed health and more sustainable water resource management.

Environmentally, the facility reflects a transition toward more sustainable water systems in response to global challenges such as climate change and ecosystem degradation. Reduced dependence on overdrawn rivers and aquifers helps preserve natural systems not only locally but also contributes to broader watershed and regional environmental health. While energy use and brine disposal present environmental challenges, integrating renewable energy and responsible waste management aligns with global efforts to reduce greenhouse gas emissions and minimize ecological impact. Overall, the facility supports a more circular water economy, where water is reused and conserved rather than continuously extracted, reflecting a model that is increasingly important worldwide.

8.4 Economic Impacts

Although this facility will have a higher initial construction and capital cost than a conventional water treatment plant, it offers substantial long-term economic benefits for the City of Phoenix. By decreasing reliance on surface water from the Salt River, groundwater aquifers, and imported supplies, the facility can reduce long-term procurement, pumping, and conveyance costs. This is particularly important as water from distant or overallocated sources becomes increasingly expensive due to scarcity, regulatory constraints, and infrastructure demands.

In addition, the facility provides cost stability and predictability. Traditional water sources are vulnerable to drought, climate variability, and interstate water agreements, all of which can lead to fluctuating costs. By creating a locally controlled and drought-resilient supply, the city can better manage future water rate increases and reduce financial uncertainty for residents and businesses. Over time, this reliability can offset the higher upfront investment through avoided costs and reduced exposure to supply disruptions.

The project also contributes to the local economy through job creation and workforce development. Construction of the facility generates short-term employment opportunities, while long-term operation and maintenance require skilled professionals in engineering, operations, and water quality management. This supports sustained job growth and can stimulate related industries, including equipment supply, maintenance services, and environmental consulting.

Furthermore, a reliable and sustainable water supply enhances international economic growth and development. Businesses—particularly those in water-intensive industries—are more likely to invest in areas with secure infrastructure. This can increase the city’s tax base, support industrial expansion, and promote long-term economic resilience. Overall, while initial investment is significant, the facility represents a strategic economic asset that reduces future costs, stabilizes water pricing, and supports continued growth in the region.

8.5 Public Outreach Plan

Due to the relative rarity of AWP systems in the United States and the understandable public skepticism about treating wastewater to drinking-water standards, robust public outreach will be essential to the success of this project. Building trust and transparency around the treatment process is just as important as the technical design itself. To address this, the project team proposes a comprehensive outreach strategy that includes community meetings, informational sessions, partnerships with public high schools and universities, and live events.

The first phase focuses on direct engagement with the local community through scheduled public meetings and informational sessions. These events will be held on a recurring basis, such as monthly or quarterly, at accessible public locations including community centers, libraries, and municipal facilities. Each session will be led by engineers, plant operators, and water quality professionals who can explain the treatment process in clear, non-technical language. Topics will include how wastewater is treated to meet drinking water standards, system reliability, safety protocols, and regulatory requirements. Attendees will be encouraged to ask questions and raise concerns, allowing for two-way communication rather than one-directional presentations. This phase ensures that the public has access to accurate information directly from technical experts, reducing misinformation, and establishing a foundation of understanding early in the project.

The second phase introduces hands-on demonstrations that allow community members to directly observe and evaluate the treatment process. These events will be hosted at demonstration facilities, pilot systems, or controlled environments designed to simulate the AWPF process. Participants will be invited to taste the treated water and compare it with the current drinking water supply, helping to demystify the process and address common misconceptions. In addition to sensory comparisons, the demonstrations will include clear presentations of the chemical composition of each water source, highlighting differences in purity, mineral content, and safety standards. Visual aids, simplified process diagrams, and live data displays will be used to make technical information accessible. Demonstrations may also include sample testing displays or side-by-side comparisons that highlight the effectiveness of each treatment step. This phase focuses on replacing uncertainty with direct observation, allowing the public to evaluate water quality based on evidence rather than perception.

The final phase focuses on long-term community impact through education and youth engagement. Partnerships will be established with local schools, colleges, and universities to introduce water reuse concepts to students at multiple grade levels. This effort will involve partnerships with local schools, including classroom presentations, interactive workshops, and facility tours that allow students to see the treatment process firsthand. Hands-on activities and age-appropriate learning materials will help students better understand both the science and the real-world importance of water purification. Importantly, this educational outreach can extend beyond the classroom. As students develop knowledge and enthusiasm for the subject, they often share what they learn with their families, helping to inform parents and reinforce community-wide understanding. In this way, youth engagement not only builds long-term awareness but also contributes to broader public acceptance in the present. By promoting early education and curiosity, this initiative aims to create a more informed, supportive, and forward-thinking community around advanced water purification.

9. Summary of Engineering Work

The project schedule was originally developed during CENE 476 and outlined a structured timeline for completing research, treatment selection, design, and final deliverables. This proposed schedule assumed a linear progression of tasks, with major phases such as treatment selection, hydraulic design, and cost analysis occurring sequentially.

However, the actual schedule differed in several key ways due to project constraints, workload distribution, and the iterative nature of the design process. A comparison between the proposed and actual Gantt charts shows that while the overall project duration remained similar, the sequencing and overlap of tasks were adjusted to better reflect real project conditions. The proposed schedule could not have constraints or barriers because it was still in planning since, they distort critical path analysis, and hide true project risks, such as resource unavailability or illogical sequencing. The proposed and actual schedules can be found in Appendix N: and Appendix O:.

9.1 Comparison of Proposed vs. Actual Schedule

In the proposed schedule, tasks such as treatment selection and site assessment were planned to occur earlier and in a more sequential order. However, the actual schedule shows that many of these tasks overlapped and were completed concurrently. For example, treatment process evaluations, including coagulation, sedimentation, membrane filtration, and advanced oxidation, were performed in parallel rather than one after another.

This shift allowed the team to evaluate multiple alternatives simultaneously, which improved efficiency but required greater coordination among team members. Additionally, design tasks such as hydraulic analysis, pump selection, and site layout were initiated earlier in the actual schedule, overlapping with ongoing treatment selection efforts.

9.2 Key Schedule Changes and Their Causes

The primary deviation between the proposed and actual project schedules was driven by the delay in the site visit, which was pushed back by approximately one month. In the original schedule, site assessment and data collection were intended to occur earlier in the project to inform treatment selection, hydraulic design, and overall system configuration. However, as reflected in the actual Gantt chart, the site visit and associated data analysis occurred later in the timeline, which delayed access to critical information about the existing wastewater treatment plant (WWTP), including flow characteristics, infrastructure constraints, and operational conditions.

As a result, several downstream tasks could not be completed as originally planned. Treatment selection, process evaluation, and design development were initiated with limited site-specific data, requiring the team to rely on preliminary assumptions and generalized design criteria. Once site-specific information became available, portions of the design had to be revisited and refined to better align with actual conditions. This led to a more iterative workflow, where treatment evaluation, hydraulic analysis, and equipment selection progressed in parallel rather than sequentially.

Additionally, the delay compressed the available time for later design phases, requiring increased overlap between major tasks such as hydraulic analysis, pump selection, and site layout. This shift is evident in the actual schedule, where multiple design components occur concurrently to maintain progress toward key deliverables. Despite the initial delay, the team adapted by restructuring task sequencing and prioritizing critical path activities, allowing the project to remain on schedule and meet required submission deadlines.

10. Summary of Engineering Costs

At the completion of the project, the proposed staffing plan was compared to the actual engineering hours recorded by the team. This comparison is shown in Table 10-1 and helps evaluate how accurately the original staffing plan estimated the level of effort required for each major project task. The proposed plan estimated 690 total engineering hours, while the actual completed work required 700.5 hours. Overall, this difference is relatively small, showing that the total project labor estimate was generally accurate. However, the distribution of hours between individual tasks changed as the project developed. A comparison of proposed and actual engineering hours can be understood through Table 10-1.

Some tasks required fewer hours than originally anticipated. Site assessment, treatment selection, and impact analysis were lower than the proposed estimates. This occurred partly because the site visit was delayed, which limited the amount of site-specific information available early in the semester. As a result, some early project tasks could not be completed in the same level of detail originally planned and were either shortened, revised, or shifted into later design tasks. Treatment selection also required fewer hours than expected because several alternatives were evaluated at a planning level rather than being fully developed into detailed designs.

Table 10-1: Proposed vs Actual Engineering Hours

	Tasks	**S ENG (hrs)**	**PM (hrs)**	**PE (hrs)**	**EIT (hrs)**	**INT (hrs)**	**Total (hrs)**
Proposed	1. Research Preparation	1	0	1	12	16	**30**
	2. Site Assessment	9	0	9	14	10	**42**
	3. Treatment Selection Process	10	10	40	80	70	**210**
	4. Final Design	10	10	30	70	38	**158**
	5. Cost Assessment	0	0	5	20	15	**40**
	6. Impact Analysis	0	0	5	10	10	**25**
	7. Project Deliverables	15	5	15	50	30	**115**
	8. Project Management	5	15	10	20	20	**70**
	Total	**50**	**40**	**115**	**276**	**209**	**690**
Actual	1. Research Preparation	0	0	0	0	25	**25**
	2. Site Assessment	0	0	3	6	8	**17**
	3. Treatment Selection Process	4	0	27	39	21.5	**91.5**
	4. Final Design	9	1.5	60	62	86	**218.5**
	5. Cost Assessment	15	5.5	0	16	12	**48.5**
	6. Impact Analysis	0	0	0	8	4	**12**
	7. Project Deliverables	21	18	39	61	61	**200**
	8. Project Management	13	18	11	29	17	**88**
	Total	**62**	**43**	**140**	**221**	**234.5**	**700.5**

Other tasks required more hours than originally estimated. Final design increased from 158 proposed hours to 218.5 actual hours, making it one of the largest differences in the staffing comparison. This increase occurred because the final design required more detailed work on the hydraulic profile, pump selection, pipe layout, process flow, and overall facility configuration than originally expected. Project deliverables also increased from 115 proposed hours to 200 actual hours, which reflects the time required to revise the report, prepare presentation materials, organize appendices, format tables and figures, and respond to review comments. These hours were necessary to turn the technical design work into a complete final submission.

Cost assessment also increased slightly from 40 proposed hours to 48.5 actual hours. This was expected because construction cost, O&M cost, and life cycle cost estimates required additional coordination with the final design assumptions. As quantities, process selections, and facility components became more defined, the cost estimate had to be updated to better reflect the proposed AWPF layout. Project management also increased from 70 proposed hours to 88 actual hours, which reflects the continuous coordination needed between team members as tasks overlapped and the schedule changed.

Overall, the comparison shows that the team's total hour estimate was close, but the original staffing plan did not fully predict where the effort would be needed most. Fewer hours were spent on early planning and assessment tasks, while more hours were required for final design, deliverable preparation, cost development, and project coordination. This is reasonable for a conceptual engineering project because early assumptions often change as more information becomes available. The results from this comparison can be used to improve future project planning by assigning more time to final design development, report production, and coordination tasks.

Table 10-2: Proposed vs. Actual Cost of Engineering Services

Personnel	**Classification**	**Billing Rate ($/hr)**	**Time (hrs)**	**Cost**
	SENG	325	50	$16,250
	Engineer (PE)	190	115	$21,850
	Project Manager	174	40	$6,960
	EIT	130	276	$35,880
	Engineer Intern	42	209	$8,778
	Total Personnel	-		$89,718
Travel/ Competition	**Classification**	**Units**	**Per. Unit**	**Cost**
	WEF Application	4 People	$21/person	$84
	2 trips (round)	620 mi	$0.42/mi	$260
	Vehicle	3 days	$77/day	$230
	Hotel: rooms x nights	3 x 3	$150/night	$1,350
	Per Diem 5 ppl	2 days	$62/day	$620
	Total Travel			$1,924
Supplies	**Classification**	**Units**	**Per. Unit**	**Cost**
	Software and Computer Lab	30	$100/day	$3,000
				$94,642
Actual COES	**Project Total**	**Per. Project**		**Cost**
	$412,178,974	20%		$82,899,999.83

Table 10-2 shows that the total of engineering service cost was determined with a different method compared to the initial proposal. The initial proposal included all engineering services involved with conceptual project standards and project work. The final includes the cost of managing, administration, and permitting over the duration of the completion of the actual project mirroring the real-world cost of engineering services commonly associated with a project of this scale.

11. Conclusion

The use of advanced water purification (AWP) in Arizona has become increasingly relevant as growing populations; prolonged drought conditions, and overallocated surface water supplies continue to strain traditional water sources. In regions like Phoenix, where reliance on the Colorado River and groundwater has become less sustainable, the development of facilities such as the 91st Avenue Advanced Water Purification Facility represents a proactive and necessary shift toward water resilience. By treating wastewater to potable standards and reintroducing it into the supply, this facility helps diversify the city's water portfolio while reducing dependence on diminishing natural resources.

The selected treatment train for this facility integrates multiple physical, chemical, and advanced treatment processes to ensure both reliability and high-water quality. Preliminary screening through vertical bar screens removes large debris, followed by coagulation and flocculation using ferric chloride and nonionic polymers to aggregate suspended particles. High-rate tube settlers and dual media rapid sand filters then provide efficient solid-liquid separation and polishing. Advanced treatment steps, including ultrafiltration, ion exchange, and an advanced oxidation process using hydrogen peroxide and ultraviolet light, target dissolved contaminants, trace organics, and pathogens at a molecular level. Final chlorination ensures residual disinfection within the distribution system, while crystallization addresses brine management by reducing waste and enabling more sustainable disposal or recovery of salts.

This treatment train was ultimately selected due to its strong performance across key evaluation criteria, including environmental impact, energy efficiency, cost-effectiveness, operational flexibility, and land use. Its design reflects meeting regulatory requirements but also supporting long-term sustainability goals.

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Appendix A: Given Water Quality at Outfall 005

Table 5 Summary of Assumed AWPF Feed Water Quality from SROF [redacted]

Parameter	Units	Average	95th Percentile
Alkalinity	mg/L as $CaCO_3$	181	210
Ammonia(1)	mg/L as N	1.2	1.9
Barium(2)	mg/L	0.0	0.1
Boron (Total)(1)	mg/L	0.4	0.4
Calcium (Total)	mg/L	78	87
Chloride	mg/L	367	510
Fluoride	mg/L	1.4	1.7
Magnesium (Total)(1)	mg/L	32	37
Manganese (Total)(1)	µg/L	0.0	0.1
Nitrate(1)	mg/L as N	4.6	6.3
Nitrite	mg/L as N	0.2	0.3
Orthophosphate(2)	mg/L as P	3.2	3.6
pH	s.u.	7.4	8.1
Potassium (Total)	mg/L	23	25
Silica[2]	mg/L as SiO_2	16	18
Sodium (Total)(1)	mg/L	256	330
Strontium (Total)(3)	mg/L	1.0	1.1
Sulfate	mg/L	181	210
TDS	mg/L	1,142	1,335
Temperature	°C	25	30
TKN	mg/L as N	3.3	4.3
TSS	mg/L	8.6	21

[redacted]

[redacted]

Appendix B: Treatment Processes Decision Matrices

Alternative	Environmental impact (0.125)	Energy Efficiency (0.125)	Operational and Capital Cost (0.25)	Operational Flexibility (0.25)	Land Use (0.25)	Total Score
Manual Screen	3	3	2	1	1	1.75
Vertical Screen	2	2	2	3	2	2.25
Curved Screen	2	2	2	1	3	2

Coarse Bar Screen

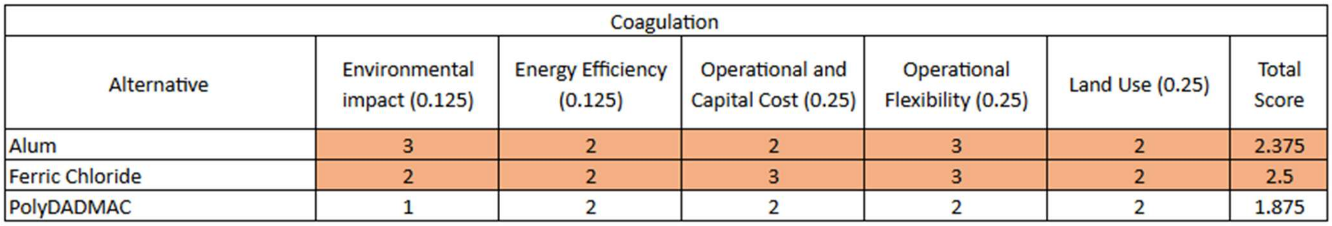
- Pros
    - Low installation and maintenance cost
    - Low energy requirements
    - Simple design
- Cons
    - Typically manual
    - Poor removal
    - Limited flexibility for operation
    - Large land use requirements

