2023-2024 WEF AZWA Student Design Competition

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Rainbow Valley Water Reclamation Facility Expansion Project Design Report

Goodyear, AZ

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Table of Contents

Figures

Tables

Equations

Appendices

Abbreviations

AAC	Arizona Administrative Code
AZWA	Arizona Water Association
BOD	Biological Oxygen Demand
CFR	Code of Federal Regulations
EIT	Engineer in Training
EPA	Environmental Protection Agency
MGD	Million Gallon per Day
MLSS	Mixed Liquor Suspended Solids
MLVSS	Mixed Liquor Volatile Suspended Solids
NTU	Nephelometric Turbidity Units
RVWRF	Rainbow Valley Water Reclamation Facility
SOUR	Specific Oxygen Uptake Rate
TBL	Triple Bottom Line
TSS	Total Suspended Solids
WEF	Water Environment Federation
WRF	Water Reclamation Facility

Acknowledgements

The 2023-2024 student capstone team for the Water Environment Federation competition wants to thank to following people for their professional and technical guidance throughout the process of completing the project:

Jeffrey Heiderscheidt, Senior Lecturer in the Department of Civil Engineering, Construction Management, and Environmental Engineering.

Adias Fostino, Environmental Engineer at Brown and Caldwell, Engineer-in-Training (EIT)

Abstract
The Rainbow Valley Water Reclamation Facility Expansion Project in Goodyear, AZ is proposed to expand the facility's capacity from 0.75 MGD to 3 MGD to support population growth and land development in the area. The plant is situated about 6 miles to the southeast of the confluence of Waterman Wash with the Gila River waters. Serving almost 3,500 people, the existing facility utilizes fine screens, an activated sludge process, disc filters, and a chlorine contact basin. Decision matrices were made to evaluate 2 – 3 alternatives for each step in the process. The proposed facility includes fine screen/vortex grit chamber combined systems, an equalization basin, primary clarifiers, activated sludge process, disc filters, ultraviolet disinfection, and centrifuges. The construction phases include preparing the site, earthwork, formwork for concrete, installation of prefabricated equipment, and the activation of systems. Operation requirements involve Grade 4 operators who oversee the daily operations supported by other Grade 1, 2, and 3 operators. The total construction cost is $31,617,180 with an annual operation and maintenance cost of $4,731,950.

Summary of Project Team Effort

The group comprising of Madysen Kambich, Gabrielle Dierking, Ayed Alnefaie, Milford Begay, and Chelsie Fowler, all seniors studying civil or environmental engineering, undertook a significant project to expand the Rainbow Valley Water Reclamation Facility in Goodyear, Arizona, from 0.75 MGD to 3 MGD. This expansion aims to meet the demands of escalating population growth and water quality standards.

Madysen Kambich, a senior majoring in civil engineering and serving as Lead Designer, utilized her expertise in civil engineering to develop and implement innovative solutions aligned with project objectives. Her astute evaluation of the existing system and diverse treatment approaches influenced robust expansion strategies.

Gabrielle Dierking, also a senior majoring in civil engineering and the project manager, orchestrated team activities and ensured punctual completion of tasks. Her adeptness in task delegation and stakeholder communication fostered seamless collaboration and effective issue resolution during the design phase.

In the role of Client Liaison, Ayed Alnefaie, a senior majoring in civil engineering, synchronized the project team's efforts with client expectations and deliveries. His adept negotiation skills and client-centric approach played a pivotal role in maintaining project momentum.

Milford Begay, a senior majoring in civil engineering and specializing in infrastructure design, provided invaluable insights into structural considerations crucial for the facility's expansion. His analytical prowess and meticulous attention to detail ensured the viability and durability of proposed expansion plans.

Chelsie Fowler, a senior majoring in environmental engineering, contributed expertise in ecologically friendly wastewater treatment techniques. Her commitment to environmental stewardship facilitated the integration of eco-friendly practices into expansion plans, minimizing environmental impact.

Additionally, Adias Fostino and Lisa Melton, with their extensive experience and insights, provided invaluable guidance essential for project understanding and coordination with existing infrastructure. Their contributions streamlined the integration of design concepts with site requirements.

Dr. Jeffrey Heiderscheidt was the team’s capstone instructor and provided guidance and technical advice throughout the project.

1.0 Project Introduction

1.1 Objectives

This project is to design an expansion for the Rainbow Valley Water Reclamation Facility in Goodyear, Arizona. The expansion is needed to accommodate population growth in the area and will increase the facility's capacity from 0.75 million gallons per day (MGD) to 3 MGD (Arizona Water Association, 2023). The facility is also lacking redundancy, which will be accounted for in the design. Goodyear, Arizona is located southwest of Phoenix and south of I-10 as shown in Figure 1-1. This project is part of the Water Environment Federation competition, and the team is required to evaluate conventional activated sludge, membrane bioreactors, and a third alternative of the team's choice.

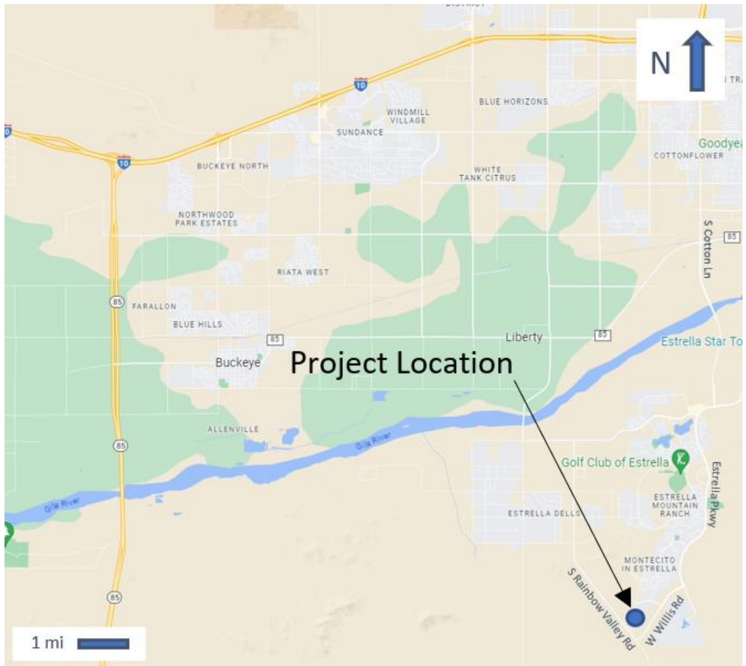

Figure 1-1: Project Vicinity Map (Google Earth , 2024)

1.2 Design Goals

The main objective of the project is to successfully design an expansion to increase the facility's capacity to 3 MGD and design for redundancy. The effluent will need to remain as Class A+ effluent and the biosolids will remain as Class B (City of Goodyear). After analyzing three different biological treatment processes, the team will select the treatment process that best suits the needs of the facility. The team will propose a construction phasing plan to ensure that the facility remains in operation during construction.

1.3 Facility Requirements

In compliance with Arizona Administrative Code R18-11-303, the Rainbow Valley Water Reclamation Facility (RVWRF) is required to generate Class A+ effluent and uphold the facility's own basic design and treatment criteria in addition to additional water quality standards (Waters, 2023). The post-expansion plant effluent needs to comply with all applicable regulations and/or surpass any upcoming discharge permits. The flow and load design criteria for the RVWRF and the effluent quality limits can be found in Appendix A-1 and A-2.

In addition, the facility will produce Class B biosolids in accordance with the applicable State and Federal regulations (Waters, 2023). At the time, the biosolids are land applied and must meet the pathogen reduction requirements established in A.A.C R18-9-1006. A Specific Oxygen Uptake Rate (SOUR) of less than 1.5 mg O_2/hr/g total solids at 20 degrees Celsius satisfies the vector attraction reduction criteria for Class B biosolids. The sludge must comply with the requirements for sewage sludge disposal in 40 Code of Federal Regulations (CFR) Part 503 and 18 A.A.C.9, Article 10 (Waters, 2023).

The facility's expansion during the proposed expansions must respect any buffer restrictions set forth by the federal, state, and local governments and remain inside the fence line. The site has an additional space of 150 ft outside of the existing fence in both the north and east directions that can be used (Waters, 2023). The values for the operations and maintenance for the RVWRF can be seen in Appendix A-3. A list of permits for the RVWRF design can be found in Appendix A-4.

1.4 Existing Conditions

The Rainbow Valley Water Reclamation facility currently produces effluent that is Class A+ non-potable water and distributed for irrigation reuse by two nearby communities (City of Goodyear). The existing facility currently has an influent wet well, fine static screens, aerobic and anoxic tanks, secondary settling clarifier, tertiary disc filters, and a chlorination contact basin. Figure 1-2 shows the layout of the existing facility. The influent is pumped to the facility by 3 influent pump stations from a wet well. The solids removed by the fine static screen are dropped into a dumpster and hauled off to a landfill. The anoxic tank following the screens introduces microorganisms to start decreasing the BOD in the influent. The aerobic tank has 4 zones that decrease in aeration going through the tank. The settling clarifier has returned activated sludge pumped back into the anoxic tank and waste activated sludge pumped into a centrifuge to be dewatered and hauled to the landfill. Two-disc filters are utilized as tertiary filtration. The water is then sent to the chlorination contact basin and is only dechlorinated if the effluent is going to be sent into the environment. Each step in the treatment process is monitored with an automated software system used by the plant operators. The existing process flow diagram of the facility can be found in Appendix A-5. Figure 1-2 shows a detailed layout of the Rainbow Valley Water Reclamation Facility.

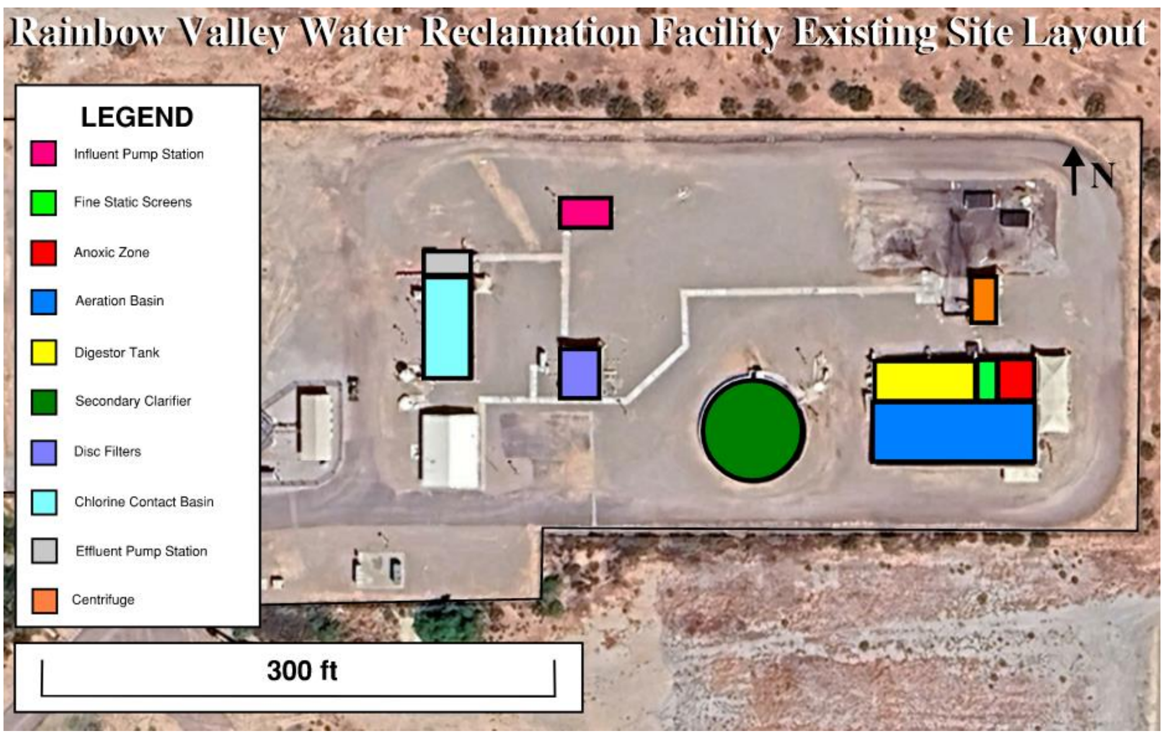

Figure 1-2: Existing Site Layout (Google Earth , 2024)

1.5 Constraints

One constraint for this project is the limited land area at the facility. The project requires expansion, so the area's layout must be planned out carefully to stay within the site location. Another constraint is the requirement of having the facility operate while the expansion to the facility is being constructed. Due to the restricted amount of land available for development and the duration of the construction project, construction phasing presents a difficulty.

2.0 Evaluation of Alternatives

2.1 Preliminary Treatment

The following section explains the design process for screening alternatives and grit chamber design. Preliminary treatment consists of physical treatment processes to remove solids from the wastewater. Each process will utilize a decision matrix with weighted criteria to score each alternative and determine the best one to be chosen for design.

2.1.1 Screening Alternatives

Screens remove larger objects and debris that could potentially damage and clog the equipment downstream in the wastewater treatment process. The three screens considered for the design were fine, static, and step screens.

2.1.1.1 Fine Screen

Fine screens were the first screening alternative considered. Fine screens can be mechanical or passive type screens to remove objects in the wastewater. Compared to other screens, the bars of a fine screen are placed closer together leaving less space for solids in the water to pass through. A fine screen would catch objects that would have passed through other types of screens and can remove more solids (EPA, 2003). Fine screens experience head loss, and the particles blocked accumulate at the screen and need to be removed regularly. Maintenance for fine screens can be automated and performed mechanically, giving it the least maintenance requirements out of the alternatives. An image of a fine screen can be found in Appendix B-1.

2.1.1.2 Static Screen

Static Screens were the second screening alternative considered. Static screens work by passively filtrating the water flowing through it. The bars in the screen restrict larger particles and debris from progressing further in the treatment process. The bars allow water to pass through, but do not stop smaller particles from flowing the screen and into the next step in the treatment process. Wastewater flowing through the bars does experience some head loss (EPA, 2003). Particles blocked by the static screen build up and must be removed manually and regularly to ensure blockages do not occur at the screen and impede the water flow. Out of the alternatives considered, static screens are the simplest and have the least constructability requirements. An image of a static screen can be found in Appendix B-2.

2.1.1.3 Step Screen

Step screens were the last screening alternative that was considered. Step screens operate mechanically to remove larger particles and objects from the wastewater. The bars are shaped in a way that they have racks that catch objects in the wastewater which are then carried out of the water and into a hopper. The system has bars that are self-cleaning (Huber Technology, 2012). Like other screens, the wastewater experiences head loss as it moves through the bars. The objects filtered out will need disposal. The machine being more complex means that any repairs will need to be done by a licensed technician. The bars in the screens clean themselves automatically without needing attention from staff. An image of a step screen can be found in Appendix B-3.

2.1.1.4 Screen Selection

The different screening alternatives were each evaluated with six criteria which were capital cost, maintenance and operation requirements, constructability of each alternative, odor control, staffing requirements, and social and environmental impacts. Capital cost was weighted the highest in the final selection due to how the price would affect the cost of construction for the project. Maintenance and operation requirements were weighted the second highest due to the level of maintenance impacting the cost for site staff. Following that is constructability, which considers how much time and effort would go into construction for each alternative. Odor control was considered due to the potential impact on the nearby area. Staffing requirements were considered and lastly, social and environmental impacts as there is a community nearby that could expand closer to the site. From the three different alternatives considered, the team selected the fine screen to use in the final design. The fine screen features a mechanical cleaning process, making it easier to operate and requires less staff attention. The fine screen had less capital cost compared to the other two options and is prefabricated by the manufacturer, only needing installation making it easier to incorporate into the design.

Table 2-1 shows the decision matrix used to select the best screening alternative. A detailed decision matrix can be found in Appendix B-4.

Table 2-1: Screening Decision Matrix

Preliminary Treatment (Screening)				
Criteria	**Weight (%)**	**Fine Screen**	**Step Screen**	**Static Screen**
Capital Cost	**30**	3	1	2
Maintenance & Operation	**25**	3	2	1
Construction Time/ Constructability	**15**	2	1	3
Odor Control	**10**	2	3	1
Social & Environmental Impacts	**10**	2	3	1
Staffing	**10**	3	2	1
Weighted Average	**100**	**2.65**	1.75	1.60

2.1.2 Grit Chamber

Grit chambers are crucial to the design process because they reduce flow rates and remove particles from water that screens are unable to completely filter out. This process aids in protecting the downstream equipment and settling out the inorganic materials. The three types of grit chambers considered for the design were aerated, horizontal-flow, and vortex-type grit chamber.

2.1.2.1 Aerated Grit Chamber

The aerated grit chamber was the first alternative considered for the preliminary treatment because of its versatility of handling different flow rates and efficiency in grit removal. This technology aids in flocculation as chemicals are added in and is consistent in removing a wide range of grit sizes. It requires additional labor for maintenance and operation because the system usually follows a propriety design, which makes modifications difficult (EPA, 2023). Overall, the aerated grit chamber is versatile in handling different flow rates but requires additional labor for the maintenance and operation. An image of an aerated grit chamber can be found in Appendix C-1.

2.1.2.2 Horizontal-Flow Grit Chamber

The horizontal-flow grit chamber was the second alternative considered as part of the preliminary treatment. This treatment is not sufficient in maintenance and operation due to the difficulty of maintaining a constant flow. With the water flowing through the horizontal channel, there is more wear to the product which results in a higher maintenance and operation weight. This type of grit chamber also requires a large land area because of the large channel that is required to handle the low flow requirements. Overall, the horizontal-flow grit chamber requires more maintenance and operation and requires a large land area. An image of a horizontal-flow grit chamber can be found in Appendix C-2.

2.1.2.3 Vortex-Type Grit Chamber

The vortex-type grit chamber was the third alternative considered due to the high removal efficiency criteria for particles larger than 300 microns and for smaller particles less than 210 microns (EPA, 2003). The technology is energy efficient which results in a medium scoring and has minimal maintenance by using a high-pressure agitation to loosen grit compacted in the sump. The head loss is minimal and has a small footprint due to its smaller size (EPA, 2003). Therefore, the vortex-type grit chamber has a high removal efficiency with minimal maintenance and operations along with a small footprint. An image of a vortex-type grit chamber can be found in Appendix C-3.

2.1.2.4 Grit Chamber Selection

A decision matrix was used to analyze and compare an aerated grit chamber, horizontal-flow grit chamber, and vortex-type grit chamber. The six criteria chosen for the comparison are capital cost, removal efficiency, construction time and constructability, maintenance and operation, footprint and surface area, and energy consumption. Since capital expenses will account for most costs and a cost analysis will be conducted after design, capital cost was assigned the highest weight. Removal efficiency and footprint and surface area received the second-highest weight because the efficiency of the treatment is the main goal, and the footprint is important because of the limited land space. Construction time and constructability was deemed the third highest in weight since construction phasing will be implemented. Maintenance and operation and energy consumption were included due to cost but was weighted the lowest since it will not be the largest cost compared to the other factors. Depending on how each alternative performs with each criterion, one will be rated a one and the other a three, with three being the greatest score. The notable difference and decision-making factors were removal efficiency, construction time and constructability and the surface area. The vortex-type grit chamber is prefabricated while the other two require a basin and additional labor. The vortex-type chamber is more energy efficient, requires less power, and has lower energy consumption. The alternative selected for the grit chamber was the vortex-type grit chamber.

Table 2-2 shows the decision matrix used in the decision-making process of selecting one alternative for the grit chambers. A detailed decision matrix can be found in Appendix C-4.

Table 2-2: Grit Chamber Decision Matrix

Preliminary Treatment (Grit Chamber)				
Criteria	**Weight (%)**	**Aerated Grit Chamber**	**Horizontal Flow Grit Chamber**	**Vortex-Type Grit Chamber**
Capital Cost	**25**	3	2	1
Removal Efficiency	**20**	2	3	3
Construction Time/ Constructability	**15**	2	1	3
Maintenance & Operation	**10**	2	1	3
Footprint/Surface Area	**20**	2	1	3
Energy Consumption	**10**	1	3	2
Weighted Average	**100**	2.15	1.85	**2.4**

2.2 Equalization Basin

The purpose of an equalization basin is to neutralize the influent flow coming into the plant and maintain a consistent average daily flow throughout the facility (Khudenko, 1985). During peak flows and surges, the basin will hold influent to maintain a constant flow throughout the plant and release stored influent during low flows. Doing this allows the facility to operate efficiently since the plant is designed to treat 3 MDG. Two types of equalization basins were considered for this design, which are an in-line basin and a side-line basin.

2.2.1 In-Line Basin

An in-line equalization basin acts as part of the treatment train with all influent passing through before the primary clarifier. None of the influent is treated in this step, just held, or released based on the flow needs. Since bypassing the basin is not an option, no additional equipment or pipes will be needed to tie into the treatment train. In-line basins work best for facilities with an inconsistent influent rate all throughout the day. An in-line basin is easier to maintain and operate with the basin being a regular part of the treatment train. Overall, in-line equalization basins are simpler and more seamless to implement in the treatment process. An image of an in-line basin can be found in Appendix D-1.

2.2.2 Side-Line Basin

A side-line equalization basin acts more as overflow influent storage and is not a part of the treatment train. As the name suggests, the side-line basin is placed aside from the main treatment train. When the influent flow is greater than the average flow that runs through the facility, untreated water will be diverted into the basin. Once the flow of the influent is low enough to need more water, the basin will release the influent accordingly back into the treatment train. With this basin being optional for influent, more equipment and piping would be needed to operate. The additional equipment and piping resulted in a higher capital cost and more difficult constructability. A side-line equalization basin can be more complex to implement into the treatment process. An image of a side-line basin can be found in Appendix D-2.

2.2.3 Equalization Basin Selection

A decision matrix was utilized to analyze and compare an in-line basin and side-line basin. With each type serving the same purpose, most of the criteria were scored very similarly. Capital cost was chosen to have the highest weight since a cost analysis will be performed after design and capital cost will be the bulk of the costs. Maintenance and operation were given the second highest weight since this criterion contributes to cost as well, but not with as much impact as capital cost. Construction time and constructability were deemed the third highest in weight since construction phasing will be implemented but is not a deciding factor. Staffing was included due to the cost that comes with it, but weighted the lowest since staffing cost will not be the largest cost. One alternative will be receiving a one and the other a two based on how the alternative performs with each criterion. The notable difference and ultimate decision-making factors were the constructability and cost. With the lack of extra equipment and piping, the in-line basin scored higher on both constructability and cost. The in-line equalization basin was chosen for implementation in the design.

Table 2-3 shows the decision matrix used to choose the optimal equalization basin design. A detailed decision matrix can be found in Appendix D-3.

Table 2-3: Equalization Basin Decision Matrix

Preliminary Treatment (Equalization Basin)			
Criteria	**Weight (%)**	**In-Line Basin**	**Side-Line Basin**
Capital Cost	**40**	2	1
Maintenance and Operation	**25**	2	1
Construction Time/Constructability	**20**	2	1
Staffing	**15**	2	1
Weighted Average	**100**	**2**	1

2.3 Primary Treatment

2.3.1 Bridge Supported Clarifier

The first primary clarifier alternative that was considered was a bridge-support clarifier. In a bridge support clarifier, the drive mechanism and scrapers are suspended from a bridge that extends across the diameter of the tank (GlobalSpec, 2024). This bridge can be utilized by operators for maintenance purposes. For economic reasons, this type of clarifier is used when the tank diameter is less than forty feet (Systems, 2018). The larger the diameter of the clarifier, the longer the bridge must be to support the scrapers and drive mechanism. An image of a bridge support clarifier can be found in Appendix E-1.

2.3.2 Column Supported Clarifier

The second primary clarifier alternative that was considered was a column-support clarifier. In a column support clarifier, the drive mechanism and scrapers are supported by a central column in the tank (GlobalSpec, 2024). Although the drive mechanism and scrapers are not supported by a bridge, column-support clarifiers can have a bridge that spans half of the tank's diameter for maintenance purposes. This type of clarifier is used when the tank diameter is more than forty feet (Systems, 2018). This type of clarifier is a more economical option for larger diameter tanks because the drive mechanism and scrapers are supported by a column rather than a bridge. An image of a column support clarifier can be found in Appendix E-2.

2.3.3 Primary Clarifier Selection

A decision matrix was utilized to analyze and compare a bridge support clarifier and a column support clarifier. The four criteria chosen when comparing the two alternatives were capital cost, surface area requirements, construction time and constructability, and maintenance and operation requirements. Capital cost was chosen to have the highest weight because a cost analysis will be performed after design and capital cost will be the bulk of the costs. Surface area requirements were given the second highest weight due to the small land area available at the project site.

Construction time and constructability were deemed the third highest in weight because construction phasing will be implemented but is not a deciding factor. Lastly, maintenance and operation were given the fourth highest weight. The alternatives will be given a score of one or two, one being the lowest and two being the highest. The alternative selected for the primary clarifier was the column-support clarifier. This option was deemed the most economical because of the size of the clarifier that is required to treat the flow of wastewater in the facility.

Table 2-4 shows the decision matrix used to choose the best primary treatment alternative. A detailed decision matrix can be found in Appendix E-3.

Table 2-4: Primary Clarifier Decision Matrix

Primary Treatment (Primary Clarifier)			
Criteria	**Weight (%)**	**Bridge Support Clarifier**	**Column Support Clarifier**
Capital Cost	**40**	1	2
Footprint	**25**	1	2
Construction Time/Constructability	**20**	1	2
Maintenance & Operation	**15**	2	1
Weighted Average	**100**	1.15	**1.85**

2.4 Secondary Treatment

2.4.1 Conventional Activated Sludge

Conventional activated sludge treatment stands as a staple in biological wastewater treatment technologies, playing a pivotal role in the removal of organic matter and nutrients through aeration and microbial activity. This process involves the mixing of wastewater with activated sludge in specialized tanks, fostering an environment where microscopic organisms diligently decrease pollutants. The design and operation of these systems are chosen to optimize crucial factors such as BOD removal efficiency, sludge retention time, and TSS removal efficacy. The activated sludge process ensures the efficient breakdown of contaminants, resulting in high-quality treated effluent. Due to the large capacity of the facility, the aeration tanks needed would require a large footprint, but a small number of tanks would be needed. The small number of tanks would require less maintenance and easier operation. The continuous refinement and optimization of conventional activated sludge systems underscore their significance in sustainable wastewater treatment practices, reflecting a commitment to environmental stewardship and the preservation of water resources (Ahansazan et al., 2014).

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2.4.2 Membrane Bioreactor

Membrane bioreactors (MBRs) combine biological treatment with advanced membrane filtration to achieve superior water quality within a compact system. In MBRs, microorganisms actively treat wastewater within bioreactor tanks, after which the clarified water undergoes filtration through membranes to eliminate solids and produce pristine water (Melin et al., 2006). The design of MBR systems at the facility involves meticulous selection of appropriate membranes, control of fouling mechanisms, and optimization of operational parameters to ensure high performance and effective water treatment. By carefully choosing the most suitable membranes, managing fouling issues, and fine-tuning system operations, the MBR technology guarantees efficient removal of contaminants and the production of high-quality treated water. A single MBR system cannot treatment a large capacity, so dozens of systems would need to be implemented for the facility. This would increase the maintenance needed and add complexity to operations.

2.4.3 Moving Bed Bioreactor

Moving Bed Bioreactors (MBBRs) represent an innovative biological water treatment technology that utilizes plastic media to facilitate the growth of biofilm. Within MBBR systems, this specialized media provides an environment for microorganisms to attach and proliferate, enhancing the removal of organic matter and nutrients effectively. The design of moving bed bioreactors encompasses the selection of appropriate media, the implementation of aeration systems, and the control of operational processes to optimize treatment efficiency and ensure seamless operation (Liao, Rasmussen, & Ødegaard, 2003). A single MBBR system treats very little water in comparison to the facility's needs which would lead to a substantial number of systems implemented. The large number of systems would exponentially increase maintenance needs and require multiple highly experienced operators.

2.4.4 Secondary Treatment Selection

A decision matrix was used to analyze and compare a conventional activated sludge process, membrane bioreactor, and moving bed bioreactor. The six criteria that were chosen for the comparison were capital cost, maintenance and operation, construction time and constructability, lifecycle cost, footprint, and the ability to meet permit limits. To maximize equipment longevity and guarantee construction phasing is executed, the criteria of maintenance and operation, and construction time and constructability were selected to carry the highest weight. The capital cost was chosen as the second highest because it accounts for most of the project cost. The footprint was placed the fourth highest weight due to the small land area available at the site. Since the treatment must comply with established permit restrictions but is not as important as constructability, maintenance, and operation, the ability to satisfy permit limits received the lowest grade. A score of one represents the worst alternative for the specific criteria and a score of three represents the best. Since the conventional activated sludge would utilize the current facilities, it will require less excavation and have lower maintenance and operation expenses than the other two designs. The alternative selected for the secondary treatment was the conventional activated sludge process.