Vertical Bar Screens

- Pros
    - Adapts well to variant flow conditions
    - Best at debris removal
    - Decent land use
- Cons
    - Higher capital and maintenance costs
    - Moderate energy consumption

Curved Bar Screens

- Pros
    - Minimal land use
    - Moderate cost
    - Efficient removal across a range of particle sizes (good operational flexibility)
- Cons
    - Less operational flexibility
    - Moderate pollutant removal efficiency
    - Medium maintenance requirements



Coagulation						
Alternative	Environmental impact (0.125)	Energy Efficiency (0.125)	Operational and Capital Cost (0.25)	Operational Flexibility (0.25)	Land Use (0.25)	Total Score
Alum	3	2	2	3	2	2.375
Ferric Chloride	2	2	3	3	2	2.5
PolyDADMAC	1	2	2	2	2	1.875

Alum

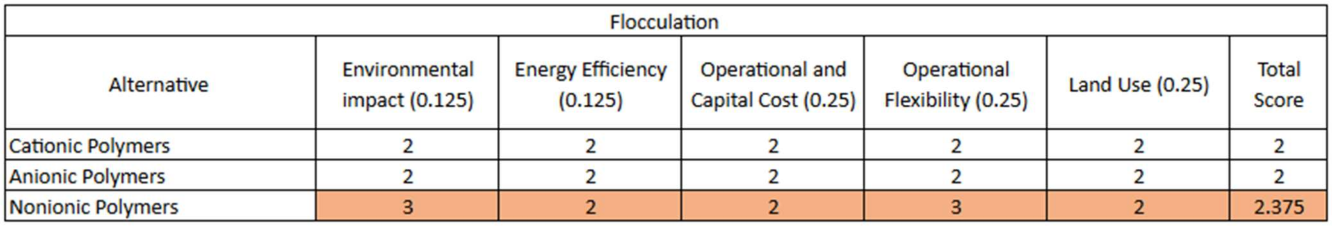
- Pros
    - Commonly used in water and wastewater treatment
    - Effective when removing turbidity
    - Low maintenance cost
- Cons
    - Large amounts of sludge
    - Can negatively impact the pH of the system
    - Concerning residuals

Ferric Chloride

- Pros
    - Strong Performance
    - Wide range of pH without impacting it
    - Removes phosphorous well
- Cons
    - Can be corrosive

PolyDADMAC

- Pros
    - Low sludge production
    - Low doses can still be effective
    - Fast reaction
- Cons
    - High cost
    - Harmful biproducts
    - Can not be useful when removing turbitity



Flocculation						
Alternative	Environmental impact (0.125)	Energy Efficiency (0.125)	Operational and Capital Cost (0.25)	Operational Flexibility (0.25)	Land Use (0.25)	Total Score
Cationic Polymers	2	2	2	2	2	2
Anionic Polymers	2	2	2	2	2	2
Nonionic Polymers	3	2	2	3	2	2.375

Cationic Polymers

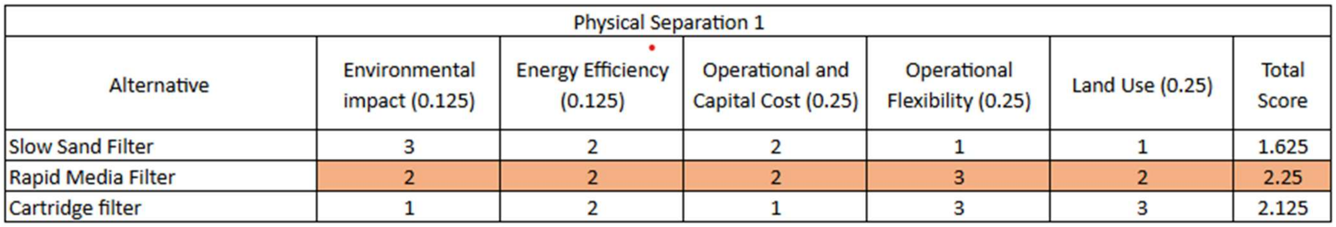
- Pros
    - Good for negatively charged particles
    - Rapid Flocc
- Cons
    - Overdosing is easy
    - Can produce negative biproducts

Anionic Polymers

- Pros
    - Good for positively charged particles
    - Cost-effective
- Cons
    - Not very versitile
    - Not good for specific water chemistry

Nonionic Polymers

- Pros
    - Very versatile
    - Stable
    - Low overdose risk
- Cons
    - Can be more expensive comparatively



Physical Separation 1

Alternative	Environmental impact (0.125)	Energy Efficiency (0.125)	Operational and Capital Cost (0.25)	Operational Flexibility (0.25)	Land Use (0.25)	Total Score
Slow Sand Filter	3	2	2	1	1	1.625
Rapid Media Filter	2	2	2	3	2	2.25
Cartridge filter	1	2	1	3	3	2.125

Slow Sand Filter

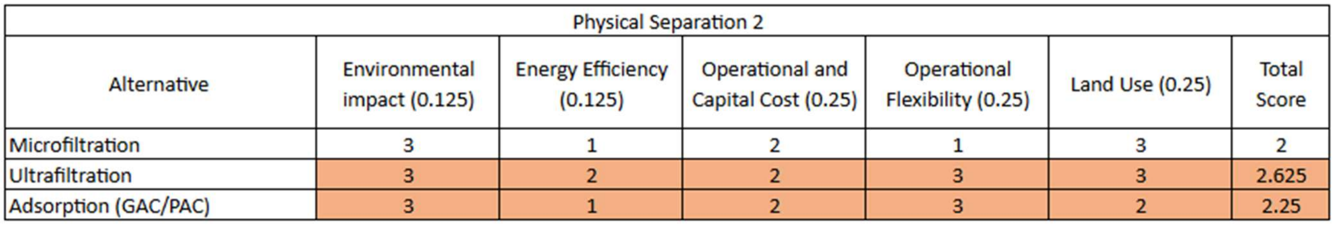
- Pros
    - Low energy requirements
    - Simple Operation
    - High operation flexibility
- Cons
    - Requires lots of land
    - Slow filtration
    - Not good for turbid waters

Rapid Media Filter

- Pros
    - High filtration rate
    - Low land use
    - Very good for TSS
- Cons
    - Moderate Energy use
    - Backwashing

Cartridge Filter

- Pros
    - High precision
    - Easy to install
- Cons
    - Needs to be replaced frequently
    - High operation cost



Physical Separation 2						
Alternative	Environmental impact (0.125)	Energy Efficiency (0.125)	Operational and Capital Cost (0.25)	Operational Flexibility (0.25)	Land Use (0.25)	Total Score
Microfiltration	3	1	2	1	3	2
Ultrafiltration	3	2	2	3	3	2.625
Adsorption (GAC/PAC)	3	1	2	3	2	2.25

MF

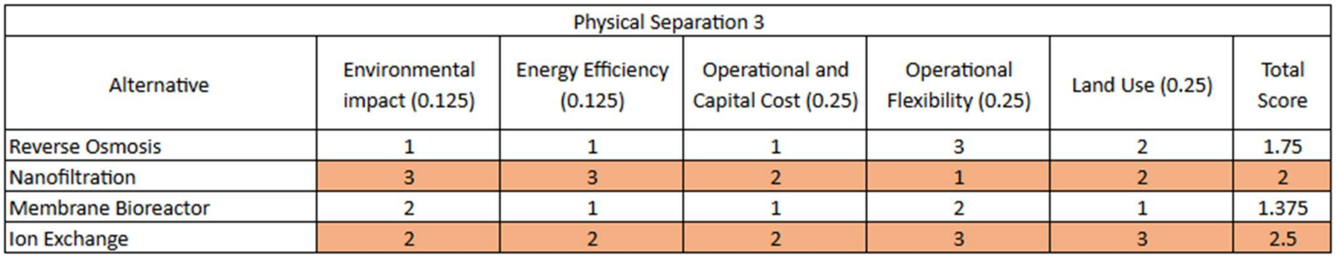
- Pros
    - Removes TSS and bacteria
    - A Reliable filter
- Cons
    - Membrane fouling
    - High Cost

UF

- Pros
    - Removes viruses, bacteria and some TDS
    - Highly filtered output
- Cons
    - High Cost
    - Fouling

GAC

- Pros
    - Removes organics and odor
    - Can be flexible in operation
- Cons
    - Media needs to be frequently replaced
    - Expensive



Physical Separation 3						
Alternative	Environmental impact (0.125)	Energy Efficiency (0.125)	Operational and Capital Cost (0.25)	Operational Flexibility (0.25)	Land Use (0.25)	Total Score
Reverse Osmosis	1	1	1	3	2	1.75
Nanofiltration	3	3	2	1	2	2
Membrane Bioreactor	2	1	1	2	1	1.375
Ion Exchange	2	2	2	3	3	2.5

RO

- Pros
    - High TDS removal
- Cons
    - Brine disposal issues
    - High energy consumption
    - High water loss
    - High Cost

Nanofiltration

- Pros
    - Removes ions
    - Low energy
- Cons
    - Limited tds removal

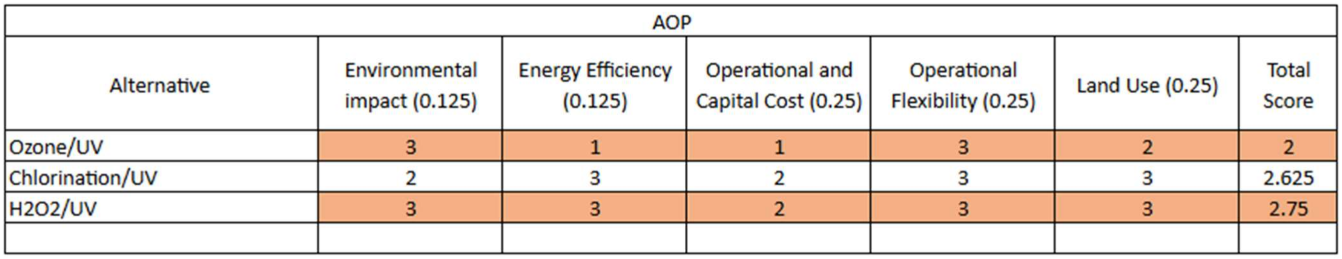
Membrane Bioreactor

- Pros
    - Combines processes
- Cons
    - Very complex
    - High energy
    - Expensive

Ion Exchange

- Pros
    - Highly selective removal
    - Regnerative

- Cons
    - Excess Brine



AOP						
Alternative	Environmental impact (0.125)	Energy Efficiency (0.125)	Operational and Capital Cost (0.25)	Operational Flexibility (0.25)	Land Use (0.25)	Total Score
Ozone/UV	3	1	1	3	2	2
Chlorination/UV	2	3	2	3	3	2.625
H2O2/UV	3	3	2	3	3	2.75

Ozone

- Pros
    - Very strong oxidant
    - Good for disinfection
- Cons
    - Complex
    - Lots of energy usage

Chlorination

- Pros
    - Low cost
- Cons
    - Harmful biproducts
    - Not effective

Hydrogen Peroxide

- Pros
    - Clean oxidation
    - Effective micropollutant removal
- Cons
    - Expensive

Brine Management						
Alternative	Environmental impact (0.125)	Energy Efficiency (0.125)	Operational and Capital Cost (0.25)	Operational Flexibility (0.25)	Land Use (0.25)	Total Score
Concentration and crystallization	3	2	3	2	1	2.125
Membrane Distillation	1	2	1	2	2	1.625
High-Pressure Reverse Osmosis	3	2	1	1	1	1.375

Concentration and Crystallization

- Pros
    - Zero-Liquid Discharge
    - Good waste recovery
- Cons
    - High cost

Membrane Distillation

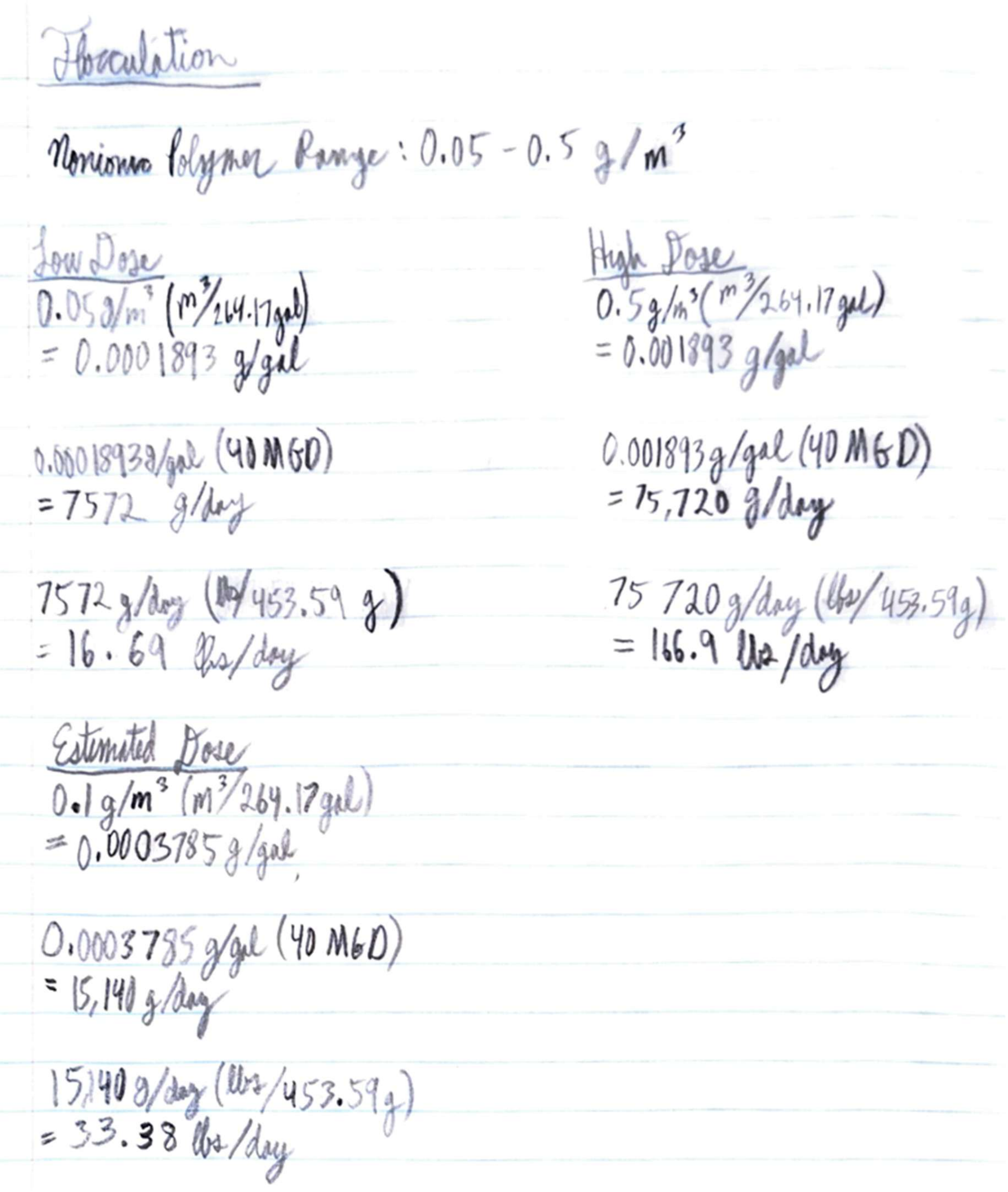
- Pros
    - Reusable Waste
- Cons
    - Membrane fouling
    - Expensive

High-Pressure Reverse Osmosis

- Pros
    - High recovery rate
    - Proven tech
- Cons
    - High energy use
    - Membrane Wear