Table 2-5 displays the decision matrix used to choose the best secondary treatment alternative. A detailed decision matrix can be found in Appendix F-6.

Table 2-5: Secondary Treatment Decision Matrix.

Secondary Treatment				
Criteria	**Weight (%)**	**Conventional Activated Sludge**	**Membrane Bioreactor**	**Moving Bed Bioreactor**
Capital Cost	**20**	1	3	2
Maintenance & Operation	**25**	3	1	1
Construction Time/ Constructability	**25**	2	1	2
Lifecycle Cost	**15**	3	1	1
Footprint	**10**	1	2	3
Removal Efficiency	**5**	1	2	3
Weighted Average	**100**	**2.05**	1.55	1.75

2.5 Advanced Treatment

2.5.1 Disc Filter

The first alternative considered for advanced treatment was a disc filter system. A disc filter works by allowing the water to seep through a cloth media with exceptionally fine mesh sizes to remove most of the solids still in the water (Evoqua, 2024). Parts for a disc filter system are prefabricated by the manufacturer then assembled on site making construction simpler.

Maintenance for disc filters involves lubrication of the system and replacement of parts as needed. Back washing occurs to clean out solids removed from the water that have accumulated on the media. An image of a disc filter can be found in Appendix G-1.

2.5.2 Sand Filter

The second alternative considered was a sand filter system. The sand filter would work by utilizing gravity, allowing the water to seep through a layer of sand to remove the suspended solids still present in the water (Evoqua, 2024). The treatment basin would need to be built on site from concrete and requires insulation of an underdrain system, pipes, and pumps used in it. Backwashing of the sands would need to be completed regularly to avoid the buildup of solids removed from the water. Sands used in the system will need to be replaced on occasion. The pumps and drains used will need to be inspected routinely and replaced as needed. An image of a sand filter can be found in Appendix G-2.

2.5.3 Advanced Treatment Selection

The team chose to use the disc filters in the design. The decision comes from considering four criteria for the decision matrix including cost, constructability, removal efficiency and maintenance. The highest weighted criterion for the final decision is the removal efficiency, to ensure at this stage that most of the TSS in the water is removed. Of the two alternatives, the disc filter system has the higher removal efficiency, being able to remove nearly all the BOD and TSS in the wastewater. The disc filter is also easier to construct as the parts are prefabricated by the manufacture and assembled on site as opposed to the sand filter which would require the construction of a treatment basin. In terms of cost the disc filter is less expensive. Additionally, there is an existing disc filtration system in operation at the treatment plant. Rather than installing a new filtration system, the project would expand on the existing disc filter system. Based on these criteria, the project will use the disc filter system.

A summary of the decision matrix for the advanced treatment is shown below in Table 2-6 with a more detailed decision matrix shown in Appendix G-3.

Table 2-6: Advanced Treatment Decision Matrix

Advanced Treatment			
Criteria	**Weight (%)**	**Disc Filters**	**Sand Filter**
Relative Cost	**30**	2	1
Construction Time/Constructability	**10**	2	1
Maintenance & Operation	**25**	2	1
Removal Efficiency	**35**	2	1
Weighted Average	**100**	**2**	1

2.6 Disinfection

2.6.1 Chlorination

Chlorination is done by adding the necessary amount of chlorine to the water in either a liquid, gaseous, or solid form. The contact time for chlorination is vital to achieve optimal disinfection and a large contact tank is designed to do so (EPA, 1999). Chlorination requires minimal maintenance if dechlorination is not needed. Dechlorination is required for drinking water or if the effluent water is going to be released into the environment. For reclaimed water such as this project, dechlorination is not required unless the effluent needs to be discharged into the environment. An image of a chlorination basin can be found in Appendix H-1.

2.6.2 Ultraviolet Disinfection

Ultraviolet disinfection is achieved using UV radiation to destroy the ability to reproduce pathogenic organisms in the water. UV lamps are placed around the channel the water flows through, requiring little contact time and a small footprint. Since UV is a physical process, there are no residual social or environmental impacts of concern (EPA, 1999). The operation of UV is simple and user-friendly requiring little monitoring. Maintenance requires the cleaning of the UV lamp tubes to ensure no residue cover is preventing optimal disinfection. An image of ultraviolet disinfection can be found in Appendix H-2.

2.6.3 Ozone Disinfection

Ozone disinfection is done by injecting ozone gas into the water. Ozone is an unstable gas and must be generated on site and requires complicated technology. The ozone gas destroys the pathogenic cells when they come into contact, then decomposes in the water as it comes into contact with oxygen, so no removal is needed. While ozone is not harmful for the environment, the gas is corrosive and reactive if handled improperly and can cause irritation to humans who come into contact with it. Three tanks are needed for ozone disinfection treatment: ozone generation, ozone contact basin, and ozone destruction (EPA, 1999). An image of ozone disinfection can be found in Appendix H-3.

2.6.4 Disinfection Selection

A decision matrix was utilized to analyze chlorination, ultraviolet disinfection, and ozone disinfection. The four criteria chosen when comparing the three alternatives were relative cost, surface area requirements, social and environmental impacts, maintenance and operation requirements, and disinfection rate. Relative cost was chosen to have the highest weight because a cost analysis will be performed after design and capital cost will be the bulk of the costs. Disinfection rate was given the second highest weight since this is the last step before the water leaves the site, making this a vital step in the treatment process. Surface area requirements were given the third highest weight due to the small land area available at the project site. Maintenance and operation were given the fourth highest weight since this criterion contributes to cost as well, but not with as much impact as capital cost. Social and environmental impacts were given the lowest weight. Since two of the three alternative use chemicals, the environmental impacts needed to be considered. Each alternative was scored one to three, with three being the best, and one being the worst. The alternative chosen based on the decision matrix was ultraviolet disinfection due to having the cheapest relative cost and smallest footprint.

Table 2-7 shows the decision matrix used to choose the best disinfection alternative. A detailed decision matrix can be found in Appendix H-4.

Table 2-7: Disinfection Decision Matrix

Disinfection				
Criteria	**Weight (%)**	**Chlorination Tank**	**UV**	**Ozone**
Relative Cost	**30**	2	3	1
Surface Area Requirements	**20**	1	3	2
Social & Environmental Impacts	**10**	1	3	2
Maintenance & Operation	**15**	3	2	1
Disinfection Rate	**25**	1	2	3
Weighted Average	**100**	1.6	**2.6**	1.8

2.7 Solids Management

The solids management at a facility is where the sludge from the wastewater is dewatered and/or treated before being hauled off site. The sludge from the wastewater must be dewatered to decrease the weight of sludge which will also decrease the cost of hauling the sludge off site. Some facilities treat sludge if it will be used for some other purpose rather than being taken to a landfill. The RVWRF requires Class B biosolids which can be achieved by simply dewatering the sludge. The three dewatering alternatives analyzed included a centrifuge, drying beds, and a filter press.

2.7.1 Centrifuge

A centrifuge dewaters sludge by rotating it in a bowl-like structure at high speeds to separate the water from the solids. A decanter centrifuge has a small drying time that typically ranges between ten and twenty minutes. Most centrifuges have a relatively small footprint which makes them ideal for facilities with limited available land area. They have few maintenance and operation requirements because the system is typically fully automated and requires limited attention from operators. An image of a centrifuge can be seen in Appendix I-1.

2.7.2 Drying Beds

Drying beds are large ponds with gravel and sand bottoms that act as filters. The gravel and sand make the bottom of the drying beds permeable so the water can filter out and leave the solids to dry. Drying beds require long drying times because the only thing drying the solids is the air in the atmosphere. Typical drying times range from multiple days to weeks. Drying beds have a large surface area so should only be used for facilities with a large available land area. They have high maintenance and operation requirements because the sludge must be manually leveled across the drying bed and removed after drying is complete. Drying beds may look concerning to the public because they are large pond-like structures in the middle of a facility. An image of a drying bed can be seen in Appendix I-2.

2.7.3 Filter Press

A filter press utilizes plates or rollers to compress the sludge to extract the water from the solids. Filter presses have a short drying time compared to drying beds but take longer to extract the water than centrifuges. The drying time for the sludge when using filter presses ranges between one to two hours. The surface area of filter presses is not as large as drying beds so they can be utilized in facilities with limited available land area. Filter presses have few operation requirements but the belts on the presses need to be replaced frequently. The average belt life for filter presses is approximately 2700 running hours. These belts also need to be washed frequently. An image of a filter press can be seen in Appendix I-3.

2.7.4 Solids Management Selection

A decision matrix was utilized to analyze and compare centrifuge, drying beds, and a filter press. The five criteria chosen when comparing the three alternatives were the relative cost of the product, environmental and social impacts, drying time, surface area, and maintenance and operation requirements. The relative cost was chosen to have the highest weight because the solids management step is important in the process so the cost of the dewatering device should be heavily considered. Surface area requirements were given the second highest weight because some of the dewatering alternatives required drastically different surface area requirements. The drying time was given the third highest score because the drying time could affect how fast the solids could be managed and the number of solids that would be produced each day. Maintenance and operation requirements were given the fourth highest weight because some of the dewatering alternatives require extra operation and maintenance requirements to manage the solids. Lastly, social and environmental impacts were given the lowest weight. In the decision matrix, the alternatives were given a score of 1 through 3, one being the worst and three being the best. The alternative selected for solids management was a centrifuge. A centrifuge was deemed the optimal alternative because of the fast-drying time and limited surface area requirements.

Table 2-8 shows the decision matrix used to choose the best solids management alternative. A detailed decision matrix can be found in Appendix I-4.

Table 2-8: Solids Management Decision Matrix

Solids Management				
Criteria	**Weight (%)**	**Centrifuge**	**Drying Beds**	**Filter Press**
Relative Cost	**30**	2	1	3
Environmental/Social Impacts	**10**	3	1	2
Drying Time	**20**	3	1	2
Surface Area Requirements	**25**	3	1	2
Maintenance & Operation	**15**	2	3	1
Weighted Average	**100**	**2.55**	1.3	2.15

3.0 Treatment Design

3.1 Preliminary Treatment Design

A vortex-type grit chamber with a spiral fine screen was chosen to incorporate into the treatment design. The Baffled Hydraulic Vortex PISTA Grit Removal Chamber is a high efficiency, fine-grit removal system with a 6 mm OBEX Spiral Fine Screen, manual bypass bar screen, flat-floor chambers, baffles, grit pump, grit washer, hydraulically produced vortex flow conditions and integrates a disposal hopper for screening and grit. All equipment components are made with stainless steel manufactured in pieces and welded together onsite. The system has a length of 44’-6”, a width of 12’-0” and a height of 15’-8”. (Loveless, 2012) The system is pre-assembled and shipped, decreasing construction and maintenance costs while maximizing service life. This combined design requires less space and allows for a smaller footprint of the treatment process in the facility where space is limited. The system has a 95% of 105-micron grit removal efficiency and handles wide variations in flow with a 10:1 turndown from peak to minimum flow. It automatically keeps the inlet velocity between 1.6-3.5 ft/s to prevent grit deposition upstream. (Loveless, 2012) Two of these chambers will be used in the expansion for redundancy and maintenance purposes. The product drawings of the system can be found in Appendix J-1. The implementation of these fine screens will decrease the odor emitted at the facility because the screens are enclosed. Adding a grit chamber to the facility will also decrease the odor emitted because there will be less solids in the primary clarifier and aeration basins that could potentially cause an excess of odor. The system will be housed in a concrete block building lined with ventilation systems and activated granular carbon drums because the majority of the odor is released during the preliminary treatment. Any substances that cause odor will be absorbed by the activated carbon, and enough ventilation will help disperse the odorous gases, preventing an accumulation of odor inside the structure.

3.2 Equalization Basin Design

An in-line equalization basin was chosen to implement into the design. A peak hour flow for 3 MGD was needed to calculate the volume required for the equalization basin. This was done by using the provided peak hour flow for 0.75 MGD and cross multiplying with 3 MGD to get an estimated peak hour flow. Using that estimation, an influent flow graph was created for 3 MGD. The basin's volume needed to be large enough to hold the volume of water represented by the area under the peak curve and above the average daily flow line. The area under the curve was estimated using the Reimann’s Sums method. Using that volume, dimensions of the basin were chosen to be 50 feet long, 40 feet wide, and 15 feet deep. The air required to prevent settling in the tank was then calculated. The last value calculated was the freeboard in the tank. All calculations and a plan and profile of the basin can be found in Appendix K-1.

3.3 Primary Treatment Design

A column support clarifier was chosen for the primary clarifier design. The available land area at the facility is limited so the team decided having one large clarifier would be more beneficial than having multiple small diameter clarifiers. This design decision will save space for additional treatments that will be required following the primary clarifier. The team has decided to use a column-support clarifier with a diameter of 65’ and a side water depth of 10’-2” (Envirodyne, 2022). Two of these clarifiers will be included in the expansion, one for daily use and another for redundancy purposes. The design calculations for this clarifier are in Appendix L-1 and L-2 and the record drawings are in Appendix L-3.

The first step in the primary clarifier design calculations was to calculate the clarifier's volume. The clarifier volume and the flow through the facility was used to calculate the detention time in the clarifier. The surface area was then calculated and was used to find the overflow rates in the clarifier. Next, the settling velocity needed to be calculated. To find the settling velocity, a particle size with a diameter of 0.2 mm and specific gravity 2.65 was used. Stokes law was originally used to calculate an initial settling velocity and then a Reynolds number associated with the initial settling velocity was calculated. The R value was in the transition range so an excel solver was used to complete iterative calculations of newtons equation until an acceptable Reynolds number was calculated. Once a final settling velocity was found, it was confirmed that it was faster than the overflow rate to ensure that particles could settle prior to exiting the clarifier. Lastly, the removal of TSS and BOD in the clarifier was calculated. A removal efficiency of 50% for TSS and 25% for BOD was used to calculate the removal of those constituents. The effluent value for TSS after the primary clarifier is 125 mg/L and BOD is 168.75 mg/L.

3.4 Secondary Treatment Design

A conventional activated sludge process was chosen for the secondary treatment design. This design would require five treatment trains to operate which would meet the criteria for the facility and have one for redundancy purposes. The team decided to use the same size aeration basin that the facility currently has with a length of 62’-6”, width of 40’, and height of 20’. The record drawing for the aeration basin can be found in Appendix M-1.

The first step in the conventional activated sludge design calculations was to find assumptions for Ks, μm, Kd, Y and MLVSS, which can be seen in Appendix M-2. The activated sludge calculations that were performed included the allowable soluble BOD5 in the effluent, mean cell residence time, safety factor, hydraulic detention time, return sludge concentration, maximum return sludge flow rate, flow rate of sludge wasting, mass flow rate, food to microorganism ratio, observed yield, net waste activated sludge produced each day, total mass produced, mass of solids lost in effluent, mass to be wasted, and mass of oxygen and air supplied. The activated sludge calculations can be found in Appendix M-2.

A secondary clarifier is the second step in the activated sludge process. A clarifier with a diameter of 55’ and a side water depth of 15’. Five of these clarifiers will be constructed at the facility, one following each of the aeration basins. The first step in the secondary clarifier design calculations was to calculate the clarifier's volume. The clarifier volume and the flow through the facility was used to calculate the detention time in the clarifier. The surface area was then calculated and was used to find the overflow rates in the clarifier. The settling velocity then needed to be calculated. To find the settling velocity, a particle size with a diameter of 1mm and specific gravity of 1.10 was used. Stokes law was used to calculate the settling velocity and then a Reynolds number associated with the settling velocity was calculated. Once the settling velocity was found, it was confirmed that it was faster than the overflow rate to ensure that particles could settle prior to exiting the clarifier. The secondary clarifier design calculations can be found in Appendix M-3. A removal efficiency of 90% for TSS and 95% for BOD was used to calculate the removal of those constituents in the activated sludge process. The effluent TSS is 12.5 mg/L, and the BOD is 8.4 mg/L. BOD is not removed in the disc filters or UV so 8.4 mg/L is the final BOD effluent value.

3.5 Advanced Treatment Design

Disc filters were chosen as the advanced treatment at the facility. The existing facility utilizes disc filters so two new disc filters will be added to treat the additional flow. The team chose to use the Hydrotech HSF2200 Disc filter from Veolia. Two additional disc filters that can each treat a flow of three million gallons per day will be installed and the existing disc filters will be used for redundancy purposes. The disc filters have a small footprint but a large removal efficiency. There are approximately 14 discs in each system with a pore size of 10 micrometers and an overall removal efficiency of 98% of suspended solids (Hydrotech Discfilter, 2024). The disc filters utilize a fifteen-horsepower backwash pump which creates a total power consumption of approximately 100 kWhr/day. The system utilizes self-cleaning backwash nozzles, and the backwash process does not require any additional water source meaning that there is continuous filtration while the backwash is being completed (Hydrotech Discfilter, 2024). Schematics and product information for the disc filter can be found in Appendix N-1 and N-2.

The disc filter is designed for an average influent TSS value of 15 mg/L and an average effluent TSS value of 5 mg/L. The influent TSS value from the secondary treatment is 12.8 mg/L which means the disc filters will remove the required total suspended solids.

3.6 Disinfection Design

Ultraviolet disinfection was chosen to implement into the design. The team chose to use the TROJANUV3000 PTP UV disinfection system. The system consists of a stainless-steel channel with UV modules around the channel. A single stainless-steel channel has a capacity of 0.499 MGD. To meet the capacity of 3MGD and provide redundancy, seven channels will be installed, six to meet capacity and one for redundancy purposes. Each channel is 9 feet 7 inches in length, 1 foot 6 inches in width, and 1 foot 11 inches in depth. There are 12 UV modules with four lamps for each channel. The UV modules have a UV transmission of 65% minimum, allowing the UV to be effective in the 1 foot 11-inch depth. The record drawing for the system can be found in Appendix O-1. The UV is installed at the facility with the purpose of deactivating the majority of the remaining microorganisms or pathogens from the activated sludge process so they can no longer reproduce.

3.7 Solids Management Design

An Andritz D4L decanter centrifuge was chosen as the solids management device for the facility. The existing facility utilizes a centrifuge so additional centrifuges will be added to treat the additional flow. Assuming a return activated sludge rate of 85%, 15% of the sludge produced at the facility will be dewatered using the centrifuge. To treat 15% of the sludge at the facility, the solids management system will need to have a capacity of at least 450,000 GPD. This centrifuge model has a design flow rate of 30 m^3/hr which is approximately 190,000 gallons/day (Separation, Decanter Centrifuges, D-Series, 2012). Three of these centrifuges will be installed at the facility to treat the flow and the existing centrifuge at the facility will be available for standby. The centrifuge has a 95% minimum solid capture indicating a high operating efficiency. Product images and information can be found in Appendix P-1 and P-2.

A conveyor belt will be installed with each centrifuge to transport the dried solids from the centrifuge to a dumpster so the solids can be hauled to a landfill off site. A 20’ belt conveyor from JDV Equipment Corporation was chosen to transport the dewatered solids to the dumpster. The conveyor belt can transport approximately 200 ft^3/hour of the dewatered sludge. Three of these conveyor belts will be installed at the facility, one at the end of each new centrifuge.

4.0 Hydraulic Analysis

A hydraulic analysis of the proposed final design was conducted to ensure the influent would properly flow through the facility. The analysis consisted of a system analysis and pump selection, then a hydraulic profile of the facility was created.

4.1 System Analysis

To select a pump for the influent pump station, a system analysis of the facility was completed. The goal of the system analysis was to determine the total dynamic headloss that occurs between the influent pump station and the first step of the treatment train. The proposed final site layout was designed to maximize gravity flow throughout the facility with a pump station at the influent wet well. The total dynamic headloss is the sum of the major headloss, minor headloss, and change in elevation between the influent wet well and the screens and grit chambers. The major headloss is due to the friction from the pipes. To calculate the major headloss, a friction factor was calculated using Equation 4-1, Swamee Jain's Equation. The major headloss was then calculated using Equation 4-2, the Darcy-Weisbach Equation. Minor headloss was then calculated for the entrance from the wet well to the pipe and the 90-degree bend through the pipe using Equation 4-3. The total dynamic headloss was then calculated using Equation 4-4. A spreadsheet in Excel was created to ease the task of determining pipe size, as pipe size contributes to major headloss. A pipe diameter of 3.5 feet made from commercial steel was chosen. All calculations for the system analysis can be found in Appendix Q-1.

Equation 4-1: Swamee Jain's Equation

$$f = \frac{0.25}{\left(\left(\log\left(\frac{e}{3.7D} + \frac{5.74}{{N_r}^{0.9}}\right)\right)^2\right)}$$

Where:
f = Friction Factor
e = Roughness Height (ft)
D = Pipe Diameter (ft)
N_r = Reynold's Number

Equation 4-2: Darcy-Weisbach Major Headloss

$$h_{lf} = f\left(\frac{L}{D}\right)\left(\frac{V^2}{2g}\right)$$

Where:
h_{lf} = Major Headloss (ft)
f = Friction Factor
L = Length of Pipe (ft)
D = Pipe Diameter (ft)
V = Velocity (ft/s)
g = Gravity Constant (ft/s^2)

Equation 4-3: Minor Headloss

$$h_{lm} = k\left(\frac{V^2}{2g}\right)$$

Where:
h_{lm} = Minor Headloss (ft)
k = Headloss Constant
V = Velocity (ft/s)
g = Gravity Constant (ft/s^2)

Equation 4-4: Total Dynamic Headloss

$$TDH = h_{lf} + \sum h_{lm} + \Delta elev$$

Where:
TDH = Total Dynamic Headloss (ft)
h_{lf} = Major Headloss (ft)
h_{lm} Minor Headloss (ft)
$\Delta elev$ = Change in Elevation (ft)

4.2 Pump Selection

Using the system analysis of the facility, the pump selection for the influent pump station was completed. The facility is designed to treat 3 MGD which equivalates to around 2100 gallons per minute. Utilizing Taco Comfort Solutions pump selection software, the criteria for the facility were input and multiple pumps resulted as possible matches. Since the facility has such a large capacity and a long pipe length, multiple pumps were needed to carry the flow. Parallel pump curves were compared to the facility's system curve to determine which pump was the best fit. To select the pump, the pump curve needed to intersect with the system curve at a flow equal or greater to the facility's demand. After doing this with multiple pumps, the Taco CI4009D pump was chosen and would operate as 5 pumps in parallel. The system and pump curve graph can be found in Appendix Q-2 and all CI4009D pump data can be found in Appendix Q-3.

4.3 Hydraulic Profile

A hydraulic profile of the proposed facility was created using AutoCAD. The profile shows the elevations of the bottom of the tank, ground level, and water surface for each step in the treatment train. All treatments with redundancy are designed at the same elevation, so one tank represents all redundant treatments on the hydraulic profile. To maximize gravity flow throughout the plant, the team decided to excavate dirt from the northwest corner of the property and transfer the dirt to the southeast corner where the screens and grit chamber will be placed. The dirt transfer will raise the elevation of the southeast corner by approximately 10 feet. The elevation change allows gravity to carry the water throughout the facility starting from the screens and grit chamber. The pumps selected will be the initial energy source to move the water from the influent wet well to the screens and grit chamber. The hydraulic profile can be found in Appendix Q-4.

5.0 Final Design Recommendations

5.1 Site Layout

A proposed site layout of the facility was created based on each treatment that was selected and designed. Before creating the proposed layout, the facility was expanded 150' in the north and east direction. The existing wet well and pump station will remain in the same place. The preliminary treatment in the site layout includes two fine screen/vortex grit chamber systems inside a concrete block building located in the southeast corner of the facility that will replace the existing fine screens. An equalization basin was placed just north of the screens that will feed into the two new primary clarifiers at the facility. The secondary treatment in the site layout includes five aeration basins and five secondary clarifiers. There are four new basins and clarifiers that will be added to the existing aeration basin and secondary clarifier in the activated sludge treatment process. Located west of the aeration basins and the secondary clarifiers are the existing disc filters and two new additional disc filters. West of the disc filters is where the new disinfection process is located. Seven UV disinfection systems will be installed to replace the existing chlorination contact basin. Lastly, located north of the disinfection process are three new centrifuges paired with the existing centrifuge to dewater the solids that will be produced at the facility. The proposed site layout can be seen in Figure 5-1. Despite the proposed site layout and the new processes at the facility, the administration building at the facility does not need to be expanded.

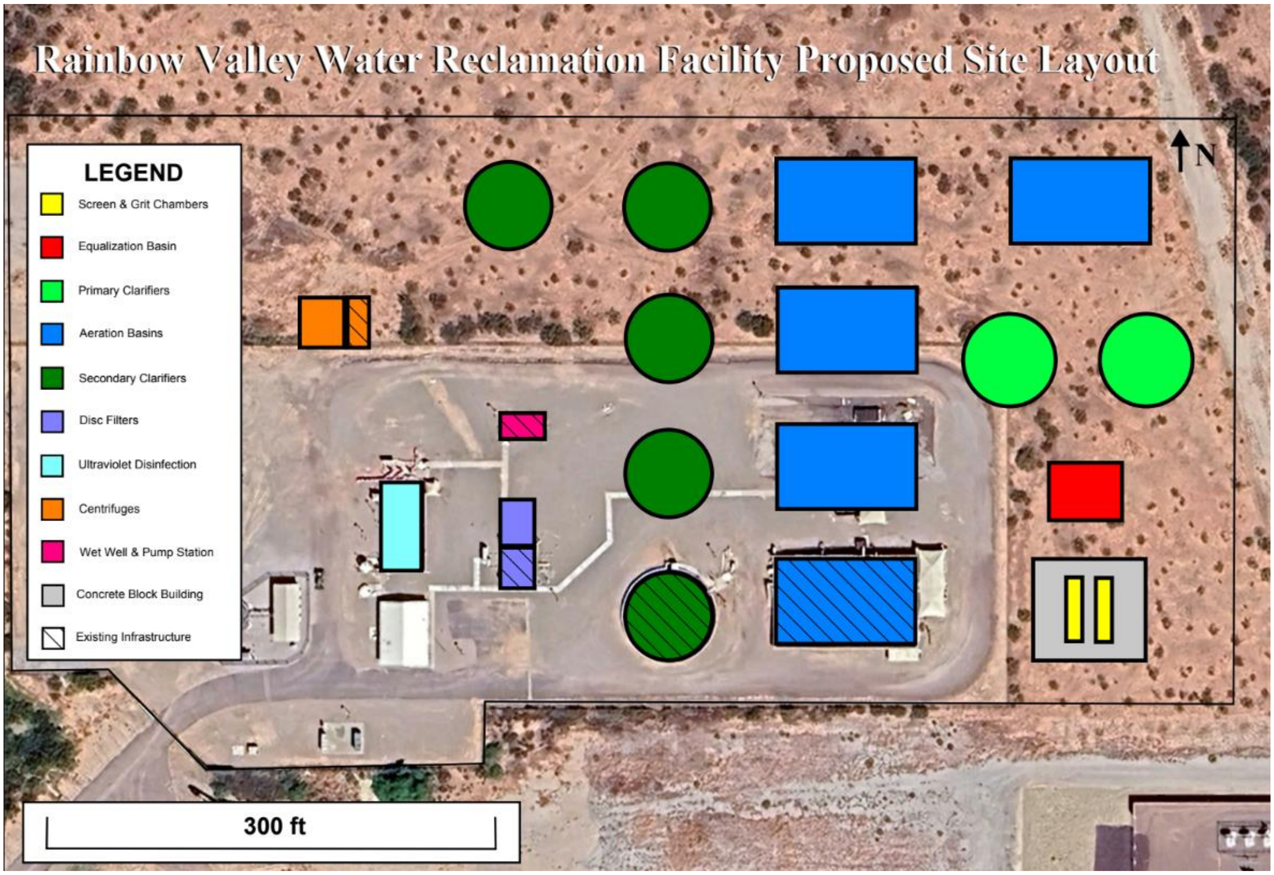


Figure 5-1: Proposed Site Layout

5.2 Process Flow Diagram

A process flow diagram of the proposed facility was created to show the relative flow path of the influent throughout the facility. For the steps in the process where waste is produced, the path and location of the of the waste is called out. The waste from the screens will be sent to a dumpster to be taken to the landfill. The waste from the primary and secondary clarifiers and the 15% WAS will be sent to the centrifuges to be dewatered then taken to the landfill. Multiple splitter boxes will be installed to allow the influent to flow to the treatment tanks that are in use since the facility was designed with redundancy. There will be valves to stop the flow into all tanks not in use. The process flow diagram can be found in Appendix R-1.