Appendix C: Treatment Processes Hand Calculations

Flocculation



Flocculation

Nonionic Polymer Range: 0.05 - 0.5 g/m^3

Low Dose

$0.05\ g/m^3\ (m^3/264.17\ gal)$
$= 0.0001893\ g/gal$

$0.0001893\ g/gal\ (40\ MGD)$
$= 7572\ g/day$

$7572\ g/day\ (lbs/453.59\ g)$
$= 16.69\ lbs/day$

High Dose

$0.5\ g/m^3\ (m^3/264.17\ gal)$
$= 0.001893\ g/gal$

$0.001893\ g/gal\ (40\ MGD)$
$= 75,720\ g/day$

$75\,720\ g/day\ (lbs/453.59\ g)$
$= 166.9\ lbs/day$

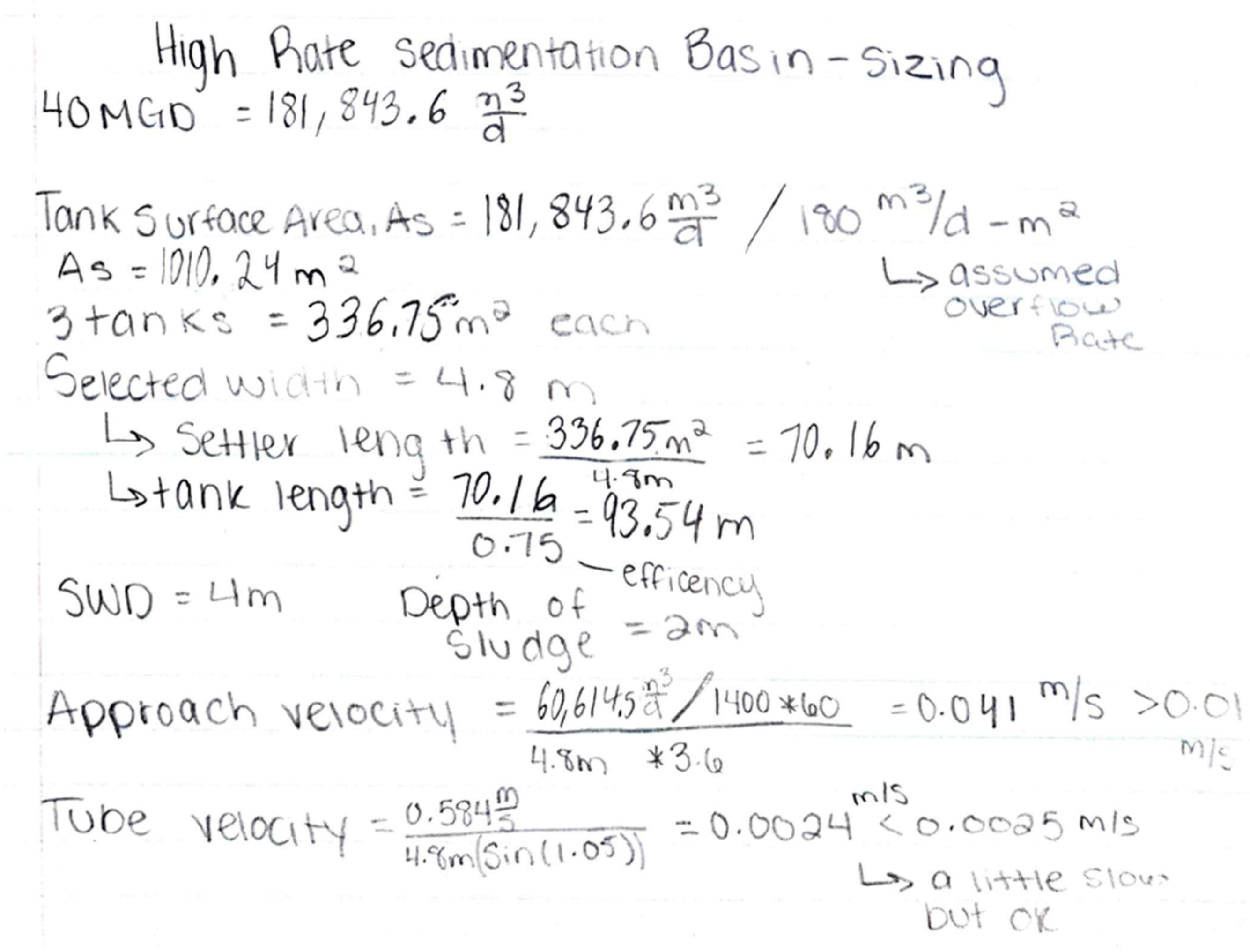
Estimated Dose

$0.1\ g/m^3\ (m^3/264.17\ gal)$
$= 0.0003785\ g/gal$

$0.0003785\ g/gal\ (40\ MGD)$
$= 15,140\ g/day$

$15,140\ g/day\ (lbs/453.59\ g)$
$= 33.38\ lbs/day$

High-Rate Sedimentation



High Rate sedimentation Basin - sizing

$40\,MGD = 181{,}843.6 \frac{m^3}{d}$

Tank Surface Area, As $= 181{,}843.6 \frac{m^3}{d} \,/\, 180\ m^3/d - m^2$ ↳ assumed overflow Rate

$As = 1010.24\ m^2$

3 tanks $= 336.75\ m^2$ each

Selected width $= 4.8\ m$

↳ Settler length $= \frac{336.75\ m^2}{4.8\ m} = 70.16\ m$

↳ tank length $= \frac{70.16}{0.75} = 93.54\ m$ — efficency

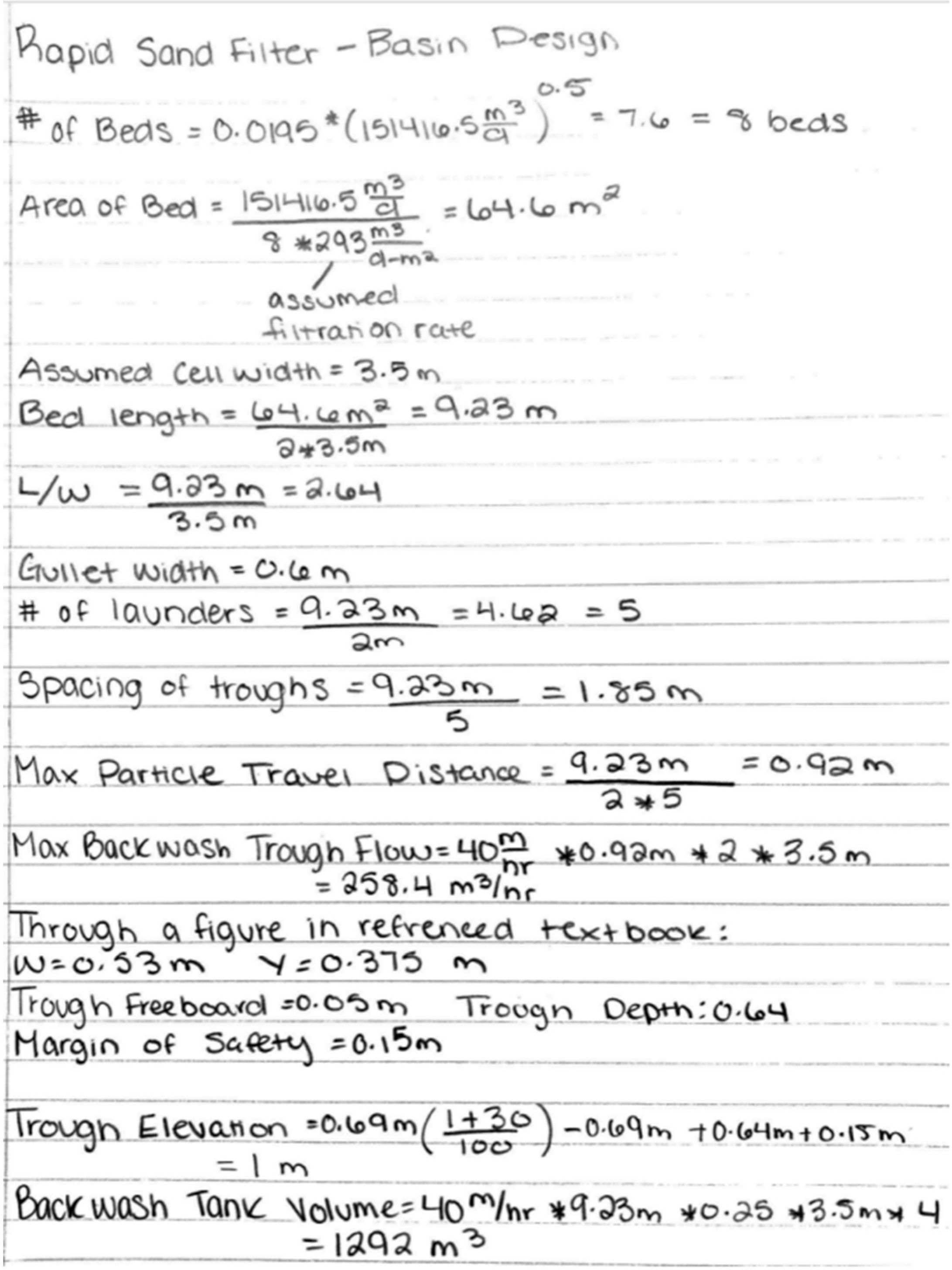
SWD $= 4m$ Depth of sludge $= 2m$

Approach velocity $= \frac{60{,}614.5 \frac{m^3}{d} / 1400 * 60}{4.8m * 3.6} = 0.041\ m/s > 0.01\ m/s$

Tube velocity $= \frac{0.584 \frac{m}{s}}{4.8m(\sin(1.05))} = 0.0024\ m/s < 0.0025\ m/s$

↳ a little slow but OK

Rapid Sand Filter Hand Calculations



Rapid Sand Filter – Basin Design

$\# \text{ of Beds} = 0.0195 * \left(151416.5 \frac{m^3}{d}\right)^{0.5} = 7.6 = 8 \text{ beds}$

$\text{Area of Bed} = \frac{151416.5 \frac{m^3}{d}}{8 * 293 \frac{m^3}{d\text{-}m^2}} = 64.6\ m^2$

assumed filtration rate

Assumed Cell width = 3.5 m

$\text{Bed length} = \frac{64.6 m^2}{2 * 3.5 m} = 9.23\ m$

$L/W = \frac{9.23 m}{3.5 m} = 2.64$

Gullet width = 0.6 m

$\# \text{ of launders} = \frac{9.23 m}{2 m} = 4.62 = 5$

$\text{Spacing of troughs} = \frac{9.23 m}{5} = 1.85\ m$

$\text{Max Particle Travel Distance} = \frac{9.23 m}{2 * 5} = 0.92\ m$

$\text{Max Backwash Trough Flow} = 40 \frac{m}{hr} * 0.92 m * 2 * 3.5 m$
$= 258.4\ m^3/hr$

Through a figure in refreneced textbook:

W = 0.53 m   Y = 0.375 m

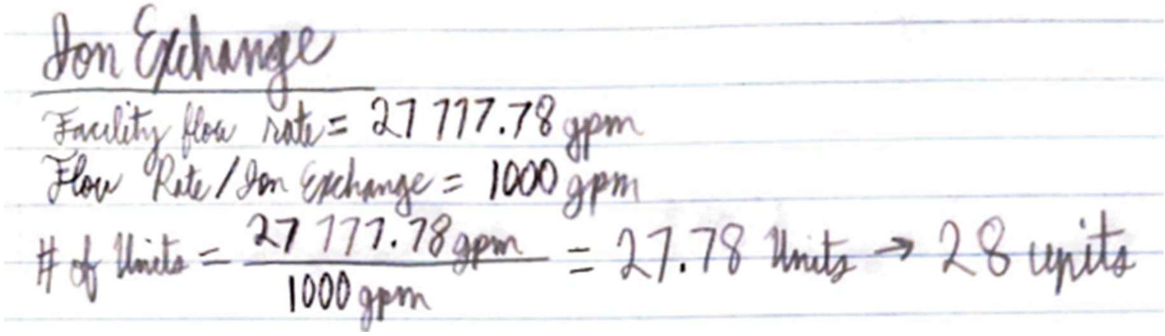
Trough Freeboard = 0.05 m   Trough Depth: 0.64

Margin of Safety = 0.15m

$\text{Trough Elevation} = 0.69 m \left(\frac{1+30}{100}\right) - 0.69 m + 0.64 m + 0.15 m$
$= 1\ m$

$\text{Backwash Tank Volume} = 40\ m/hr * 9.23 m * 0.25 * 3.5 m * 4$
$= 1292\ m^3$

Ion Exchange



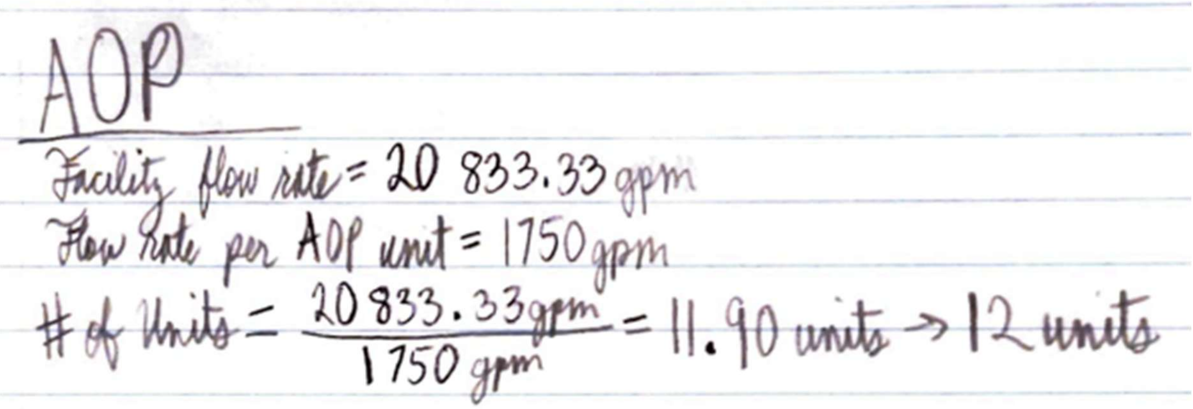
Ion Exchange

Facility flow rate = 27 777.78 gpm

Flow Rate / Ion Exchange = 1000 gpm

# of Units = $\frac{27\,777.78 \text{ gpm}}{1000 \text{ gpm}}$ = 27.78 Units → 28 units

AOP



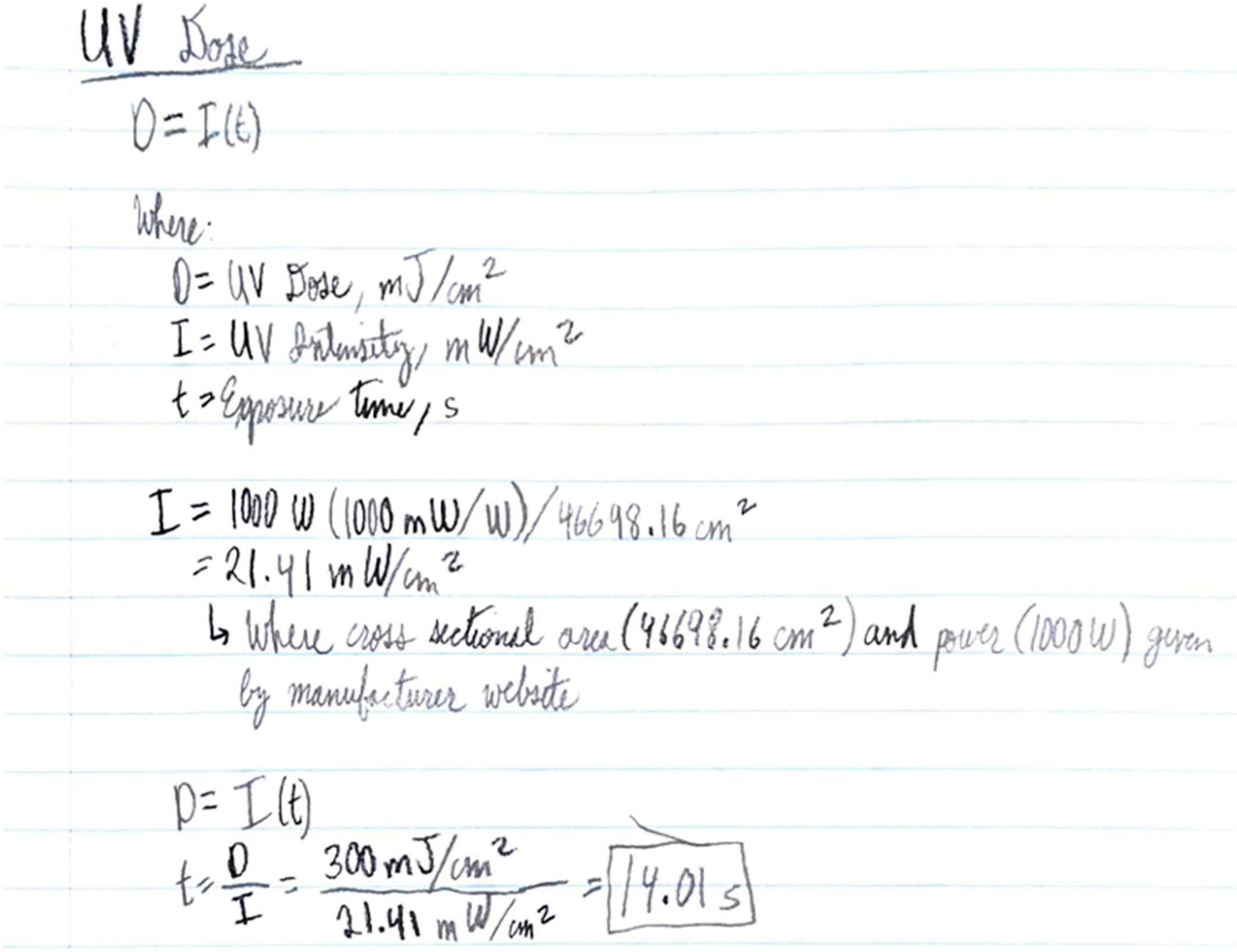
AOP

Facility flow rate = 20 833.33 gpm

Flow Rate per AOP unit = 1750 gpm

# of Units = $\frac{20\,833.33 \text{ gpm}}{1750 \text{ gpm}}$ = 11.90 units → 12 units

UV Dose



UV Dose

$D = I(t)$

Where:

D = UV Dose, mJ/cm^2
I = UV Intensity, mW/cm^2
t = Exposure time, s

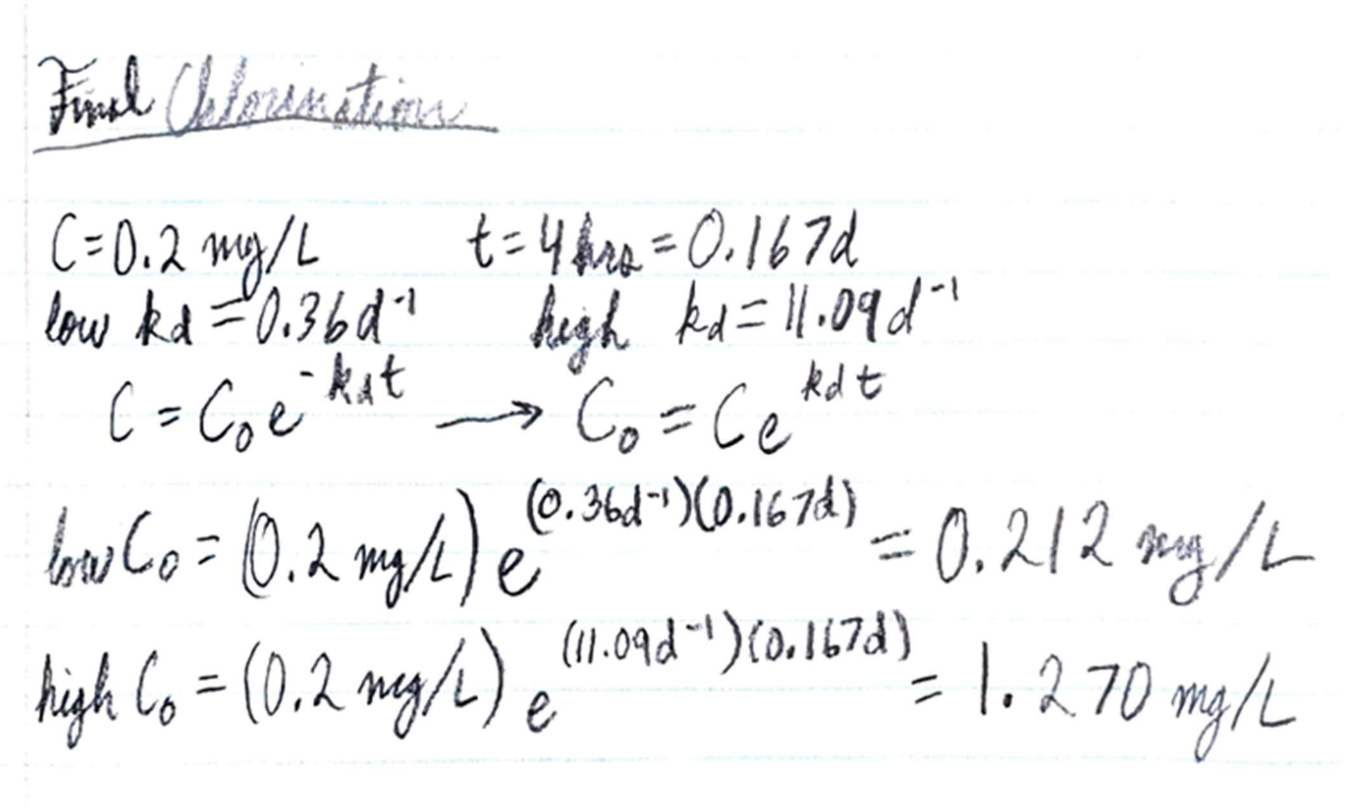
$I = 1000\ W\ (1000\ mW/W)/46698.16\ cm^2$
$= 21.41\ mW/cm^2$

↳ Where cross sectional area ($46698.16\ cm^2$) and power (1000 W) given by manufacturer website

$D = I(t)$

$$t = \frac{D}{I} = \frac{300\ mJ/cm^2}{21.41\ mW/cm^2} = \boxed{14.01\ s}$$

Final Chlorination



Final Chlorination

$C = 0.2\ mg/L$ $t = 4\ hrs = 0.167d$

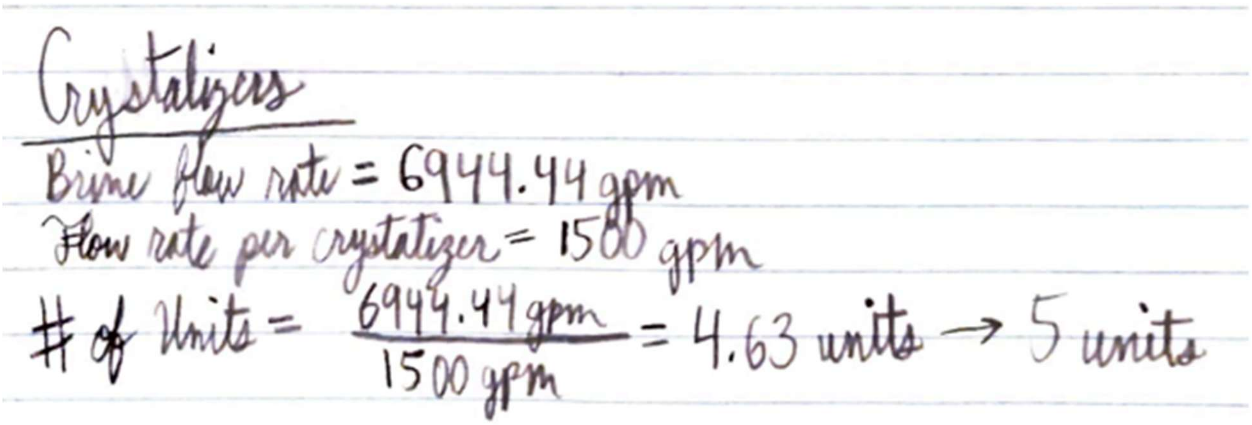
low $k_d = 0.36d^{-1}$ high $k_d = 11.09d^{-1}$

$$C = C_0 e^{-k_d t} \longrightarrow C_0 = C e^{k_d t}$$

$$\text{low } C_0 = (0.2\ mg/L)\, e^{(0.36d^{-1})(0.167d)} = 0.212\ mg/L$$

$$\text{high } C_0 = (0.2\ mg/L)\, e^{(11.09d^{-1})(0.167d)} = 1.270\ mg/L$$

Brine Management/Crystallizers



Crystalizers

Brine flow rate = 6944.44 gpm

Flow rate per crystalizer = 1500 gpm

$$\# \text{ of Units} = \frac{6944.44\ gpm}{1500\ gpm} = 4.63 \text{ units} \rightarrow 5 \text{ units}$$

Appendix D: Proposed Pipe Material & Wet Well Diameter Decision Matrices

Table 12-1: Proposed Wet Well Volume Decision Matrix

Decision Matrix Pipe Material					
Alternative	Weight	PCCP	DIP	Steel	HDPE
Hydraulic Performance	20%	4	4	4	3
Structural Strength	20%	5	4	4	2
Availability at 35in D.	25%	5	4	4	2
Sustainability at Length	25%	5	3	3	2
Corrosion Resistance	10%	4	3	3	5
Total Score		4.7	3.65	3.65	2.5

Decision Matrix Wet Well Vol.					
Alternative	Weight	25'D	30'D	35'D	40'D
Footprint	30%	5	4	3	2
Effective Volume Control	30%	5	4	4	2
Constructability (excavation)	20%	2	3	4	5
Safety	20%	4	4	4	5
Total Score		4.2	3.8	3.7	3.2

Appendix E: Hydraulic Calculations

Table 12-2. Hydraulic Profile Summary

Parameter	Value
Hydraulic Grade Line, HGL (ft)	961.00
Energy Grade Line, EGL (ft)	961.64
Velocity Head (ft)	0.643

Table 12-3. MGD Demand Required vs Proposed Daily Values

Parameter	Value
Population Served	300,000
Per Capita Demand (gpcd)	100
Base Demand (MGD)	30
Peak Hour Factor	2.0
Peak Day Factor	4.8
Actual Daily Demand (gpm)	20,833
Minimum Daily Demand (gpm)	10,417
Maximum Daily Demand (gpm)	48,611
Proposed Design Flow (gpm)	27,778
Total Daily Volume (gal/day)	30,000,000

Table 12-4. Daily Demand Distribution (40 MGD Design)

Flow (gpm)	Flow (MGD)	Daily Volume (gal/day)	Population Served	Per Capita (gpcd)	Losses (MGD)	Operating Time (hr)
25,000	36	36,000,000	300,000	120	6	8.64
20,833	30	30,000,000	300,000	100	0	10.36
27,778	40	40,000,000	300,000	133	10	7.77

Table 12-5. Equalization Tank Design Summary

Diurnal Flow (MGD)	ΔQ (MGD)	4-hr Peak Flow (MGD)	Freeboard (ft)	Required Volume (gal)	Tank Diameter (ft)	Tank Depth (ft)
70	34	6.2	3	28,863	179	30

Table 12-6. Wet Well Hydraulic and Storage Design (35 ft Diameter)

Flow (MGD)	Q (gpm)	Area (ft²)	Volume per ft (gal/ft)	Δh (ft)	Buffer Volume (gal)	Total Volume (gal)	Wet Well Depth (ft)
30	20,833	962	7,197	8.68	62,500	187,122	26
36	25,000	962	7,197	10.42	75,000	187,122	26
40 (Design)	27,778	962	7,197	11.58	83,333	187,122	26
70	48,611	962	7,197	20.26	145,833	187,122	26

Table 12-7. Combined Pipe Hydraulic Parameters and headloss Analysis (42-in DIP)

Flow (MGD)	Q (gpm)	Q (cfs)	A (ft²)	V (ft/s)	R_h (ft)	S (ft/ft)	h_l (ft)
0	0	0.00	9.62	0.00	0.88	0.0000	0.00
10	10,000	22.28	9.62	2.32	0.88	0.0005	5.07
20.8	20,833	46.42	9.62	4.82	0.88	0.0018	19.74
36	25,000	55.70	9.62	5.79	0.88	0.0025	27.68
40 (Design)	27,778	61.89	9.62	6.43	0.88	0.0031	33.64
43	30,000	66.84	9.62	6.95	0.88	0.0036	38.79
57.5	40,000	89.13	9.62	9.26	0.88	0.0061	66.08
70	48,611	108.31	9.62	11.26	0.88	0.0090	94.82

Table 12-8. Pipe Hydraulic Parameters per Pump

Q (gpm/pump)	Q (cfs)	A (ft^2)	V (ft/s)	R_h (ft)	S (ft/ft)
0	0.00	9.62	0.00	0.88	0.0000
2,500	22.28	9.62	2.32	0.88	0.0004
6,250	55.70	9.62	5.79	0.88	0.0026
6,945	61.89	9.62	6.43	0.88	0.0031
10,000	89.13	9.62	9.26	0.88	0.0053
10,417	92.84	9.62	9.65	0.88	0.0057
12,000	106.95	9.62	11.12	0.88	0.0086
14,000	124.78	9.62	12.97	0.88	0.0114
16,000	142.60	9.62	14.82	0.88	0.0146

Table 12-9. Headloss and TDH Calculations per Pump

h_l (ft)	h_{minor} (ft)	TDH (ft)	h_{static} (ft)	h_{major} (ft)
0.00	0.00	24.61	24.61	0.00
4.38	0.50	29.49	24.61	4.38
23.90	3.12	51.63	24.61	23.90
33.64	3.86	62.11	24.61	33.64
57.07	7.99	89.67	24.61	57.07
61.55	8.68	94.84	24.61	61.55
92.68	11.52	128.80	24.61	92.68
123.21	15.67	163.49	24.61	123.21
157.85	20.47	202.93	24.61	157.85

$$h_f = 10.67 \cdot \frac{L \cdot Q^{1.852}}{C^{1.852} \cdot \ D^{4.87}}$$

$h_f = \mathit{Friction\ Head}$

$L = \text{Length of pipe}$

$Q = \text{Total Flow}$

$D = \mathit{Diameter\ of\ Pipe}$

$C = \mathit{Friction\ Coeficient}\ (120)$

Equation 12-1:Hazen Williams Friction Head Eqn.

$$h_s = z_2 - z_1$$

$h_s = \mathit{Static\ Head}$

$z_2 = \mathit{End\ Elevation}$

$z_1 = \mathit{Start\ Elevation}$

Equation 12-2:Static Head Eqn.

$$h_p = (z_2 - z_1) + h_f + h_m$$

$h_p = \mathit{Pump\ Head}$

$z_2 - z_1 = \mathit{Static\ Head}$

$h_m = \mathit{Minor\ Head\ Loss}$

Equation 12-3:Pump head Eqn.

$$TDH = h_s + h_f + h_m$$

$\text{TDH} = \text{Total Dynamic Head}$

$\text{h}_\text{p} = Pump\ Head$

$\text{h}_\text{s} = Static\ Head$

$\text{h}_\text{m} = Minor\ Head\ Loss$

Equation 12-4:Total Dynamic Head Eqn.

$\text{Veff} = \Delta Q \cdot \Delta t$

Veff = Volume Effective

ΔQ = Change in flow

Δt = Change in time

Equation 12-5: Effective Wet Well Volume Eqn.

$$NPSHA = \frac{P_{atm}}{\gamma} + z_s - \frac{P_v}{\gamma} - h_{f,s}$$

Equation 12-6:Net Positive Suction Head Available (NPSHA) Eqn.

$HGL = z + \frac{p}{\gamma}$

$HGL = \text{Hydraulic Grade Line}$

$z = \text{Elevation}$

$p = \text{Pressure}$

$\gamma = Specific\ Gravity\ of\ Water\ (62.4)$

Equation 12-7: HGL Eqn.

$$EGL = HGL + \frac{V^2}{2g}$$

$$EGL = Energy\ Grade\ Line$$

$$HGL = Hydraulic\ Grade\ Line$$

$$V = Velocity$$

Equation 12-8:EGL Eqn.

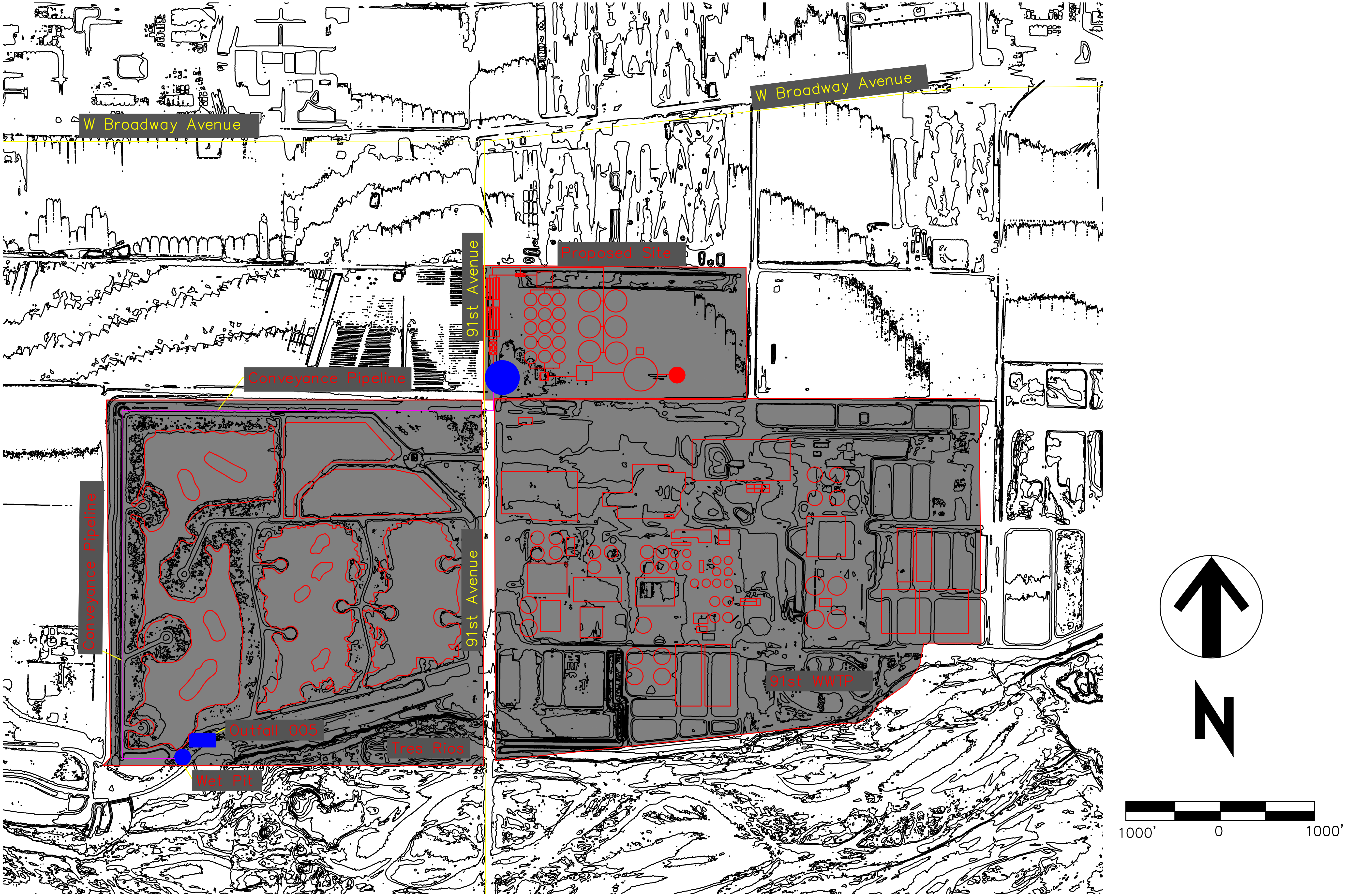
$Q = V * A$

$$Q = Flow$$

$$V = Velocity$$

$$A = Area\ of\ Pipe$$

Appendix F: Site Layout – Overall Site Layout



Description:

Site Layout including:
- Proposed Site
- Existing Site
- Tres Rios Wetlands

Location:

91st Avenue WWTP
– Tolleson, Arizona

Copperhead Engineering

Reviews:

Project:

91st Avenue –
Advanced Water
Purification Facility

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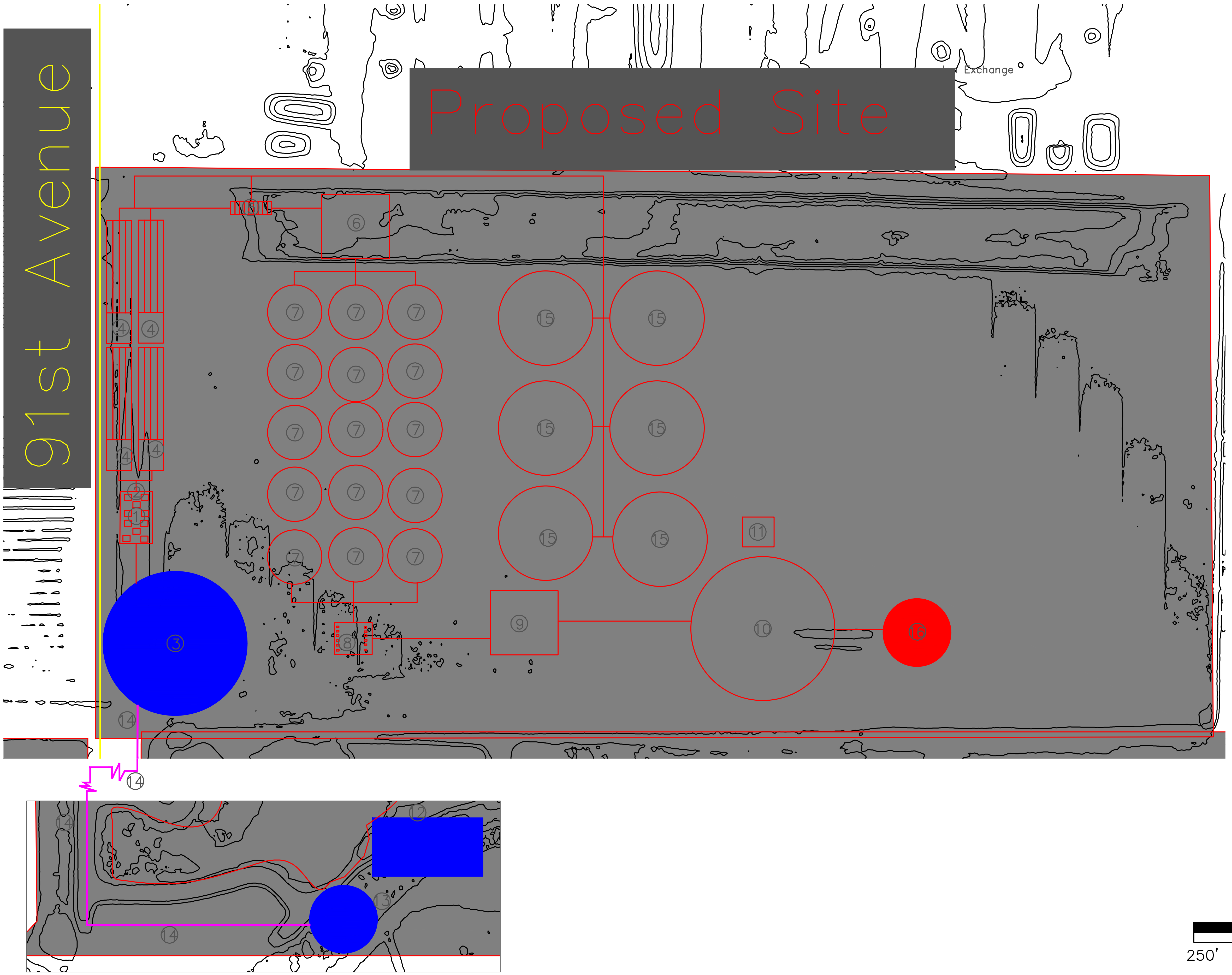
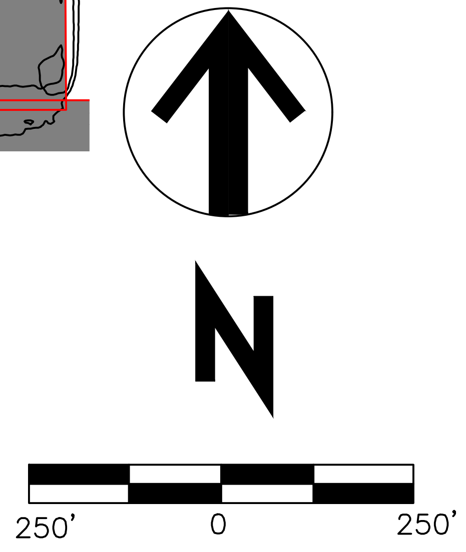
Appendix G: Site Layout – Proposed Site



Legend

①	Curved Screen
②	Headworks
③	Equalization Storage Tank
④	High Rate Sedimentation Basin
⑤	Rapid Sand Filter w/GAG
⑥	Microfiltration
⑦	Ion Exchange
⑧	Advanced Oxidation/ UV+h202
⑨	Final Chlorination
⑩	Final Storage Tank/Distribution
⑪	Electrical & Control Building
⑫	Outfall 005
⑬	Wet Pit
⑭	Proposed Conveyance Pipeline
⑮	Crystallization Basin
⑯	Final Distribution Pump



Description:
Proposed Site Layout

Location:
91st Avenue WWTP
– Tolleson, Arizona

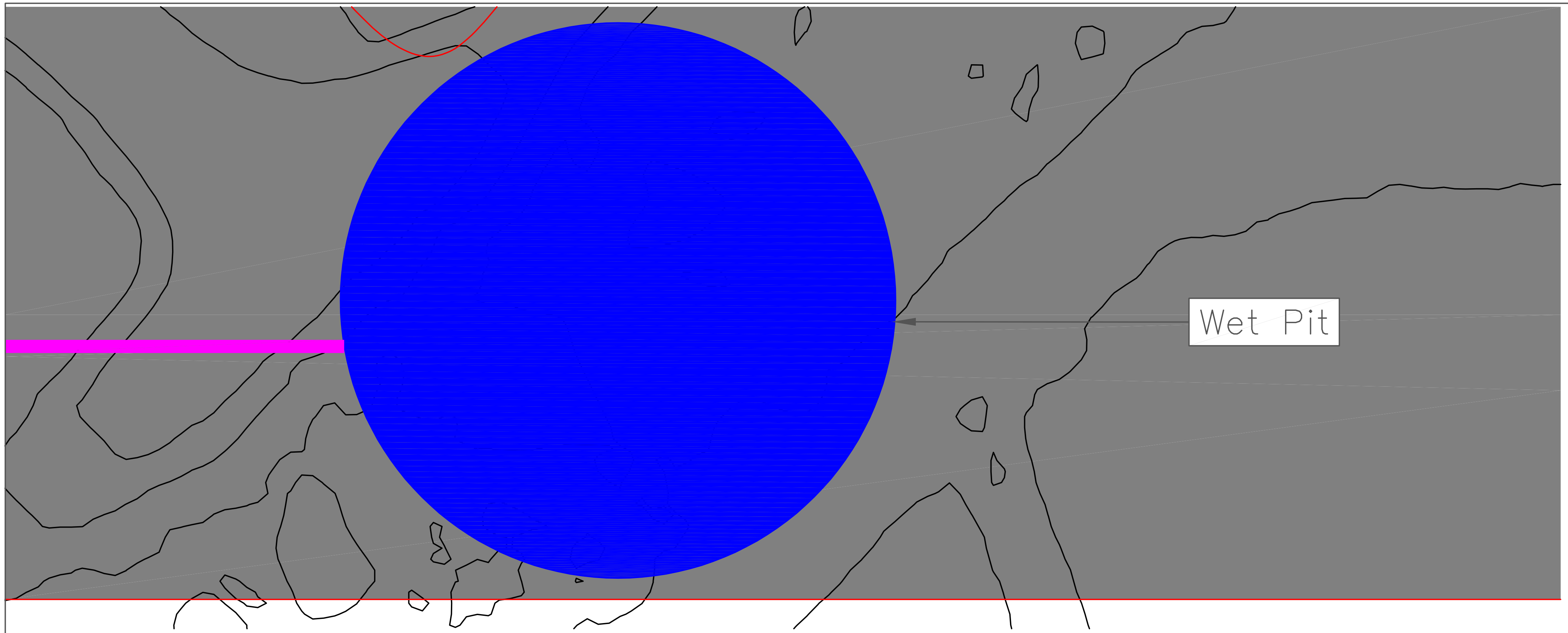
Copperhead Engineering

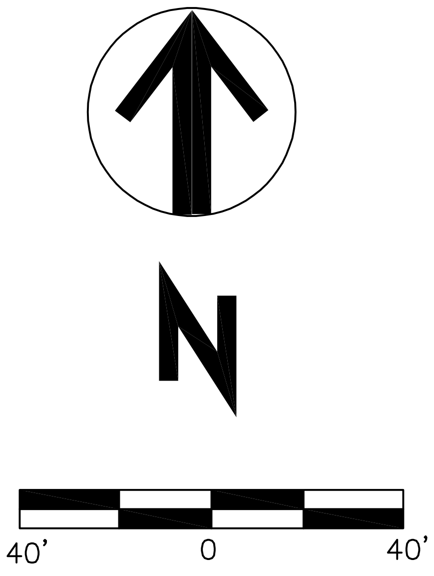
Reviews:

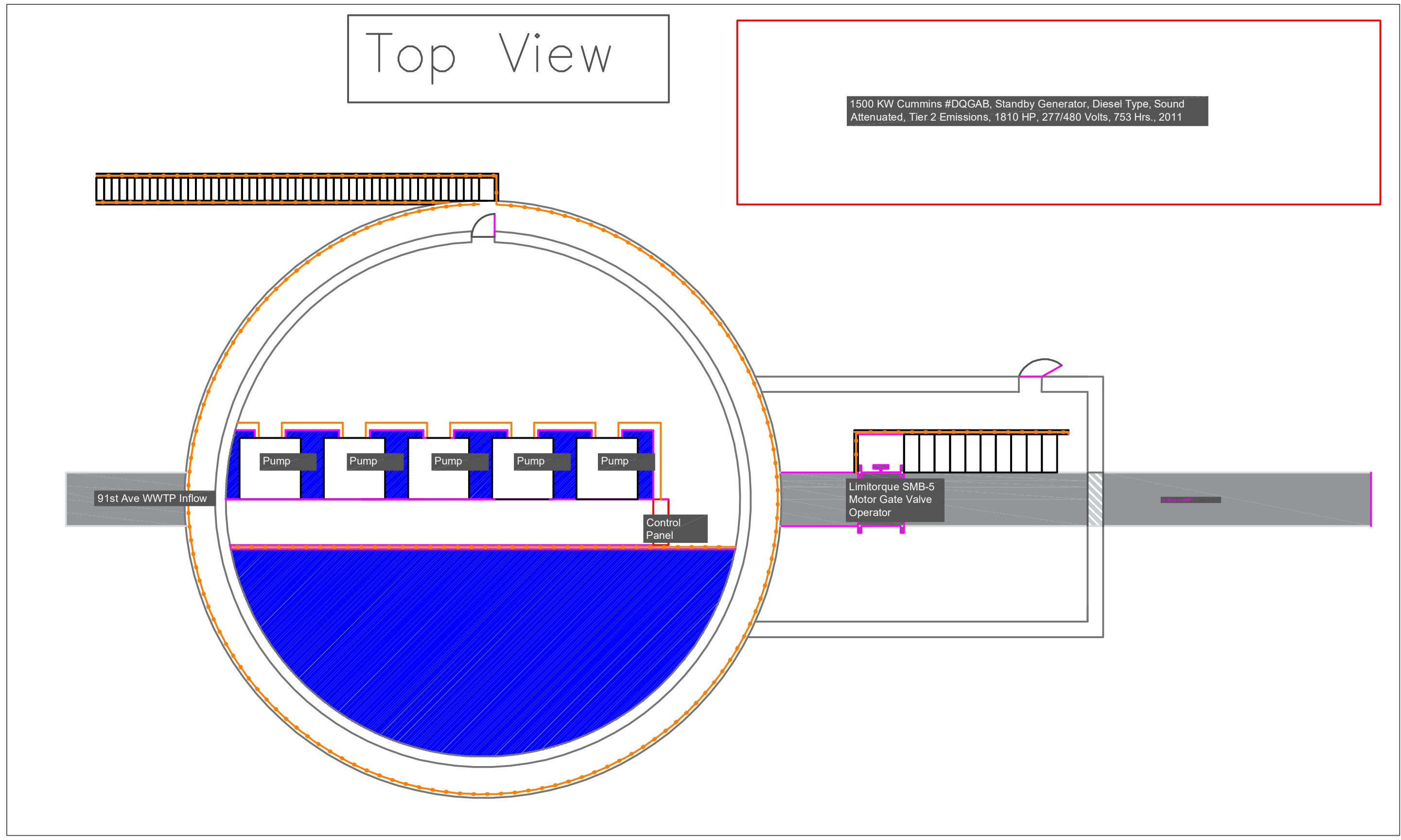
Project:
91st Avenue –
Advanced Water
Purification Facility