5.3 Construction Phasing

5.3.1 Phase 1: Site Preparation

The first phase of construction will be the preparation of the site for the proposed expansions. Most of the new expansions will be constructed in areas that do not have existing equipment or structures except for the proposed UV disinfection system which will be constructed where the existing chlorine contact basin is located. To keep the plant operational, UV disinfection will be installed in two parts. The first two UV systems will be installed immediately adjacent to the chlorine contact basin. When the first two UV systems are operational, the chlorine basin will be brought offline then demolished. After which an additional five UV disinfection systems will be installed and brought into operation. The only other existing equipment affected by the construction are the existing static screens, which will be removed after the new fine screen systems are installed and operational. The remainder of the existing equipment will be incorporated into the new site design. This phase is expected to take one month to complete.

5.3.2 Phase 2: Earthwork and Pipe and Pump Installation

The second phase of construction will be the earthwork done to prepare for the installation of the proposed expansions of the facility. Excavation will be completed to transfer soil to elevate the preliminary treatment an additional 10 feet. Soil will be removed from the northwest area of the site and transferred to the southeast corner where the screening and grit chambers will be installed. Additional earthwork will prepare for construction of foundations for the proposed site layout. While excavation takes place, all additional pipes will be laid. The pipes will be placed to deliver the wastewater once all equipment is in place. For treatments that require concrete foundations, the pipes will be placed so that the foundation can be poured around it. The new influent pump station will be installed next to the existing pump station. It will take an estimated time of two months to complete this stage of construction.

5.3.3 Phase 3: Concrete Formwork

The second phase of construction will be the form work for all of the proposed expansion. Concrete will be cast to create the basins for the primary and secondary clarifiers. Concrete will be placed to build the five aeration basins and the equalization basin. The foundations for other equipment and expansions will be laid during this time. It will take an estimated three months to lay the concrete and allow it time to cure.

5.3.4 Phase 4: Installation of Equipment

The third phase of construction will be installing the new equipment for the treatment facility. Equipment included in the expansion will be prefabricated by the respective manufacturers and transported to the site where they will be assembled and installed. Equipment to be installed includes the fine screen/grit chamber systems, primary clarifier equipment, and the disc filters. Additional work includes construction of equipment for the primary clarifiers and installation of the centrifuges. Instillation of the equipment will take an estimated two months.

5.3.5 Phase 5: New Equipment Activation

The final phase of construction is to activate the new systems around the treatment plant that have not yet been put online. The equalization basin will begin operation and the new clarifiers will be activated and incorporate the existing clarifier. The new aeration basins will begin operation as will the new pumps. The additional disc filters and UV systems will be put online at this point. At the end of this phase the new site layout will operate at a new capacity of 3 MGD.

5.4 Operation and Maintenance Requirements

According to the Arizona Department of Environmental Quality, the proposed Rainbow Valley Water Reclamation Facility would be classified as a Grade 3 facility. This is based on the point system used based on plant characteristics in Title 18 Chapter 5 Article 1 of the ADEQ. The AEDQ states a facility is Grade three if it consists of "Activated sludge serving 5,001 to 20,000 persons" (Arizona Department of Environmental Quality).

The grade of an operator is based on the education and years of experience with Grade 1 requiring a high school diploma and no experience and Grade 4 requiring 2 years of post-secondary education or a bachelor's degree and 3 years of experience. Grade 2 and 3 operators must have 2 years of post-secondary degree or a bachelor's degree. Along with that, a Grade 2 operator must have 6 months to 1 year of experience, and a Grade 3 operator must have 1 to 1.5 years of experience. Since the facility system is not too complex it will only require one Grade 4 operator. This senior operator will supervise all facility operations on a daily basis. Along with the Grade 4 operator, one Grade 2 or 3 operator will assist the senior operator with daily operations and maintenance. For extra assistance and to ensure the facility runs smoothly, one Grade 1 operator will be employed. The three operators will work five days a week with the Grade 4 and Grade 2 or 3 operators being on call over the weekends for emergencies.

6.0 Cost of Implementing the Design

6.1 Engineers Opinion of Probable Cost

The first part of the economic analysis for this project was completing the engineer's opinion of probable cost. All costs were found utilizing RS means and quotes from manufacturers. The first thing included in this cost is the capital cost for each product, which includes the cost of the pre-manufactured systems and any other concrete work required. The second thing included in this cost is the earthwork required for the project which was determined by calculating the amount of dirt that needs to be excavated and moved to other areas of the facility to create different elevations throughout the site. There is also a concrete excavation cost that is included for the removal of existing facilities that will not be utilized in the proposed design. Other costs included in this construction cost analysis is the costs for the construction of the building where the preliminary treatment system will be installed, the installation of the new influent pumps, new piping that is required, and new splitter boxes. The total estimated construction cost for this project is $31,617,181 which includes all labor, installation, and constructions costs. The full OPCC can be found in Appendix S-1.

6.2 Operation and Maintenance Cost

The operation and maintenance cost analysis for the proposed facility design includes all operation and maintenance costs required in the first year of operation. The costs in the analysis include energy consumption, replacement of parts, the maintenance of the products which may include oil changes and greasing of mechanical parts, and the labor costs for the number of operators at the facility. Most of these prices were given to us by manufacturers and any remaining costs were found using online resources. The operation and maintenance costs reflect the number of each system proposed in the design. The total estimated yearly operation and maintenance cost for this project is $4,731,951. This cost analysis can be found in Appendix S-2.

7.0 Impact Analysis

A Triple Bottom Line (TBL) impact analysis of the implementation of the proposed project was conducted and evaluated. The following table describes the positive and negative impacts for people, planet, and price for both implementing and not implementing the project. After qualitatively analyzing each alternative, a score is given for each kind of impact. Those scores are then used to determine the sustainability index (SI) of each alternative. A higher SI indicates that the positive impacts outweigh the negative impacts and the alternative with the higher Si is the better option.

Table 7-1: TBL Analysis

<table>
<tr><th colspan="2"></th><th colspan="2">People (Social)</th><th colspan="2">Planet (Environmental)</th><th colspan="2">Price (Economic)</th><th>Total</th><th>Max-Min</th><th>SI</th></tr>
<tr><td rowspan="2">Alternative 1: Implementation of the Project</td><td>Positive Impacts</td><td>-More residential opportunity
-More jobs provided</td><td rowspan="2">70</td><td>-More wastewater treated
-Addition of odor control</td><td rowspan="2">55</td><td>-More money into economy
-More land development
Score: 75</td><td rowspan="2">75</td><td rowspan="2">200</td><td rowspan="2">20</td><td rowspan="2">180</td></tr>
<tr><td>Negative Impacts</td><td>-Close housing to the facility
-City may grow too fast</td><td>-Construction will disrupt area
-More odorous gas</td><td>-Very expensive project
-Higher O&M cost</td></tr>
<tr><td rowspan="2">Alternative 2: Not Implementing the Project</td><td>Positive Impacts</td><td>-City would remain less crowded
-City resources used elsewhere</td><td rowspan="2">40</td><td>-No disruption to the existing land
-More free land around the facility</td><td rowspan="2">65</td><td>-City can use money elsewhere
-Lower O&M cost remain</td><td rowspan="2">65</td><td rowspan="2">140</td><td rowspan="2">30</td><td rowspan="2">110</td></tr>
<tr><td>Negative Impacts</td><td>-Less housing opportunities
-Less access to additional reclaimed water.</td><td>-Natural water sources would be utilized more
-Lack of odor control</td><td>-Less land development
-Additional treated water needed transported to the city</td></tr>
</table>

The implementation of the project, alternative one, resulted in a higher sustainability index meaning the positive impacts outweigh the negative impacts more than if the project is not implemented. Alternative one received a higher score than alternative two in each category. The scores for alternative one for the people and planet categories were much higher than alterative two which indicates that impacts to the people and planet for the implementation of the project are more beneficial than leaving the facility as is. The price, or economic, category had the closest score between the two alternatives. This is because alterative one would cost a lot of money upfront but result in more economic revenue in the long run while alternative two would have no additional costs upfront but would result in economic decline.

8.0 Summary of Engineering Work

A preliminary project schedule was created during the proposal phase of this project which can be found in Appendix T-1. This schedule included estimated start dates, end dates, and durations for each task and subtask. As this project progressed, the preliminary schedule was updated with actual finish dates and the correct durations for each task. This new schedule can be found in Appendix T-2. One noticeable difference that can be noted when comparing the new schedule to the preliminary schedule is the completion of the treatment design subtasks. In the preliminary schedule, the subtasks in the treatment design were all start-to-finish. Once the treatment design had started, it became clear that it was going to take much longer to get information from manufacturers than what was previously expected. Because of that, the tasks had to be changed to start-to-start so the team could contact manufacturers about different steps in the treatment process all at the same time. The other main difference between the two schedules is that the life-cycle cost analysis subtask in the final design task was removed in the new schedule. It was discovered that the competition did not require a life-cycle cost analysis so only an EOPC and O & M cost analysis was completed. Overall, all tasks were completed on time so the entire duration of the project stayed the same.

9.0 Summary of Engineering Costs

9.1 Project Staffing

A preliminary staffing hours estimation was created in the project proposal and estimated 900 working hours spent on the project between the four project roles. Table 9-1 shows details of the distribution of hours estimated. A more detailed estimation of hours is shown in Appendix T-1. A significant amount of the hours was expected to be spent on Task 3, the treatment design. The EIT was expected to have the greatest number of hours of the team at 405 working hours estimated. The senior engineer was expected to have the least number of hours, at 88 hours. Table 9-2 shows the actual number of actual hours the team worked on for the project.

Table 9-1: Estimated Project Staffing Hours

Task	SENG	ENG	EIT	INT	Total
Task 1: Preliminary Assessment	1	1	40	35	77
Task 2: Site Assessment	3	13	11	5	32
Task 3: Treatment Design	24	105	214	67	410
Task 4: Final Design	14	45	85	16	160
Task 5: Project Impacts Analysis	1	4	0	0	5
Task 6: Project Deliverables	20	35	35	31	121
Task 7: Project Management	25	30	20	20	95
Total Hours	**88**	**233**	**405**	**174**	**900**

Table 9-2: Actual Project Staffing Hours

Task	SENG	ENG	EIT	INT	Total
Task 1: Preliminary Assessment	1	0	31	35	67
Task 2: Site Assessment	3	25	18	12	58
Task 3: Treatment Design	9	62.5	108	81.5	261
Task 4: Final Design	1	15	28	23	67
Task 5: Project Impacts Analysis	0	0	0	2	2
Task 6: Project Deliverables	16	55	68	54	193
Task 7: Project Management	26	39	50	34	169
Total Hours	**56**	**196.5**	**303**	**241.5**	**797**

Comparing the estimated hours to the real number of hours, the project team worked a total of 797 hours as shown by table 8-2. 103 hours less than what the preliminary estimate stated. The EIT had the greatest number of hours worked at 303 hours, 102 hours less than what was estimated. The intern had more hours worked than estimated, 241.5 hours instead of 174 hours. The task that had the greatest number of hours worked was task 3, treatment design at a total of 261 hours. Task 3 had the biggest difference between estimated hours and actual hours, 149 hours less than what was estimated. The task that was the closest to estimated hours was the project impact analysis which has a difference of 3 hours between actual to estimated hours. A more detailed description of the hours spent on the project is shown in Appendix T-2. Overall, the project management, project deliverables, and site assessment tasks took more hours than estimated while the rest of the tasks took less hours than estimated.

9.2 Design Budget

The team first developed an estimated design budget and then produced an actual design project budget analysis to account for real project costs over the course of the project. The estimated budget and the actual budget are shown in the following tables.

Table 9-3: Estimated Project Budget

1.0 Personnel	Classification	Hours	Rate, $/hr	Cost, $
	SENG	88	250	22,000
	ENG	233	190	44,270
	EIT	405	142	57,510
	INT	174	73	12,702
			Personnel Sub-total	**136,482**
2.0 Travel	**Classification**	**Items**	**Cost Per, $**	**Cost, $**
	Car Rental	3 Days	$34/day	102
	Mileage	2 Trips, 300 Miles Each	$0.40/mi	240
	Hotel	4 Rooms, 1 Night	$113/night	452
	Per Diem	6 Persons, 2 Days	$36.75/person/day	441
			Travel Sub-total	**1,235**
3.0 Supplies	**Classification**	**Items**	**Cost Per, $**	**Cost, $**
	Computer Lab	10 Days	$100/day	1,000
	3D Printing	500 grams	500 grams	60
			Supplies Sub-total	**1,060**
Total				**138,777**

Table 9-4: Actual Project Budget

1.0 Personnel	Classification	Hours	Rate, $/hr	Cost, $
	SENG	56	250	14,000
	ENG	195.5	190	37,145
	EIT	301	142	42,742
	INT	238.5	73	17,410.50
			Personnel Sub-total	**111,297.50**
2.0 Travel	**Classification**	**Items**	**Cost Per, $**	**Cost, $**
	Car Rental	2 Days	$34/day	68
	Mileage	2 Trips, 300 Miles Each	$0.40/mi	240
	Hotel	0 Rooms, 0 Nights	$113/night	0
	Per Diem	6 Persons, 2 Days	$36.75/person/day	441
			Travel Sub-total	**749**
3.0 Supplies	**Classification**	**Items**	**Cost Per, $**	**Cost, $**
	Computer Lab	0 Days	$100/day	0
	3D Printing	0 grams	500 grams	0
			Supplies Sub-total	**0**
Total				**112,046.50**

Based on the quantity of work accomplished and the way the responsibilities were distributed among the team, each personnel job has a different set of hours. Regarding the ranks of the number of hours each personnel had in the anticipated cost, the Senior Engineer had the fewest hours and the Engineer in Training the most, which is similar to the actual hours. The actual subtotal is less than the projected cost, coming in at about $111,297.50. Due to the team's decision not to stay overnight for the competition conference, the total travel time for the competition and site visit was reduced to 2 days from 3. That means the sub-total cost of the trip is $749 and there is no cost for the hotel stay. The proposed cost included supplies for 3D printing in the computer lab and the team chose not to do that, so the subtotal for the supplies is $0. Due to the reduced hours needed to design and not needing the supplies, the project's overall cost of $112,046.50 was less than the estimated cost.