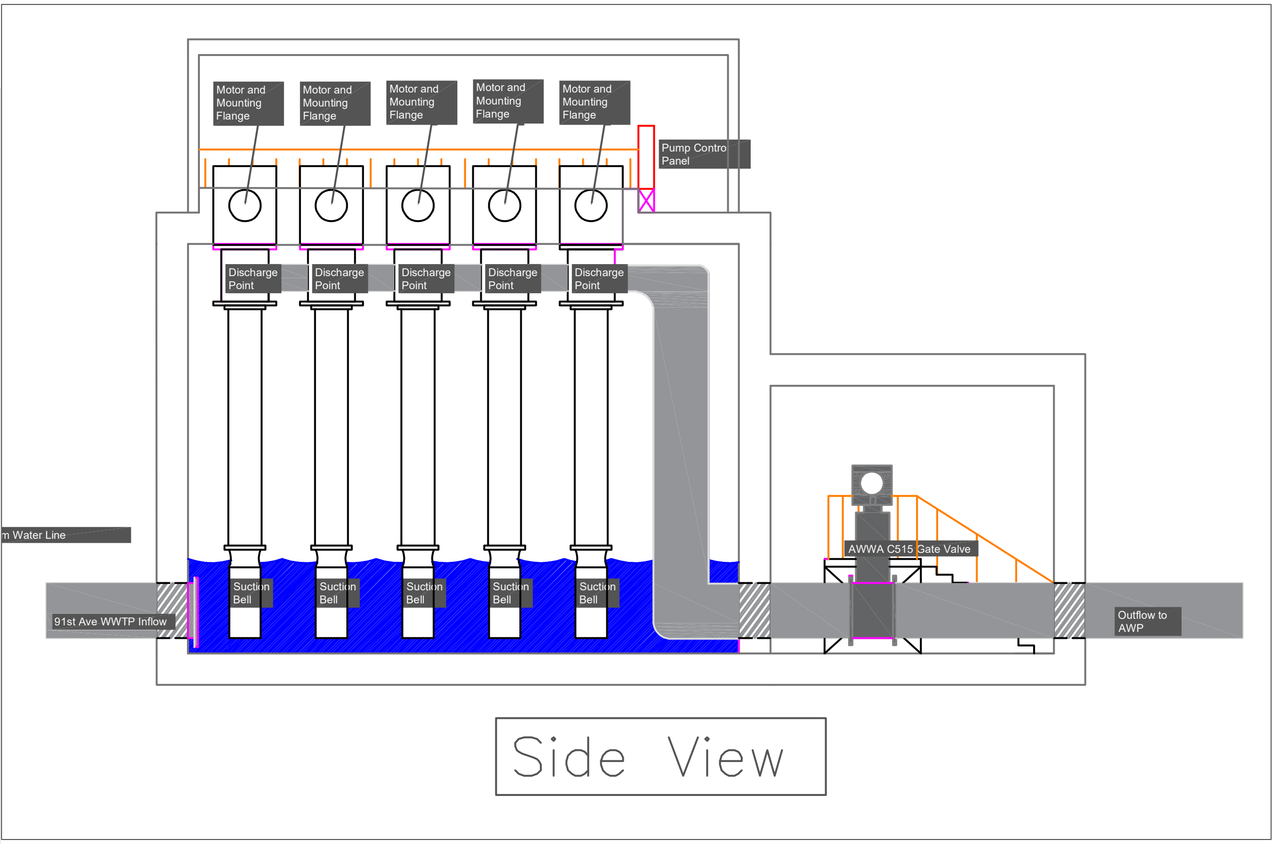
Page Number
1 OF 3

Appendix H: Wet Pit Detail






Top View


Side View

Description:

Wet Well Detail
- Top View
- Side View
- Wet Pit Location

Location:

91st Avenue WWTP
– Tolleson, Arizona

Copperhead Engineering

Reviews:

Project:

91st Avenue –
Advanced Water
Purification Facility

Page Number

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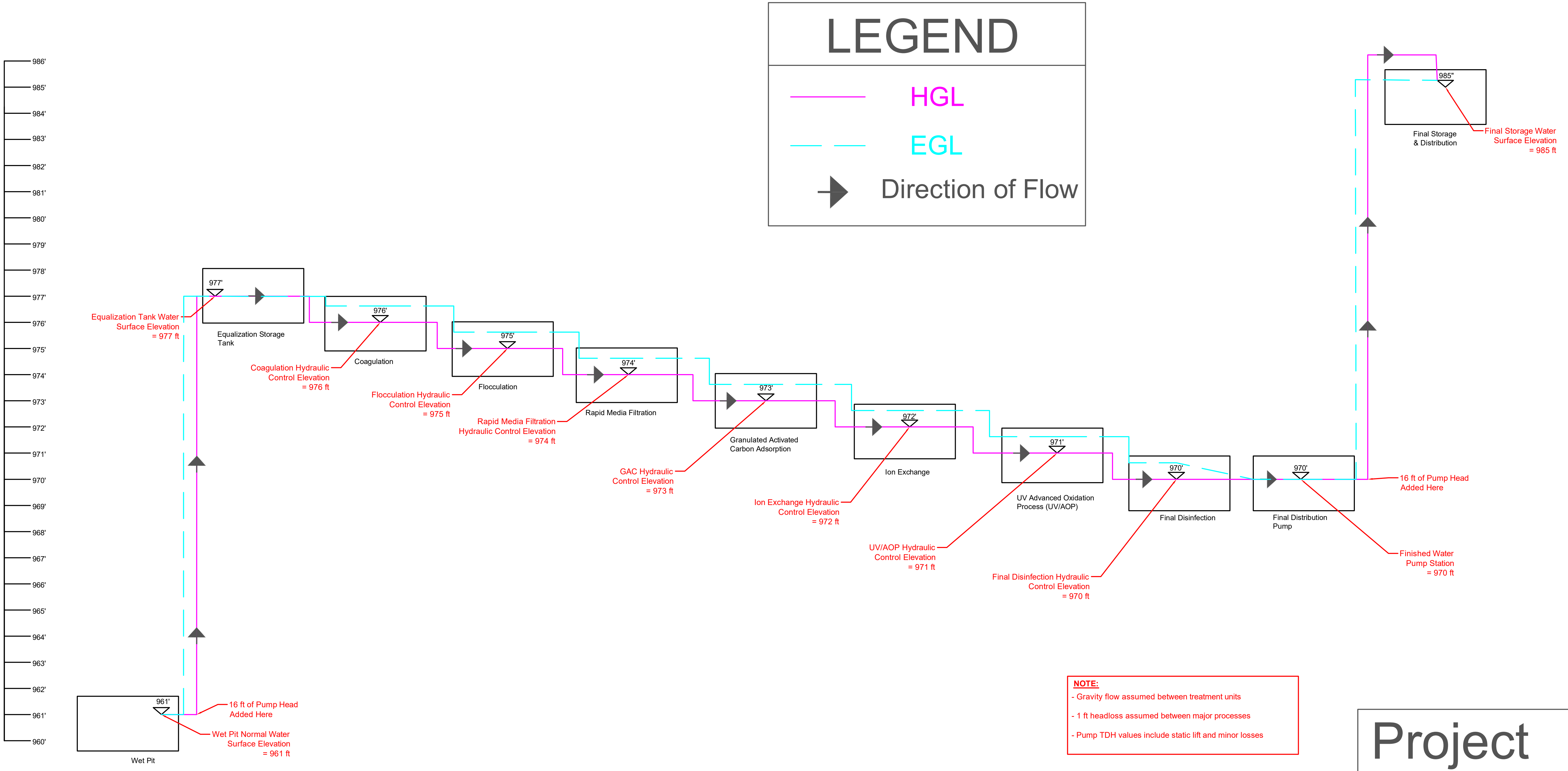
Appendix I: Hydraulic Profile



NOTE:

- Gravity flow assumed between treatment units
- 1 ft headloss assumed between major processes
- Pump TDH values include static lift and minor losses

Copperhead Engineering

COPPERHEAD
ENGINEERS

Description:
Hydraulic profile, showing HGL and EGL

Reviews:

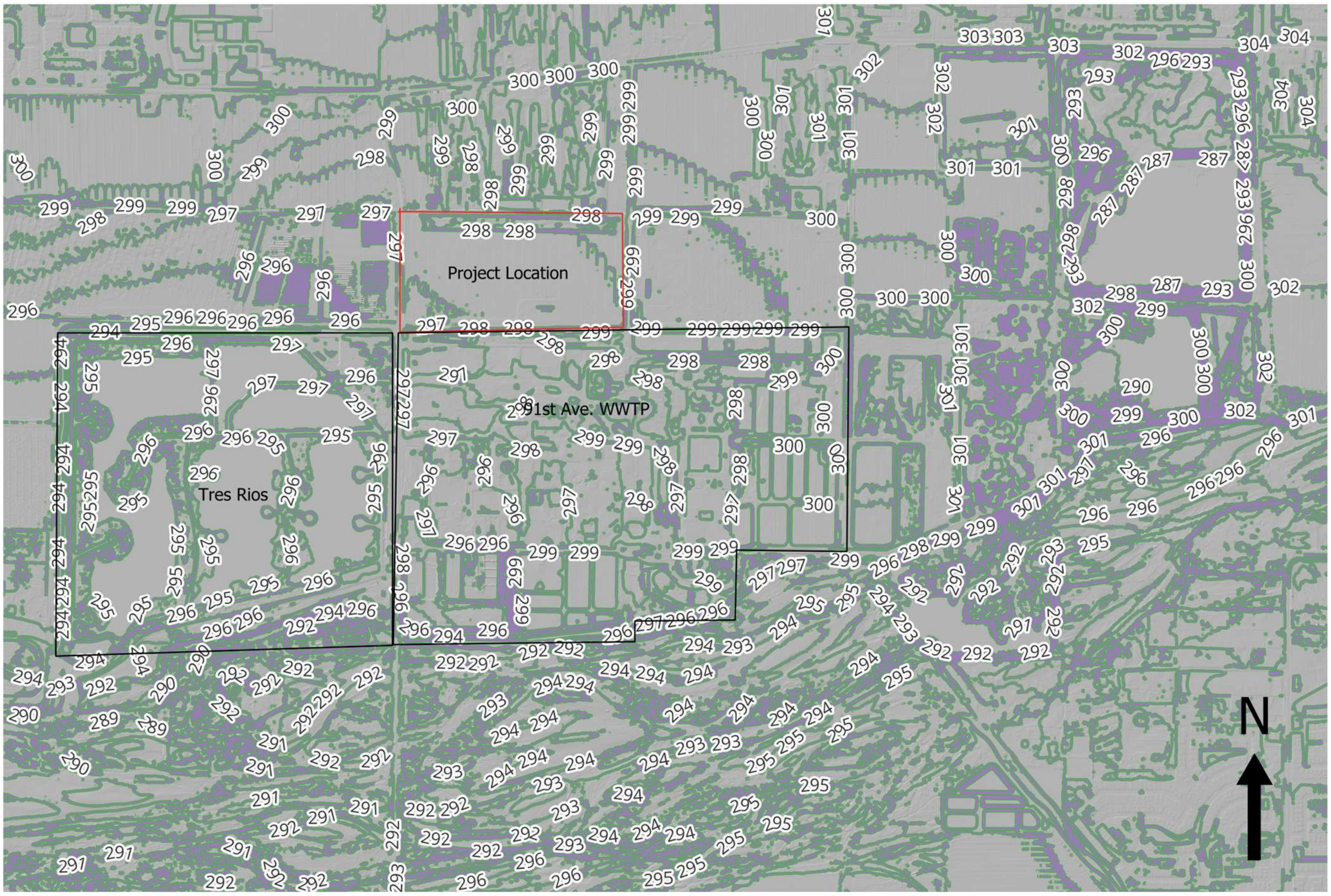
Location:
91st Avenue, Tolleson, AZ
85353

Project

91st Avenue Proposed Advanced Water Purification Facility

PAGE 1 of 1

Appendix J: AWPF Topography Map


Project Location
91st Ave. WWTP
Tres Rios
N

Appendix K: Goulds Pump Curve

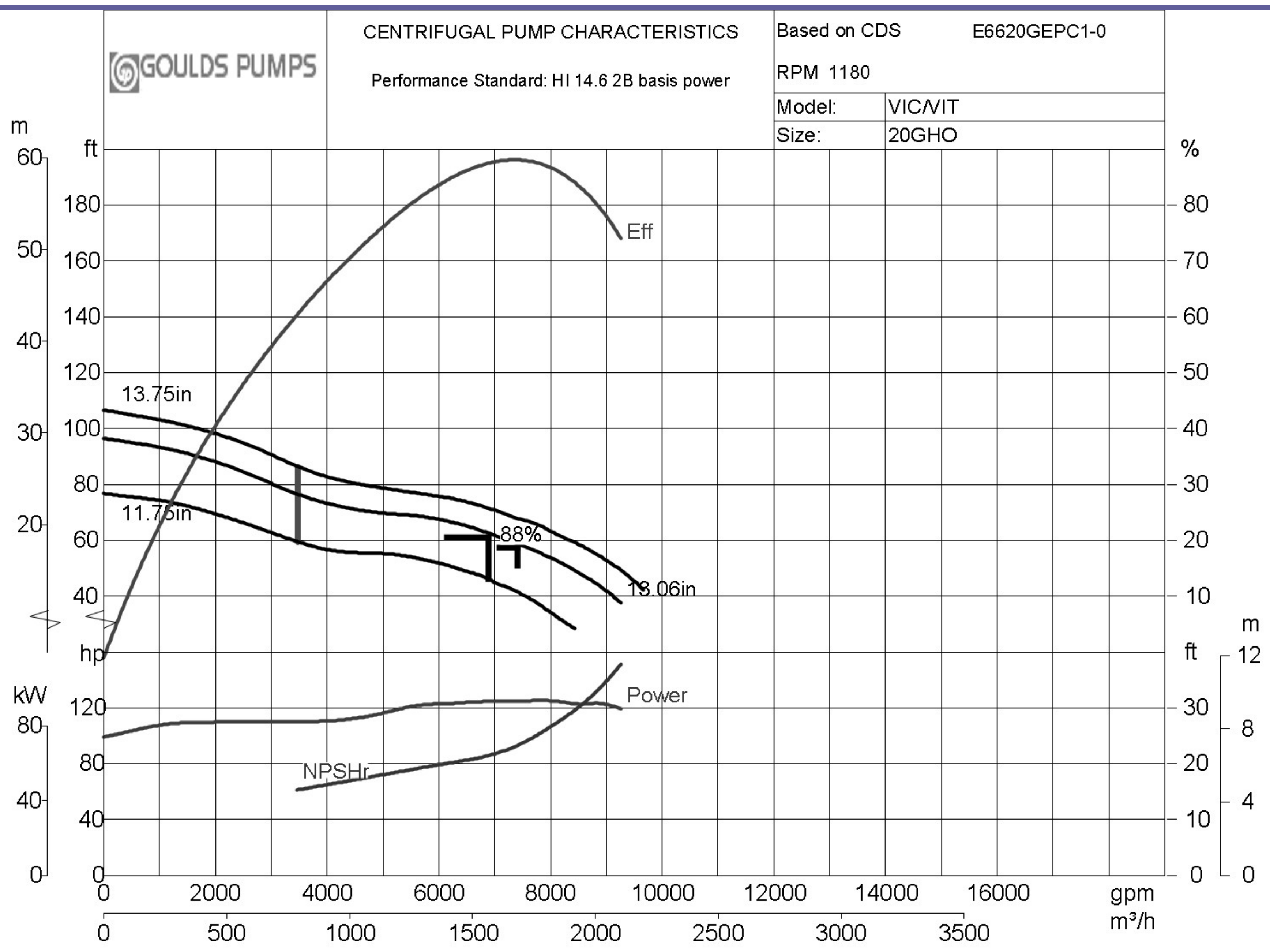
Model: VIT **Size: 20GHO** **Group:** **60Hz** **RPM: 1180** **Stages: 1**