10.0 Conclusion

The purpose of this project was to increase the Rainbow Valley Water Reclamation Facility capacity from 0.75 MGD to 3 MGD. The improved design needed to meet the permit limits that are stated in Appendix A. The team evaluated 2 – 3 alternatives for each step in the process to select the best treatment processes for the facility. The final design consisted of fine screen/vortex-grit chamber combined systems, an equalization basin, primary clarifiers, aeration basins, secondary clarifiers, disc filters, ultraviolet disinfection systems, and centrifuges. Based on the final design, the expected effluent value for BOD is 8.4 mg/L and the effluent values for TSS is 5 mg/L. The total construction cost for the project was estimated to be roughly $31.6 million and the annual operation and maintenance cost was estimated to be roughly $4.7 million. A construction phasing plan was created to ensure the existing facility could remain in operation while the expansions to the facility were being construction. The construction of the expansion was estimated to take roughly 10-12 months. Some additional improvements that could be implemented in the future is the potential to expand the emergency generator capabilities. This generator expansion may be needed in order to power the additional facilities that are being constructed. Another improvement that may be made in the future is the instillation of renewable energy sources to power the new facility. This could include wind or solar power to decrease the cost of energy to operate the facility. The proposed facility operates at a capacity of 3 MGD and meets all effluent permit limits.

Appendix A: RVWRF Provided Data

Appendix A-1: RVWRF Flow and Load Design Criteria

Appendix A-1: RVWRF Flow and Load Design Criteria (Waters, 2023)

Item	Criteria
Population Served	7732
Average waste water production rate, gpcd	97
Average Day Max Month Flow, mgd	0.75
Minimum Flow, mgd	0.30
Maximum Day Flow, mgd	1.50
Peak Hour Flow, mgd	*2.66*
BOD, mg/l Annual Average Max Month	 225 275
TSS, mg/l Annual Average Max Month	 250 300
TKN, mg/l-N Annual Average Max Month	 35 40
Temperature, deg C Average Winter Minimum Summer Maximum	 25 22 32

Appendix A-2: RVWRF Effluent Quality Limits

Appendix A-2: RVWRF Effluent Quality Limits (Waters, 2023)

Parameter	Allowable Limits
Class A+	
Fecal Coliform 7 sample median Single sample maximum	None detectable <23 CFU/100ml
Filtered Effluent Turbidity 24 hour average Maximum at any time	2 NTU 5 NTU
Total Nitrogen, (5-month geometric mean)	8 mg/l alert level <10 mg/l-N
Other	

Appendix A-3: RVWRF Phase 1 Operations and Maintenance Manual Values

Appendix A-3: RVWRF Phase 1 Operations and Maintenance Manual Values (Waters, 2023)

pH	6.5 – 9.0
Enteric Viruses	<125/40L
Entamoeba Histolytica	None detectable
Total Suspended Solids	<10 mg/l
Biochemical Oxygen Demand	<10 mg/l
Ascaris Lumbricoi Des	None Detectable
Common Large Tape Worm	None Detectable

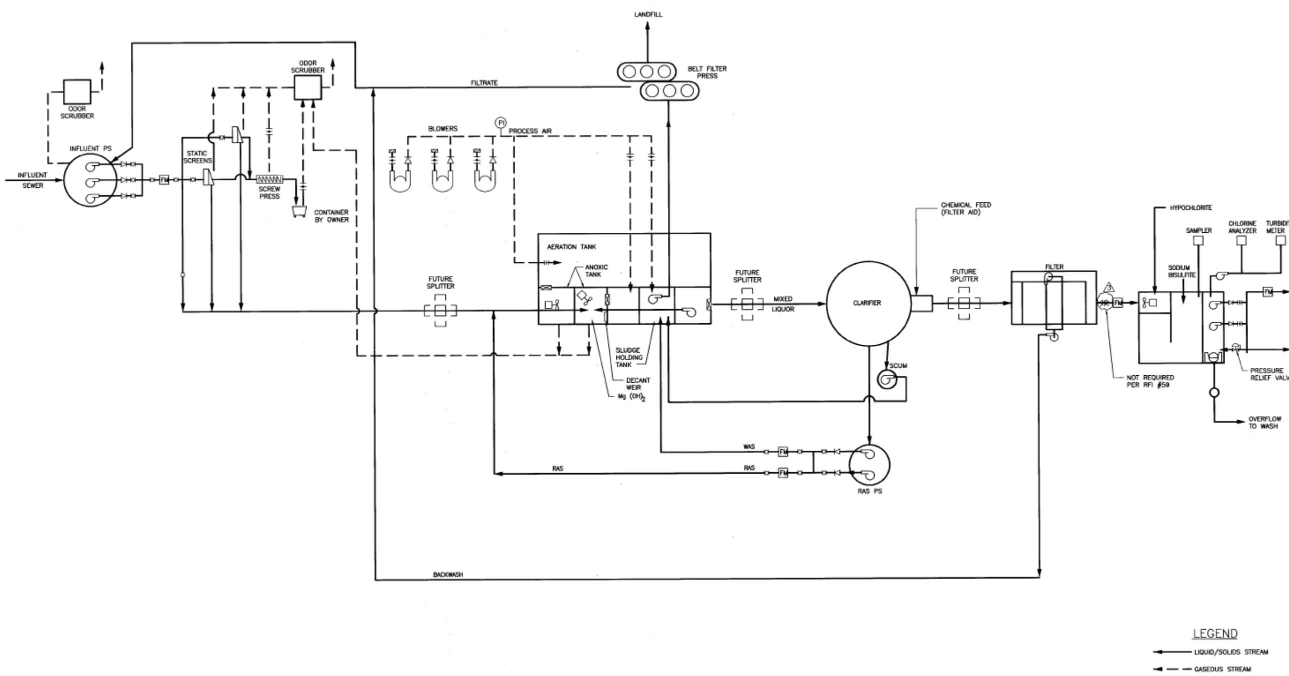
Appendix A-4: RVWRF List of Permits

Appendix A-4: RVWRF List of Permits (Waters, 2023)

Title	Permit No.	Issue Date	Expiration Date
Aquifer Protection Permit	P-105416	October 22, 2004	-N/A-
Arizona Pollution Discharge Elimination System Permit	AZ0025135	June 2, 2004	June 2, 2009
MCESD Air Quality Permit	040089	September 1, 2004	-N/A-

Appendix A-5: RVWRF Existing Flow Diagram

Appendix A-5: RVWRF Existing Flow Diagram (Waters, 2023)



Appendix B: Screening Alternatives

Appendix B-1: Fine Screen Example

Appendix B-1: Fine Screen (Parkson, 2022)

Appendix B-2: Static Screen Example

Appendix B-2 Static Screen (Vortex Engineering, 2024)

Appendix B-3: Step Screen Example

Appendix B-3 Step Screen (Pump Systems, 2020)

Appendix B-4: Detailed Decision Matrix for Screening Alternatives

Appendix B-4: Detailed Screening Decision Matrix

Preliminary Treatment (Screening)				
Criteria	**Weight (%)**	**Fine Screen**	**Step Screen**	**Static Screen**
Capital Cost	30	3	1	2
		$180,000.00	$250,000.00	$200,000.00
Maintenance & Operation	25	3	2	1
		Regular inspections to ensure proper functioning, mechanical/self-cleaning design, easy maintenance	Periodic inspection of step surfaces, no regular lubrication, adjustment of step spacing as needed, removal of accumulated debris	Frequent inspections of screens and damages/wear, self-cleaning design, chemical use for cleaning
Construction Time/Constructability	15	2	1	3
		Moderate construction time, prefabricated (involves welding or bolting), requires skilled labor for precise installation	Higher construction time, prefabricated (involves welding), mechanical components to install, less specialized labor	Shorter construction time due to straightforward design, prefabricated so simple installation process, minimal labor skills
Odor Control	10	2	3	1
		Are mostly installed with enclosures to route fouled air through an odor control system	Installed with enclosures and includes proper ventilation system to mitigate odors	Must be uncovered to clean, would need additional technologies to properly ventilate odors
Social & Environmental Impacts	10	2	3	1
		Good worker safety from minimized hazards, reduced risk of clogging downstream and has sustainable operation	Improves worker safety because of enclosed design, reduces odor efficiently, reduces wear of downstream equipment and has efficient screening operation	Enclosed system helps worker safety, limited flexibility for adjusting screens, prevents clogging, sustainable operation
Staffing	10	3	2	1
		Minimal staffing since they are self-cleaning and automated, requires little attention, operators inspect for damage while supervisors ensure proper functioning	Moderate staffing, need to monitor mechanical bars for specific spacing and maintain screens, remove accumulated debris, bars manually cleaned and inspected by maintenance staff, supervisors oversee efficient operation	Some staffing needed, regularly inspected and maintained by maintenance personnel, clean screen surface, need supervisors to oversee operation
Weighted Average	**100**	**2.65**	**1.75**	**1.6**

Appendix C: Grit Chamber Alternatives

Appendix C-1: Aerated Grit Chamber Example

Appendix C-1: Aerated Grit Chamber (SPIRAC, 2018)

Appendix C-2: Horizontal-Flow Grit Chamber Example

Appendix C-2: Horizontal-Flow Grit Chamber (Schreiber, 2022)

Appendix C-3: Vortex-Type Grit Chamber Example

Appendix C-3: Vortex-Type Grit Chamber (Huber, 2024)

Appendix C-4: Detailed Decision Matrix for Grit Chamber Alternatives

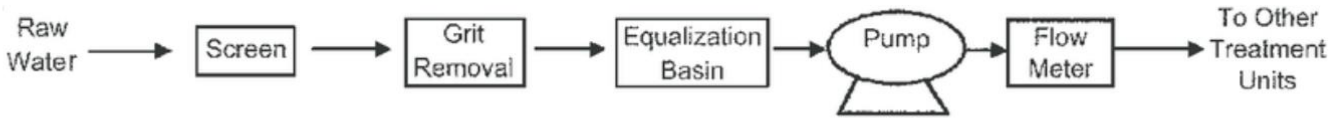
Appendix C-4: Detailed Grit Chamber Decision Matrix

Preliminary Treatment (Grit Chamber)				
Criteria	**Weight (%)**	**Aerated Grit Chamber**	**Horizontal Flow Grit Chamber**	**Vortex-Type Grit Chamber**
Capital Cost	25	3	2	1
		$134,000.00	$148,800.00	$186,000.00
Removal Efficiency	20	2	3	3
		Removal of particles greater than 0.21mm	Removal of particles greater than 0.2mm	Removal of particles greater than 0.2mm
Construction Time/Constructability	15	2	1	3
		Prefabricated and has moderate construction time, flexible constructability, has mechanical components and concrete structures	Moderate to long construction time, requires concrete channel/basin, not complicated construction, flexible and straight forward design, oldest and widely used type of grit removal	Prefabricated and has short construction time, good constructability, relatively straightforward design, requires skilled labor
Maintenance & Operation	10	2	1	3
		Requires additional labor for operation due to complexity of equipment	Extensive maintenance required due to excessive wear on equipment	Requires high-pressure agitation to loosen grit compacted in the sump
Footprint	20	2	1	3
		Relatively large due to aeration tank needed	Large land area required for long channel/basin required	Small land area required due to small equipment
Energy Consumption	10	1	3	2
		High energy consumption due to air being introduced at a high rate	Low energy consumption since flow is controlled to be slow to allow particles to settle	Moderate energy consumption needed for rotating turbine
Weighted Average	**100**	**2.15**	**1.85**	**2.4**

Appendix D: Equalization Basin Alternatives

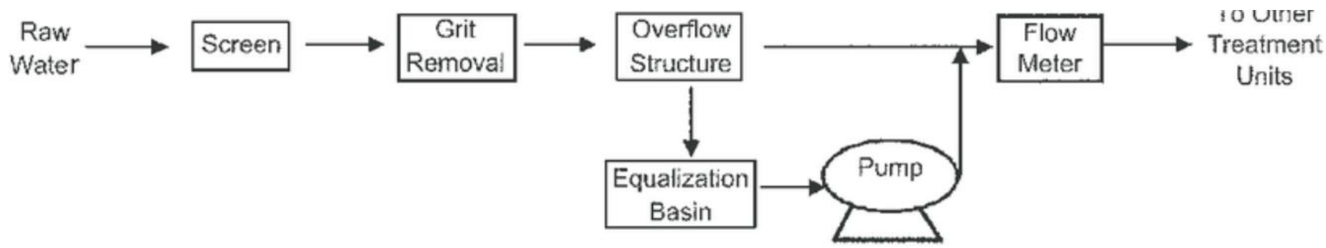
Appendix D-1: In-Line Basin Example

Appendix D-1: In-Line Basin Diagram (Goel, Flora, & Chen, 2007)



Appendix D-2: Side-Line Basin Example

Appendix D-2: Side-Line Basin Diagram (Goel, Flora, & Chen, 2007)



Appendix D-3: Detailed Decision Matrix for Equalization Basin Alternatives

Appendix D-3: Detailed Equalization Basin Decision Matrix

Preliminary Treatment (Equalization Basin)			
Criteria	**Weight (%)**	**In-Line Basin**	**Side-Line Basin**
Relative Cost	40	2	1
		No additional equipment and piping	Additional equipment and piping
Maintenance and Operation	25	2	1
		No additional equipment and piping	Additional equipment and piping
Construction Time/Constructability	20	2	1
		No additional equipment and piping	Additional equipment and piping
Staffing	15	2	1
		No additional equipment and piping	Additional equipment and piping
Weighted Average	**100**	2	1

Appendix E: Primary Clarifier Alternatives

Appendix E-1: Bridge-Support Clarifier Example

Appendix E-1: Bridge-Support Clarifier (Bridge Support Clarifiers, 2024)

Appendix E-2: Column-Support Clarifier Example

Appendix E-2: Column-Support Clarifier (Column Supported Clarifiers, 2024)