Customer :	cene 486	Issued By :	Moses Marsico	Date :	03/23/2026
Item No. :	ITEM 001	Quotation No. :	MM26-03-23 01	Service :	

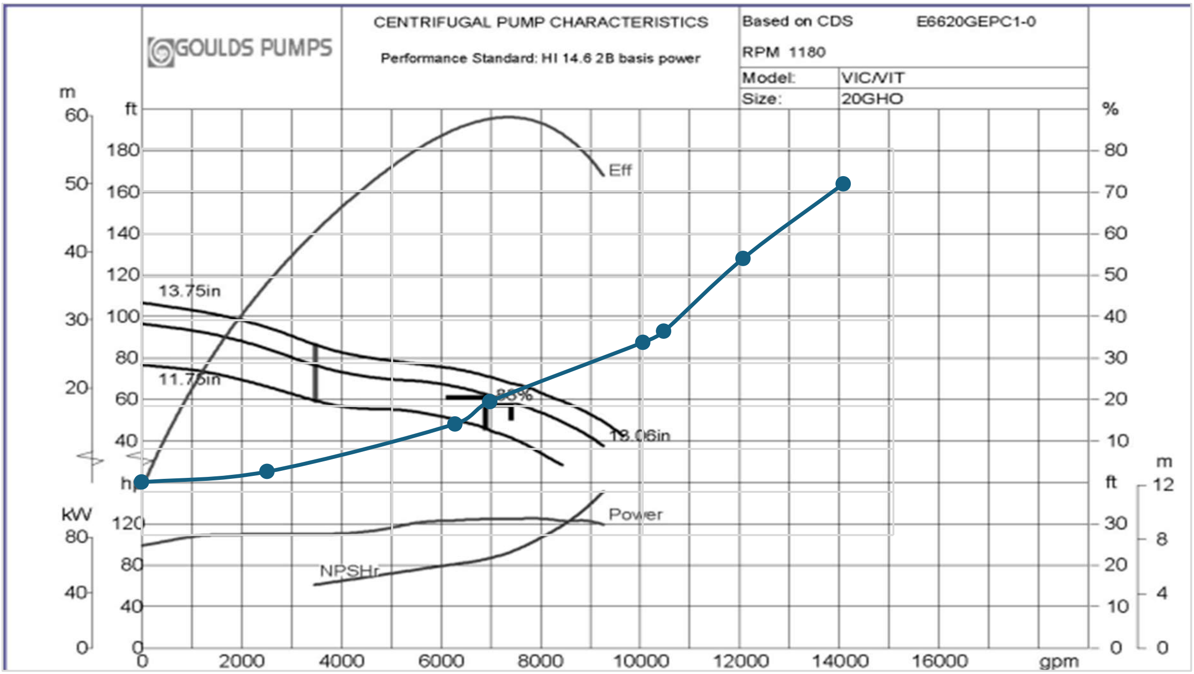
Operating Conditions / Pump Performance

Operating Conditions		Pump Performance			
Liquid:	Water	Bowl Efficiency:	87.3 %	Suction Specific Speed:	9,754 gpm(US) ft
Temp.:	70.0 deg F	Min. Hydraulic Flow:	3,479.9 gpm	Imp. Dia. First 1 Stg(s):	13.06 in
Flow:	6,944.0 gpm	Rated Power:	124.7 hp	Non-Overloading Power:	124.9hp
TDH:	62.1 ft	NPSHr:	21.8 ft	Shut off Head:	96.5 ft
NPSHa:		Max. Solids Size:	1.5600 in	Susp. Solids%:	
S.G./Visc.:	1.000/1.000 cp				

Notes:
1. The Mechanical seal increased drag effect on power and efficiency is not included.
2. Magnetic drive eddy current on power and efficiency is not included.
3. Curve shown is at ambient temperature conditions.
4. Non Overloading power does not reflect v-belt/gear losses.



Appendix L: Pump & System Curve Design Calculation



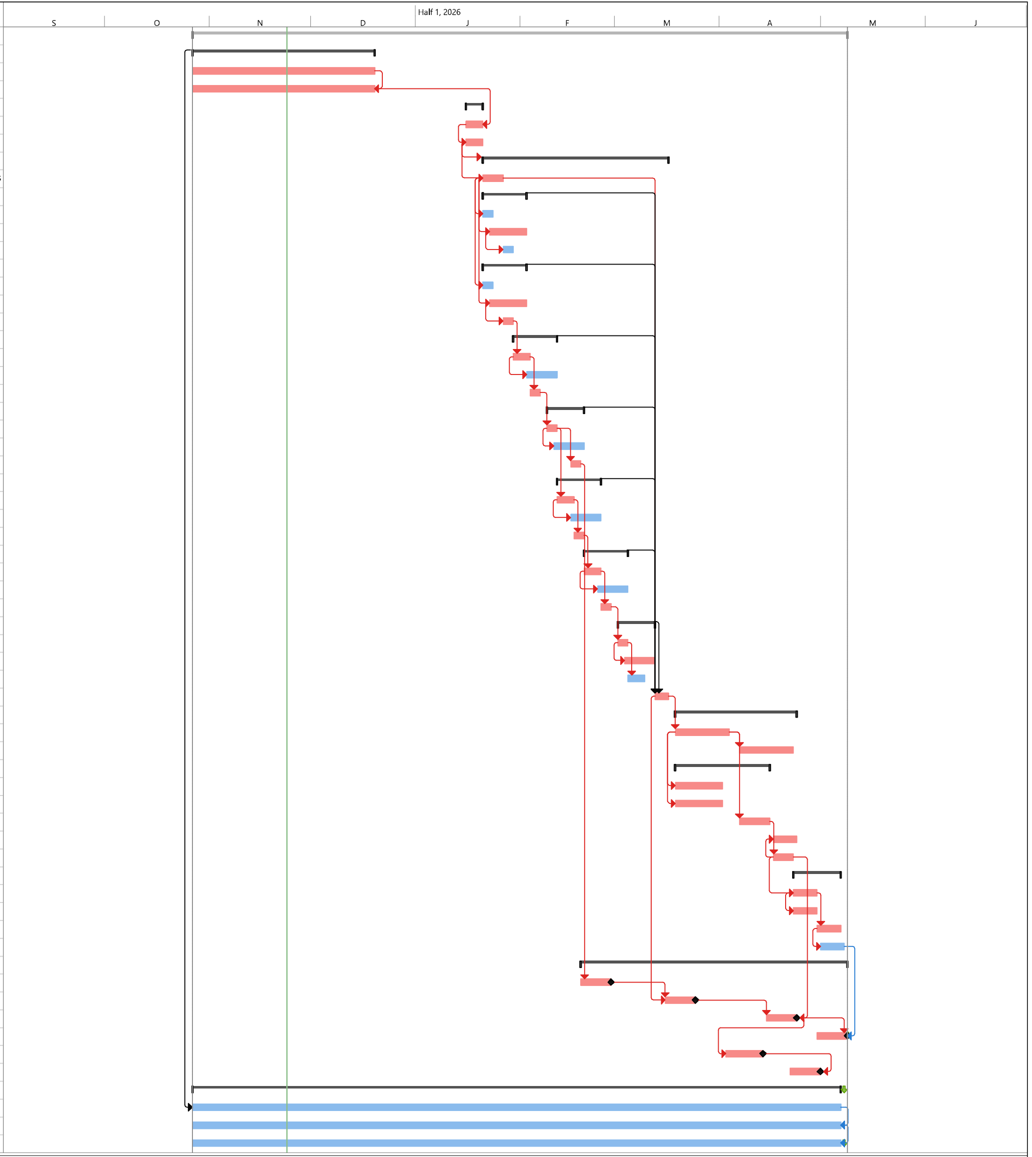
Graph 1. Pump & System Curve Design Calculation

Appendix M: Cost Analysis

Items: Lift Station				
Item Description	Unit	Quantity	Unit Price	Extended Amount
Sitework	L.SUM	1	$10,615	$10,615
Hydraulics (piping+pumps)	L.SUM	1	$11,984,400	$11,984,400
Lift Station Concrete Structure	L. SUM	1	$449,500	$449,500
Additional Contracting	L. SUM	1	$1,025,266	$1,025,266
	Total 4.0 Item:			$13,469,781
Items: Proposed Site				
Sitework	L. SUM	1	$7,513,160	$7,513,160
Hydraulics (20"DIP)	L. SUM	1	$1,592,500	$1,592,500
Curved Screen	EACH	10	$61,500	$615,000
Headworks	EACH	2	$6,000,000	$12,000,000
Equalization Tank	L.SUM	1	$12,000,000	$12,000,000
High Rate Sedimentation Basin	EACH	4	$20,000,000	$80,000,000
Rapid Sand Filter	L.SUM	1	$15,000,000	$15,000,000
Microfiltration	L.SUM	1	$50,000,000	$50,000,000
Ion Exchange	EACH	15	$300,000	$4,500,000
Advanced Oxidation	L.SUM	1	$35,000,000	$35,000,000
Crystallization Tank	EACH	6	$7,000,000	$42,000,000
Final Chlorination	L.SUM	1	$4,000,000	$4,000,000
Final Storage Tank	L.SUM	1	$13,000,000	$13,000,000
Final Distribution Pump	L.SUM	1	$8,000,000	$8,000,000
Electrical Control	L.SUM	1	$15,000,000	$15,000,000
Additional Contracting	L.SUM	1	$15,588,533	$15,588,533
	Total Item:			$315,809,193
Total Item:				$412,178,974

Appendix N: Proposed Schedule

ID	ℹ	Task Mode	Task Name	Duration	Start	Finish	Predecessors	Successors
0			**91st Avenue Regional Advanced Water Purification Facility Project**	**140 days**	**Mon 10/27/25**	**Fri 5/8/26**		
1			**Task 1.0: Research Preparation**	**40 days**	**Mon 10/27/25**	**Fri 12/19/25**		**60SS**
2			Task 1.1 WEF Application	40 days	Mon 10/27/25	Fri 12/19/25		3FF
3			Task 1.2 Regulation Research	40 days	Mon 10/27/25	Fri 12/19/25	2FF	5FF+22 days
4			**Task 2.0: Site Assessment**	**3 days**	**Fri 1/16/26**	**Tue 1/20/26**		
5			Task 2.1 Site Visit	3 days	Fri 1/16/26	Tue 1/20/26	3FF+22 days	6SS
6			Task 2.2 Data Analysis	3 days	Fri 1/16/26	Tue 1/20/26	5SS	7SS+2 days,8SS+3 days
7			**Task 3.0: Treatment Selection Process**	**39 days**	**Wed 1/21/26**	**Mon 3/16/26**	**6SS+2 days**	
8			Task 3.1 Determine Plant Requirements	4 days	Wed 1/21/26	Mon 1/26/26	6SS+3 days	10SS,11SS+2 days,37,14SS,15SS+2 days
9			**Task 3.2 Coagulation/Flocculation**	**9 days**	**Wed 1/21/26**	**Mon 2/2/26**		**37**
10			Task 3.2.1 Define Criteria	3 days	Wed 1/21/26	Fri 1/23/26	8SS	
11			Task 3.2.2 Treatment Alternatives	7 days	Fri 1/23/26	Mon 2/2/26	8SS+2 days	12SS+2 days
12			Task 3.2.3 Select Best Alternatives	3 days	Tue 1/27/26	Thu 1/29/26	11SS+2 days	
13			**Task 3.3 Sedimentation**	**9 days**	**Wed 1/21/26**	**Mon 2/2/26**		**37**
14			Task 3.3.1 Define Criteria	3 days	Wed 1/21/26	Fri 1/23/26	8SS	
15			Task 3.3.2 Treatment Alternatives	7 days	Fri 1/23/26	Mon 2/2/26	8SS+2 days	16SS+2 days
16			Task 3.3.3 Select Best Alternative	3 days	Tue 1/27/26	Thu 1/29/26	15SS+2 days	18
17			**Task 3.4 Membrane Filtration**	**9 days**	**Fri 1/30/26**	**Wed 2/11/26**		**37**
18			Task 3.4.1 Define Criteria	3 days	Fri 1/30/26	Tue 2/3/26	16	20,19SS+2 days
19			Task 3.4.2 Treatment Alternatives	7 days	Tue 2/3/26	Wed 2/11/26	18SS+2 days	
20			Task 3.4.3 Select best Alternative	3 days	Wed 2/4/26	Fri 2/6/26	18	22
21			**Task 3.5 Advanced Oxidation**	**9 days**	**Mon 2/9/26**	**Thu 2/19/26**		**37**
22			Task 3.5.1 Define Criteria	3 days	Mon 2/9/26	Wed 2/11/26	20	24FS+2 days,26,23SS+2 days
23			Task 3.5.2 Treatment Alternatives	7 days	Wed 2/11/26	Thu 2/19/26	22SS+2 days	
24			Task 3.5.3 Select best Alternative	3 days	Mon 2/16/26	Wed 2/18/26	22FS+2 days	53
25			**Task 3.6 Ultraviolet Disinfection**	**9 days**	**Thu 2/12/26**	**Tue 2/24/26**		**37**
26			Task 3.6.1 Define Criteria	3 days	Thu 2/12/26	Mon 2/16/26	22	28,27SS+2 days
27			Task 3.6.2 Treatment Alternatives	7 days	Mon 2/16/26	Tue 2/24/26	26SS+2 days	
28			Task 3.6.3 Select Best Alternative	3 days	Tue 2/17/26	Thu 2/19/26	26	30
29			**Task 3.7 Final Chlorination**	**9 days**	**Fri 2/20/26**	**Wed 3/4/26**		**37**
30			Task 3.7.1 Define Criteria	3 days	Fri 2/20/26	Tue 2/24/26	28	32,31SS+2 days
31			Task 3.7.2 Treatment Alternatives	7 days	Tue 2/24/26	Wed 3/4/26	30SS+2 days	
32			Task 3.7.3 Select Best Alternative	3 days	Wed 2/25/26	Fri 2/27/26	30	34
33			**Task 3.8 Brine Management**	**9 days**	**Mon 3/2/26**	**Thu 3/12/26**		**37**
34			Task 3.8.1 Define Criteria	3 days	Mon 3/2/26	Wed 3/4/26	32	36,35SS+2 days
35			Task 3.8.2 Treatment Alternatives	7 days	Wed 3/4/26	Thu 3/12/26	34SS+2 days	
36			Task 3.8.3 Select Best Alternative	3 days	Thu 3/5/26	Mon 3/9/26	34	
37			Task 3.9 Develop & Select Treatment Trains	2 days	Fri 3/13/26	Mon 3/16/26	8,9,13,17,21,25,29,33	39FS+2 days,54SS+1 day
38			**Task 4.0: Final Design**	**26 days**	**Thu 3/19/26**	**Thu 4/23/26**		
39			Task 4.1 Selected Treatment Process Design	12 days	Thu 3/19/26	Fri 4/3/26	37FS+2 days	40FS+1 day,42SS,43SS,44FS+1 day
40			Task 4.2 Site Layout	12 days	Tue 4/7/26	Wed 4/22/26	39FS+1 day	
41			**Task 4.3 Hydraulic Analysis**	**20 days**	**Thu 3/19/26**	**Wed 4/15/26**		
42			Task 4.3.1 Lift Station Design	10 days	Thu 3/19/26	Wed 4/1/26	39SS	
43			Task 4.3.2 Pump Selection	10 days	Thu 3/19/26	Wed 4/1/26	39SS	
44			Task 4.3.3 Piping Design	7 days	Tue 4/7/26	Wed 4/15/26	39FS+1 day	46FS+1 day
45			Task 4.4 Public Outreach Plan	5 days	Fri 4/17/26	Thu 4/23/26	46SS	
46			Task 4.5 Construction Phasing	4 days	Fri 4/17/26	Wed 4/22/26	44FS+1 day	48SS+4 days,55FF,45SS
47			**Task 5.0: Cost Assessment**	**10 days**	**Thu 4/23/26**	**Wed 5/6/26**		
48			Task 5.1 Opinion of Probable Construction Costs	5 days	Thu 4/23/26	Wed 4/29/26	46SS+4 days	49SS,50
49			Task 5.2 Operation and Maintenance Costs	5 days	Thu 4/23/26	Wed 4/29/26	48SS	
50			Task 5.3 Life Cycle Costs	5 days	Thu 4/30/26	Wed 5/6/26	48	51SS+1 day
51			Task 6.0: Impact Analysis	5 days	Fri 5/1/26	Thu 5/7/26	50SS+1 day	56FF
52			**Task 7.0: Project Deliverables**	**57 days**	**Thu 2/19/26**	**Fri 5/8/26**		
53			Task 7.1 30% Deliverable	7 days	Thu 2/19/26	Fri 2/27/26	24	54FS+10 days
54			Task 7.2 60% Deliverable	7 days	Mon 3/16/26	Tue 3/24/26	53FS+10 days,37SS+1 day	55FS+15 days
55			Task 7.3 90% Deliverable	7 days	Wed 4/15/26	Thu 4/23/26	54FS+15 days,46FF	56FS+4 days,57FS-15 days
56			Task 7.4 Final Deliverable	7 days	Thu 4/30/26	Fri 5/8/26	55FS+4 days,51FF	
57			Task 7.5 Competition Final Report	7 days	Fri 4/3/26	Mon 4/13/26	55FS-15 days	58FF+13 days
58			Task 7.6 Competition Final Presentation	7 days	Wed 4/22/26	Thu 4/30/26	57FF+13 days	
59			**Task 8.0: Project Management**	**138 days**	**Mon 10/27/25**	**Wed 5/6/26**		
60			Task 8.1 Schedule Management	138 days	Mon 10/27/25	Wed 5/6/26	1SS	61FF
61			Task 8.2 Resouce Management	138 days	Mon 10/27/25	Wed 5/6/26	60FF	62FF
62			Task 8.3 Meetings	138 days	Mon 10/27/25	Wed 5/6/26	61FF	