Appendix E-3: Detailed Decision Matrix for Primary Treatment Alternatives

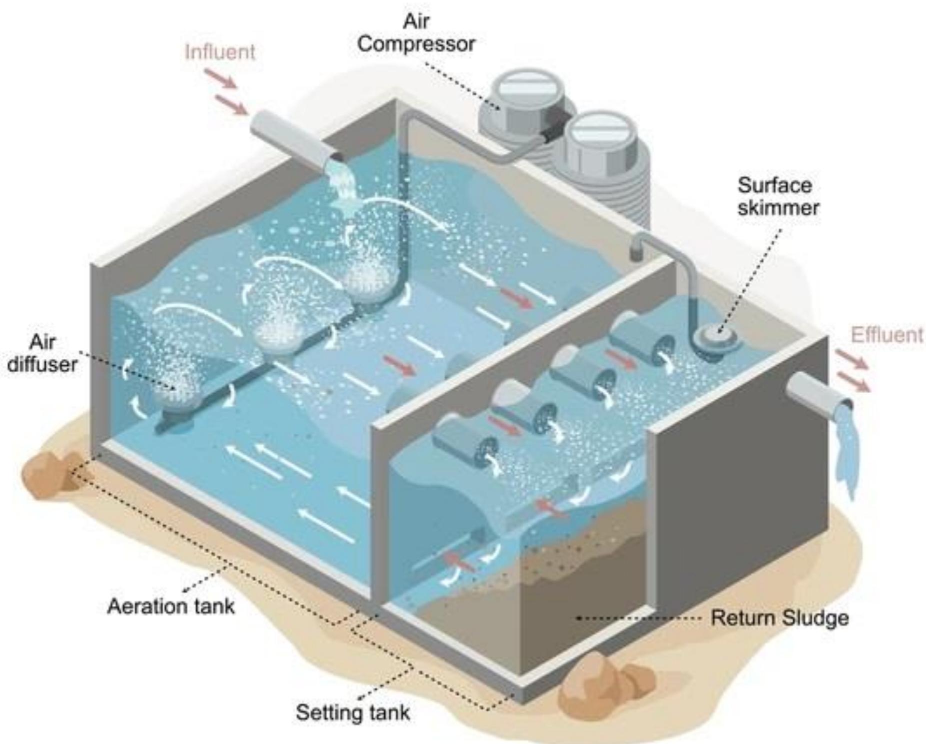
Appendix E-3: Detailed Primary Clarifier Decision Matrix

Primary Treatment (Primary Clarifier)			
Criteria	**Weight (%)**	**Bridge Support Clarifier**	**Column Support Clarifier**
Capital Cost	40	1	2
		65' diameter~ $450,000	65' diameter~ $314,000
Surface Area Requirements	25	1	2
		Multiple clarifiers <40' diameter	One clarifier >40' diameter
Construction Time/Constructability	20	1	2
		Full span bridge	Half span bridge
Maintenance & Operation	15	2	1
		Supports accessible by bridge	Supports submerged
Weighted Average	**100**	**1.15**	**1.85**

Appendix F: Secondary Treatment Alternatives

Appendix F-1: Conventional Activated Sludge Example

Appendix F-1: Conventional Activated Sludge (Aeration, 2024)

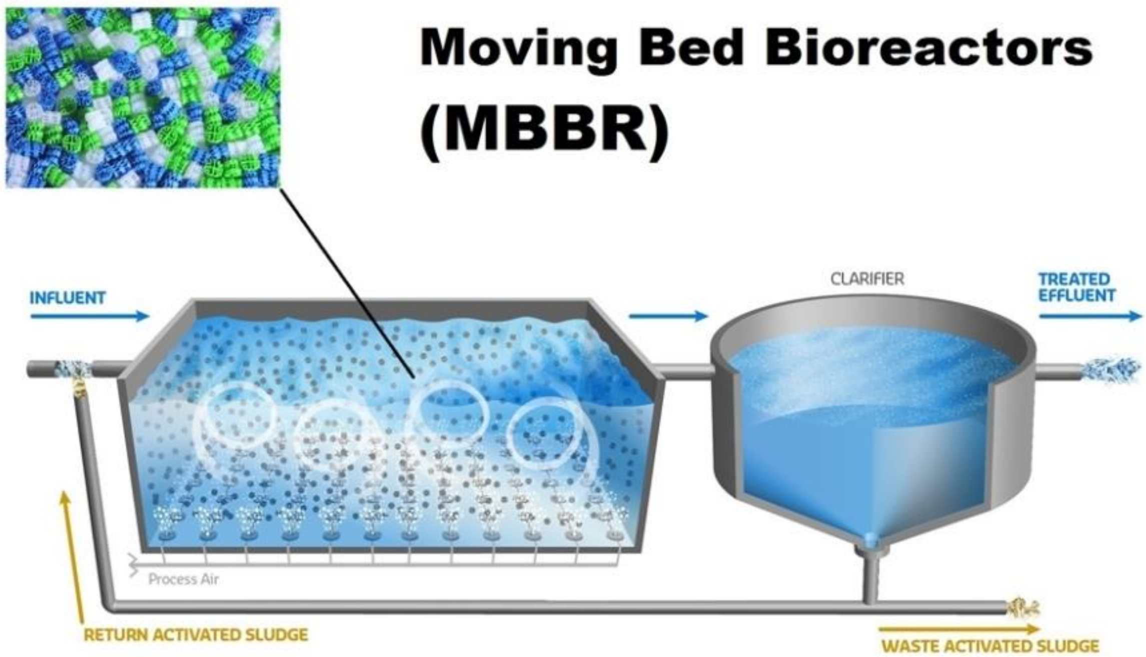


Appendix F-2: Membrane Bioreactor Example

Appendix F-2: Membrane Bioreactor (Evoqua, 2024)

Appendix F-3: Moving Bed Bioreactor Example

Appendix F-3: Moving Bed Bioreactor (Gustawater, 2023)



Appendix F-4: Detailed Decision Matrix for Secondary Treatment Alternatives

Appendix F-4: Detailed Secondary Treatment Decision Matrix

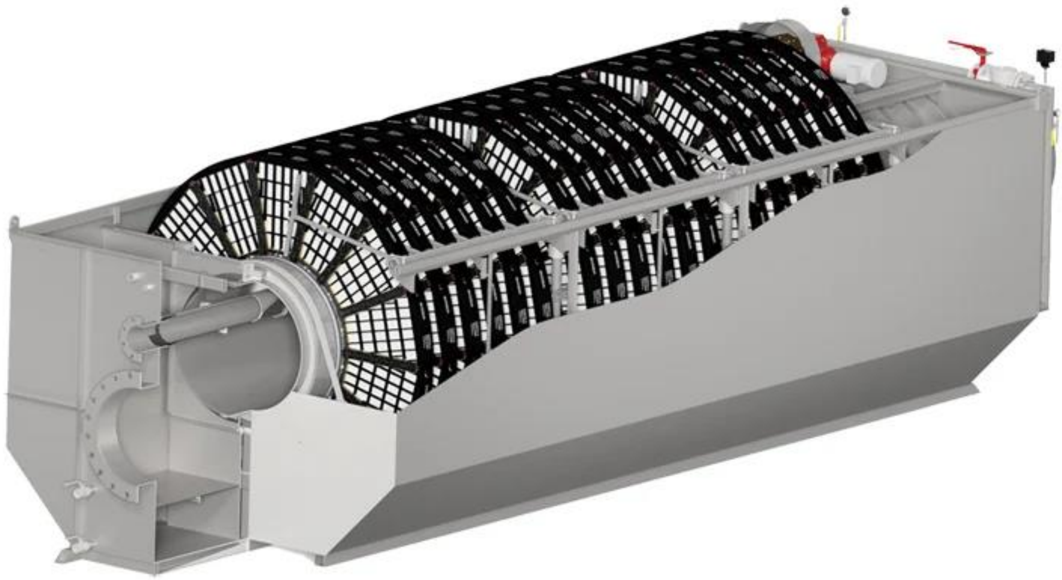
Secondary Treatment				
Criteria	**Weight (%)**	**Conventional Activated Sludge**	**Membrane Bioreactor**	**Moving Bed Bioreactor**
Capital Cost	20	1	3	2
		$11,000,000.00	$4,431,818.00	$6,352,500.00
Maintenance & Operation Cost	25	3	1	1
		Would require 5 treatment trains to operate. A continuous and well-timed supply of oxygen is required during operation. No media or filters to clean. Blowers may need to be inspected 1-2 times a year to ensure proper aeration is completed. Small maintenance and operation costs.	Would require 36 small treatment trains to operate. Require in-place membrane cleaning 2-4 times per year. Air scour is also used to clean the membranes. They can be cleaned in the MLSS so does not require the basin to be drained. Continuous aeration and sludge management is required. Higher maintenance and operation costs because of the units required.	Would require 42 small units to maintain and operate. Cleaning of biofilm on the media is required frequently. Sludge removal in the system is required along with continuous aeration. Relatively higher maintenance and operation costs because of the number of units required.
Construction Time/Constructability	25	2	1	2
		5 treatment trains required (1 train existing, 4 new to construct). Concrete tanks must be constructed on site. Assembly units like pumps, motors, pipes, and blowers must be installed. Requires relatively large construction time. Less excavation required because the existing facility would be utilized.	36 treatment trains required. Concrete tanks must be constructed on site. Membrane unit is prefabricated and can be installed by local technicians. Requires a long construction time because of the number of tanks that need to be built. Existing infrastructure will have to be demolished.	42 units are required. Prefabricated units available that can be installed by local technicians. Placement of 42 units will take a long time. Existing infrastructure would have to be demolished.
Life Cycle Cost	15	3	1	1

		Low life-cycle assessment (LCA) due to low operating and maintenance requirements.	Higher LCA as compared to CAS due to high electricity requirement and low capacity, moderate cost for maintenance, lower initial capital and operating cost compared to MBBR	Highest life cycle cost due to high electricity requirement and low capacity. Similar operating and maintenance cost to MBR, moderate membrane replacement costs but generally higher capital cost than MBR
Footprint	10	1	2	3
		Relatively large footprint. Approx 39272 additional square ft required for 3 MGD	Larger footprint than moving bed but smaller than CAS. Approx 15618 square ft required for 3 MGD. Additional square footage between units will be required	Smaller footprint but requires more facilities. Approx 10510 square ft required for 3 MGD. Additional square footage between units will be required
Removal Efficiency	5	1	2	3
		Meets almost all (≈90%) NPDES permit discharge limitations except for fecal coliform (requires additional disinfection). NPDES limits: BOD of 30 mg/L, meets TSS of 30-45 mg/L, achieves pH range of 6-9, meets limit residual chlorine of 0.5 mg/L, fecal coliform of 200/100 mL (30 day mean) or 400/100 mL (max daily), meets 40 mg/L TKN, achieves ammonia of 10 mg/L, achieves P limit of 5 mg/L.	Meets all (90%) NPDES permit discharge limitations: BOD of 10 mg/L, TSS of 10 mg/L, pH between 6.5 and 8.5, ammonia Nitrogen of 5 mg/L, fecal coliform of less than 200 MPN/100 mL, P range of 2-5 mg/L	Meets all (>90%) NPDES permit discharge limitations and has wide range. BOD of 20 mg/L, meets TSS of 20 mg/L, pH between 6.5 and 8.5 ammonia Nitrogen of 10 mg/L, fecal coliform of 200-1000 MPN/100 mL, P of < 1 mg/L
Weighted Average	**100**	**2.05**	**1.55**	**1.75**

Appendix G: Advanced Treatment Alternatives

Appendix G-1: Disc Filter Example

Appendix G-1: Disc Filter (Evoqua, 2024)



Appendix G-2: Sand Filter Example

Appendix G-2: Sand Filter (Evoqua, 2024)

Appendix G-3: Detailed Advanced Treatment Decision Matrix

Appendix G-3: Detailed Advanced Treatment Decision Matrix

Advanced Treatment			
Criteria	**Weight (%)**	**Disc Filters**	**Sand filters**
Capital Cost	30	2	1
		$720,000.00	$1,080,000.00
Constructability/Construction Time	10	2	1
		Parts are prefabricated by the manufacture and assembled on site	Concrete for treatment basin will need to be cast onsite, pipes, pumps and underdrain will be installed
Maintenance & Operation	25	2	1
		Requires lubrication and replacement of parts and back washing of discs	Requires backwashing of soil media, inspections of pumps, and occasional replacement of soil
Removal Efficiency	35	2	1
		Removal of particles larger than 10 microns, removes nearly all BOD and TSS	Removes most of the TSS and BOD in the water
Weighted Average	**100**	**2**	**1**

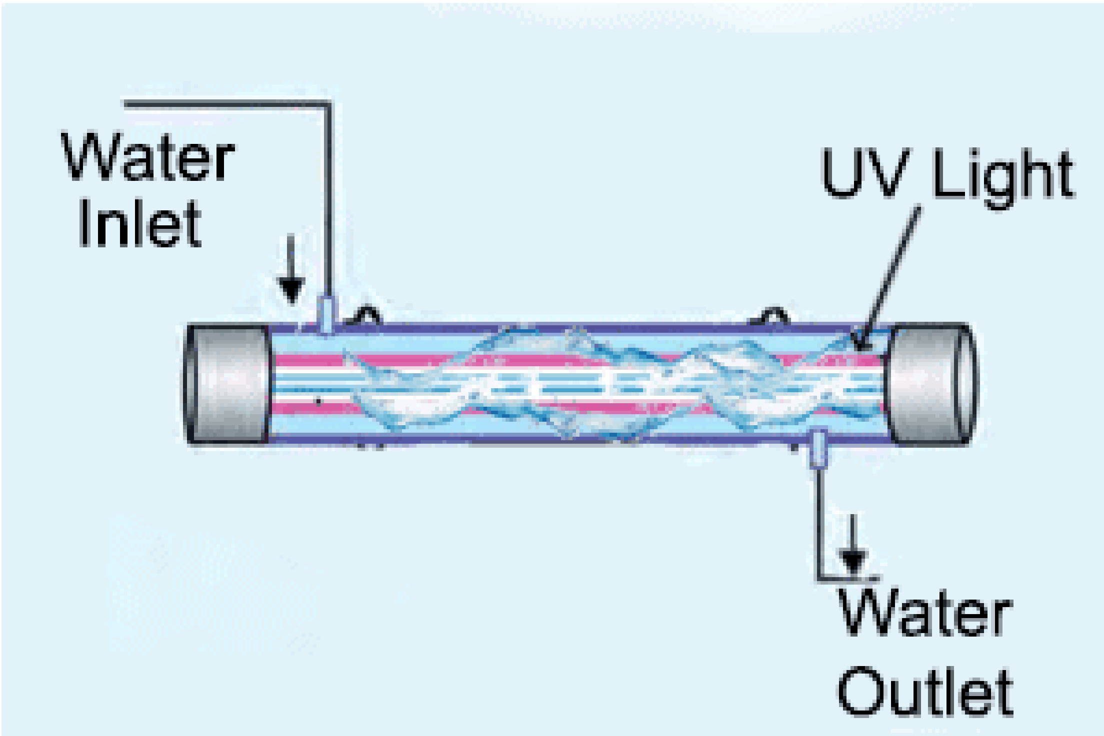
Appendix H: Disinfection Alternatives

Appendix H-1: Chlorination Contact Basin Example

Appendix H-1: Chlorination Contact Basin (Eawag, 2020)