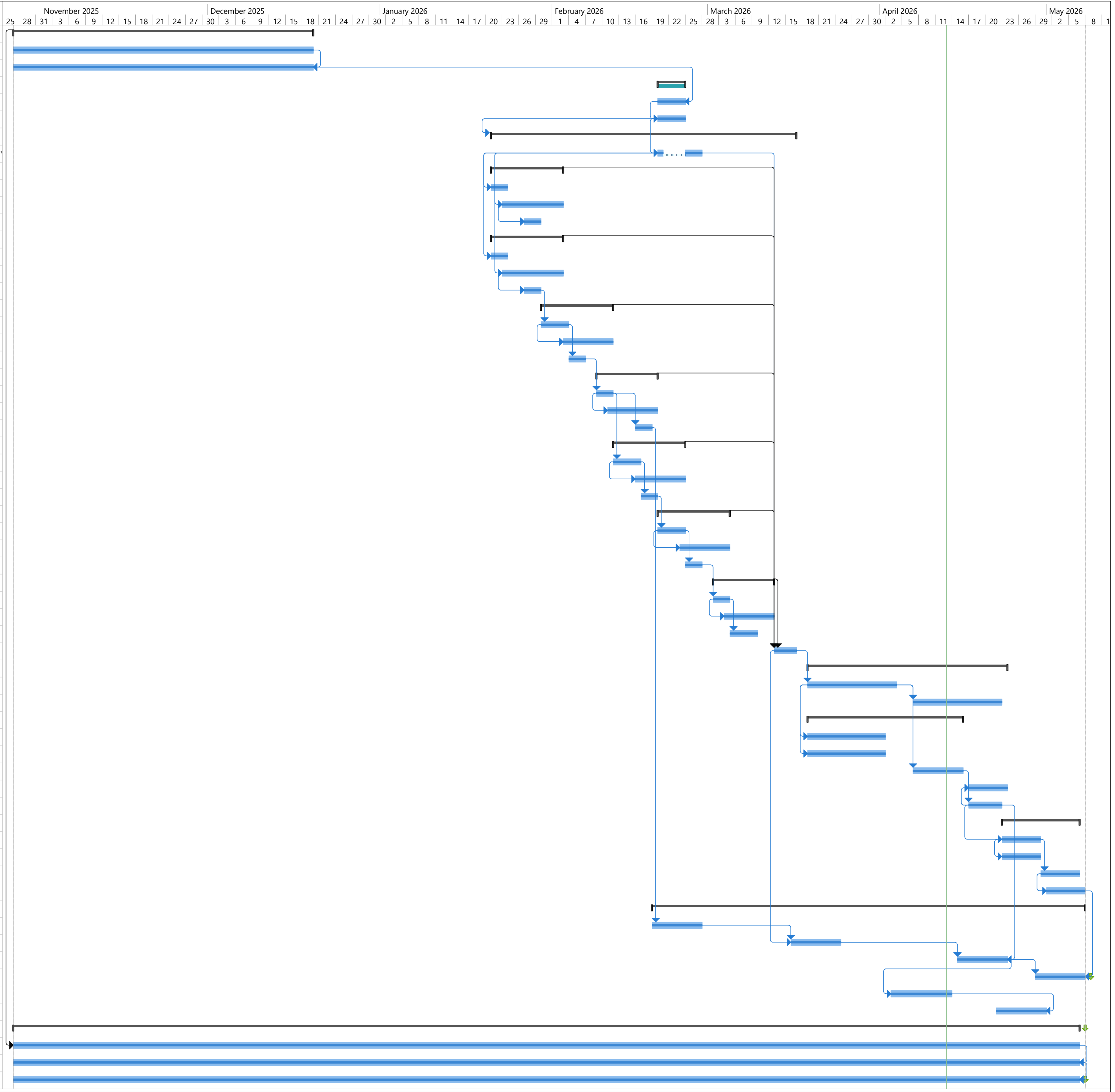


Project: 91st Ave. Regional
AWPF Project
Date: Mon 11/24/25

Task	▬	Summary	┏━┓	Inactive Milestone	◇	Duration-only	▬	Start-only	[	External Milestone	◇	Critical Split	┈
Split	┈	Project Summary	┏━┓	Inactive Summary	┏━┓	Manual Summary Rollup	▬	Finish-only	]	Deadline	⇩	Progress	─
Milestone	◆	Inactive Task	▭	Manual Task	▬	Manual Summary	┏━┓	External Tasks	▬	Critical	▬	Manual Progress	─

Appendix O: Actual Schedule

ID	ℹ	Task Mode	Task Name	Duration	Start	Finish	Predecessors	Successors
1	✓		**Task 1.0: Research Preparation**	**40 days**	**Mon 10/27/25**	**Fri 12/19/25**		**60SS**
2	✓		Task 1.1 WEF Application	40 days	Mon 10/27/25	Fri 12/19/25		3FF
3	✓		Task 1.2 Regulation Research	40 days	Mon 10/27/25	Fri 12/19/25	2FF	5FF+22 days
4	✓		**Task 2.0: Site Assessment**	**3 days**	**Fri 2/20/26**	**Tue 2/24/26**		
5	✓		Task 2.1 Site Visit	3 days	Fri 2/20/26	Tue 2/24/26	3FF+22 days	6SS
6	✓		Task 2.2 Data Analysis	3 days	Fri 2/20/26	Tue 2/24/26	5SS	7SS+2 days,8SS+3 days
7	✓		**Task 3.0: Treatment Selection Process**	**39 days**	**Wed 1/21/26**	**Mon 3/16/26**	**6SS+2 days**	
8	✓		Task 3.1 Determine Plant Requirements	4 days	Fri 2/20/26	Fri 2/27/26	6SS+3 days	10SS,11SS+2 days,37,14SS,15SS+2 da
9	✓		**Task 3.2 Coagulation/Flocculation**	**9 days**	**Wed 1/21/26**	**Mon 2/2/26**		**37**
10	✓		Task 3.2.1 Define Criteria	3 days	Wed 1/21/26	Fri 1/23/26	8SS	
11	✓		Task 3.2.2 Treatment Alternatives	7 days	Fri 1/23/26	Mon 2/2/26	8SS+2 days	12SS+2 days
12	✓		Task 3.2.3 Eliminate Least Suitable Alternativ	3 days	Tue 1/27/26	Thu 1/29/26	11SS+2 days	
13	✓		**Task 3.3 Sedimentation**	**9 days**	**Wed 1/21/26**	**Mon 2/2/26**		**37**
14	✓		Task 3.3.1 Define Criteria	3 days	Wed 1/21/26	Fri 1/23/26	8SS	
15	✓		Task 3.3.2 Treatment Alternatives	7 days	Fri 1/23/26	Mon 2/2/26	8SS+2 days	16SS+2 days
16	✓		Task 3.3.3 Eliminate Least Suitable Alternativ	3 days	Tue 1/27/26	Thu 1/29/26	15SS+2 days	18
17	✓		**Task 3.4 Membrane Filtration**	**9 days**	**Fri 1/30/26**	**Wed 2/11/26**		**37**
18	✓		Task 3.4.1 Define Criteria	3 days	Fri 1/30/26	Tue 2/3/26	16	20,19SS+2 days
19	✓		Task 3.4.2 Treatment Alternatives	7 days	Tue 2/3/26	Wed 2/11/26	18SS+2 days	
20	✓		Task 3.4.3 Select best Alternative	3 days	Wed 2/4/26	Fri 2/6/26	18	22
21	✓		**Task 3.5 Advanced Oxidation**	**9 days**	**Mon 2/9/26**	**Thu 2/19/26**		**37**
22	✓		Task 3.5.1 Define Criteria	3 days	Mon 2/9/26	Wed 2/11/26	20	24FS+2 days,26,23SS+2 days
23	✓		Task 3.5.2 Treatment Alternatives	7 days	Wed 2/11/26	Thu 2/19/26	22SS+2 days	
24	✓		Task 3.5.3 Eliminate Least Suitable Alternativ	3 days	Mon 2/16/26	Wed 2/18/26	22FS+2 days	53
25	✓		**Task 3.6 Ultraviolet Disinfection**	**9 days**	**Thu 2/12/26**	**Tue 2/24/26**		**37**
26	✓		Task 3.6.1 Define Criteria	3 days	Thu 2/12/26	Mon 2/16/26	22	28,27SS+2 days
27	✓		Task 3.6.2 Treatment Alternatives	7 days	Mon 2/16/26	Tue 2/24/26	26SS+2 days	
28	✓		Task 3.6.3 Eliminate Least Suitable Alternativ	3 days	Tue 2/17/26	Thu 2/19/26	26	30
29	✓		**Task 3.7 Final Chlorination**	**9 days**	**Fri 2/20/26**	**Wed 3/4/26**		**37**
30	✓		Task 3.7.1 Define Criteria	3 days	Fri 2/20/26	Tue 2/24/26	28	32,31SS+2 days
31	✓		Task 3.7.2 Treatment Alternatives	7 days	Tue 2/24/26	Wed 3/4/26	30SS+2 days	
32	✓		Task 3.7.3 Eliminate Least Suitable Alternativ	3 days	Wed 2/25/26	Fri 2/27/26	30	34
33	✓		**Task 3.8 Brine Management**	**9 days**	**Mon 3/2/26**	**Thu 3/12/26**		**37**
34	✓		Task 3.8.1 Define Criteria	3 days	Mon 3/2/26	Wed 3/4/26	32	36,35SS+2 days
35	✓		Task 3.8.2 Treatment Alternatives	7 days	Wed 3/4/26	Thu 3/12/26	34SS+2 days	
36	✓		Task 3.8.3 Eliminate Least Suitable Alternativ	3 days	Thu 3/5/26	Mon 3/9/26	34	
37	✓		Task 3.9 Develop & Select Treatment Trains	2 days	Fri 3/13/26	Mon 3/16/26	8,9,13,17,21,25,29,33	39FS+2 days,54SS+1 day
38	✓		**Task 4.0: Final Design**	**26 days**	**Thu 3/19/26**	**Thu 4/23/26**		
39	✓		Task 4.1 Selected Treatment Design Process	12 days	Thu 3/19/26	Fri 4/3/26	37FS+2 days	40FS+1 day,42SS,43SS,44FS+1 day
40	✓		Task 4.2 Site Layout	12 days	Tue 4/7/26	Wed 4/22/26	39FS+1 day	
41	✓		**Task 4.3 Hydraulic Analysis**	**20 days**	**Thu 3/19/26**	**Wed 4/15/26**		
42	✓		Task 4.3.1 Lift Station Design	10 days	Thu 3/19/26	Wed 4/1/26	39SS	
43	✓		Task 4.3.2 Pump Selection	10 days	Thu 3/19/26	Wed 4/1/26	39SS	
44	✓		Task 4.3.3 Piping Design	7 days	Tue 4/7/26	Wed 4/15/26	39FS+1 day	46FS+1 day
45	✓		Task 4.4 Public Outreach Plan	5 days	Fri 4/17/26	Thu 4/23/26	46SS	
46	✓		Task 4.5 Construction Phasing	4 days	Fri 4/17/26	Wed 4/22/26	44FS+1 day	48SS+4 days,55FF,45SS
47	✓		**Task 5.0: Cost Assessment**	**10 days**	**Thu 4/23/26**	**Wed 5/6/26**		
48	✓		Task 5.1 Opinion of Probable Construction Cost	5 days	Thu 4/23/26	Wed 4/29/26	46SS+4 days	49SS,50
49	✓		Task 5.2 Operation and Maintenance Costs	5 days	Thu 4/23/26	Wed 4/29/26	48SS	
50	✓		Task 5.3 Life Cycle Costs	5 days	Thu 4/30/26	Wed 5/6/26	48	51SS+1 day
51	✓		Task 6.0: Impact Analysis	5 days	Fri 5/1/26	Thu 5/7/26	50SS+1 day	56FF
52	✓		**Task 7.0: Project Deliverables**	**56 days**	**Thu 2/19/26**	**Thu 5/7/26**		
53	✓		Task 7.1 30% Deliverable	7 days	Thu 2/19/26	Fri 2/27/26	24	54FS+10 days
54	✓		Task 7.2 60% Deliverable	7 days	Mon 3/16/26	Tue 3/24/26	53FS+10 days,37SS+1 day	55FS+15 days
55	✓		Task 7.3 90% Deliverable	7 days	Wed 4/15/26	Thu 4/23/26	54FS+15 days,46FF	56FS+1 day,57FS-15 days
56	✓		Task 7.4 Final Deliverable	7 days	Wed 4/29/26	Thu 5/7/26	55FS+1 day,51FF	
57	✓		Task 7.5 Competition Final Report	7 days	Fri 4/3/26	Mon 4/13/26	55FS-15 days	58FF+13 days
58	✓		Task 7.6 Competition Final Presentation	7 days	Wed 4/22/26	Thu 4/30/26	57FF+13 days	
59	✓		**Task 8.0: Project Management**	**138 days**	**Mon 10/27/25**	**Wed 5/6/26**		
60	✓		Task 8.1 Schedule Management	138 days	Mon 10/27/25	Wed 5/6/26	1SS	61FF
61	✓		Task 8.2 Resouce Management	138 days	Mon 10/27/25	Wed 5/6/26	60FF	62FF
62	✓		Task 8.3 Meetings	138 days	Mon 10/27/25	Wed 5/6/26	61FF	



Project: 91st Ave. Regional
AWPF Project
Date: Mon 4/13/26

Task		Summary		Inactive Milestone		Duration-only		Start-only		External Milestone	
Split		Project Summary		Inactive Summary		Manual Summary Rollup		Finish-only		Deadline	
Milestone	◆	Inactive Task		Manual Task		Manual Summary		External Tasks		Critical	

Critical Split
Progress
Manual Progress