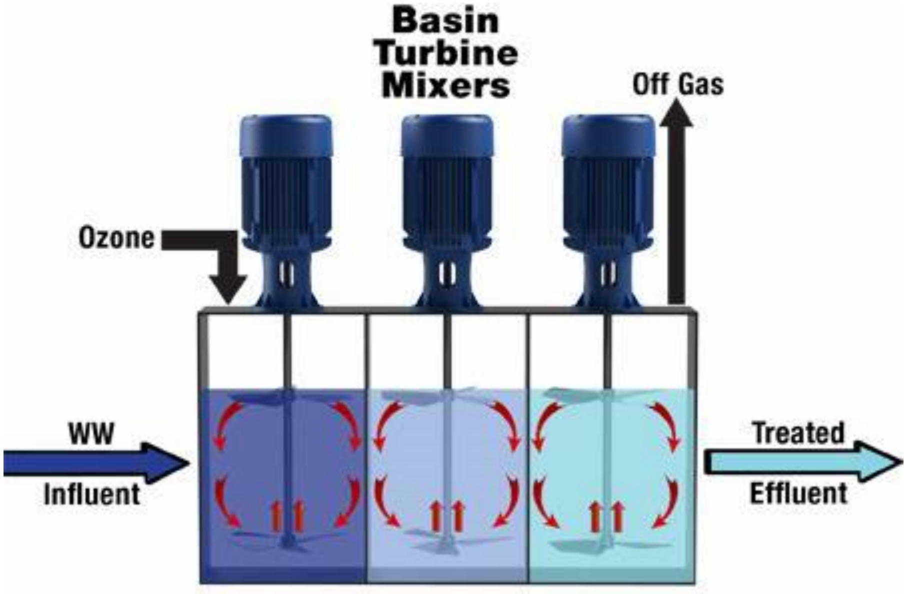
Appendix H-2: Ultraviolet Disinfection Example

Appendix H-2: Ultraviolet Disinfection (Alfaa UV, 2022)



Appendix H-3: Ozone Disinfection Example

Appendix H-3: Ozone Disinfection (Mazzei Injector Company, LLC, n.d.)



Appendix H-4: Detailed Decision Matrix for Disinfection Alternatives

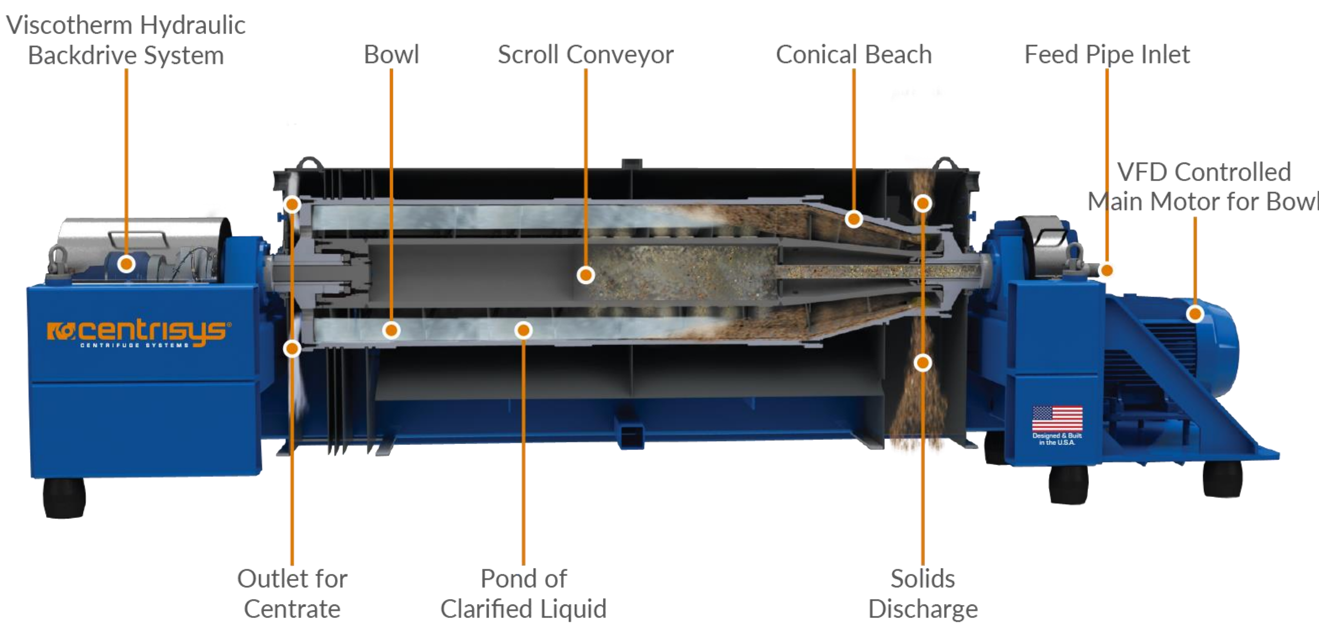
Appendix H-4: Detailed Disinfection Decision Matrix

Disinfection				
Criteria	**Weight (%)**	**Chlorination Tank**	**UV**	**Ozone**
Relative Cost	30	2	3	1
		Cost for large contact tank and chemicals	Cost for equipment (less than chlorination)	The cost of treatment can be relatively high in capital and in power intensiveness
Surface Area Requirements	20	1	3	2
		Most area required for effective disinfection	Equipment requires less space than other methods	Three tanks required for ozone treatment
Social & Environmental Impacts	10	1	3	2
		Even at low concentrations, chlorine is toxic to aquatic life. Can produce large chemical smell	Physical process, so no residual effect that can be harmful to humans or aquatic life	No harmful residuals that need to be removed
Maintenance & Operation	15	**3**	**2**	**1**
		More cost effective than UV or ozone when dechlorination is not required	UV is user-friendly for operators; preventative maintenance program is necessary to control fouling of tubes	Ozone is generated onsite, so there are fewer safety problems with shipping and handling, but more complex technology, very corrosive and reactive
Disinfection Rate	25	**1**	**2**	**3**
		Can prolong disinfection even after initial treatment and can be measured to evaluate the effectiveness	Effective at inactivating most viruses, spores, and cysts	More effective than chlorine in destroying viruses and bacteria
Weighted Average	**100**	**1.6**	**2.6**	**1.8**

Appendix I: Solids Management Alternatives

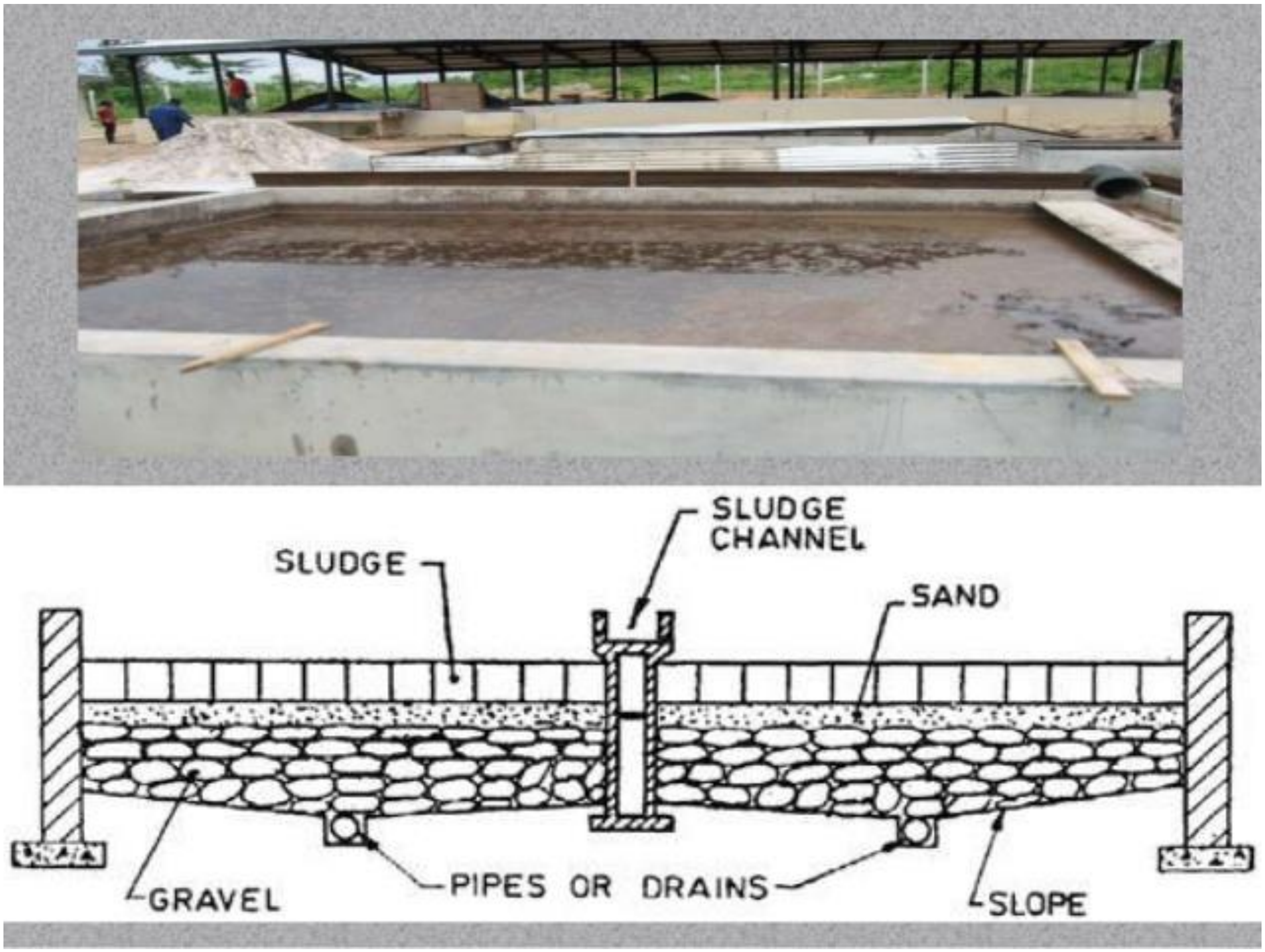
Appendix I-1: Centrifuge Example

Appendix I-1: Decanter Centrifuge (A Comprehesive Guide to Decanter Centrifuge Operation, Service, Maintenance, and Repair, 2024)



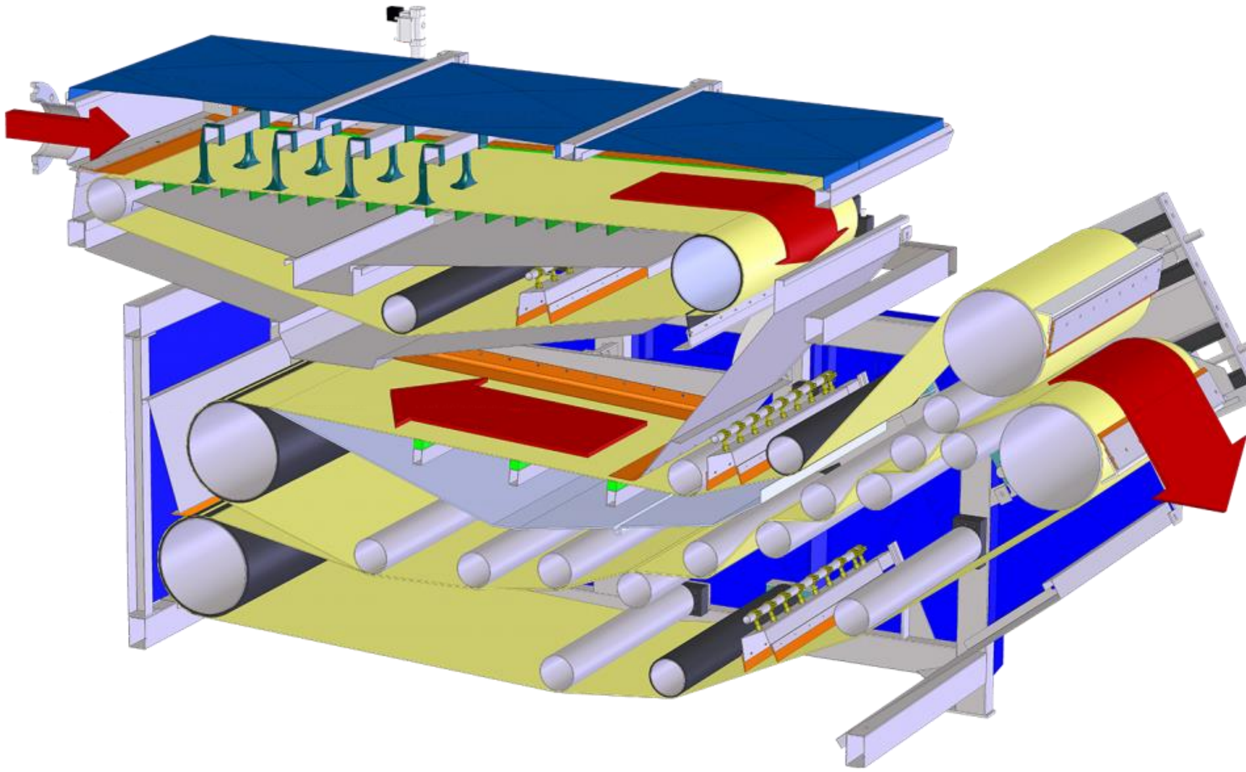
Appendix I-2: Drying Bed Example

Appendix I-2: Drying Bed (Sludge Drying Beds , 2018)



Appendix I-3: Filter Press Example

Appendix I-3: Filter Press (Belt Filter Press N-PD XL, 2024)



Appendix I-4: Detailed Decision Matrix for Solids Management Alternatives

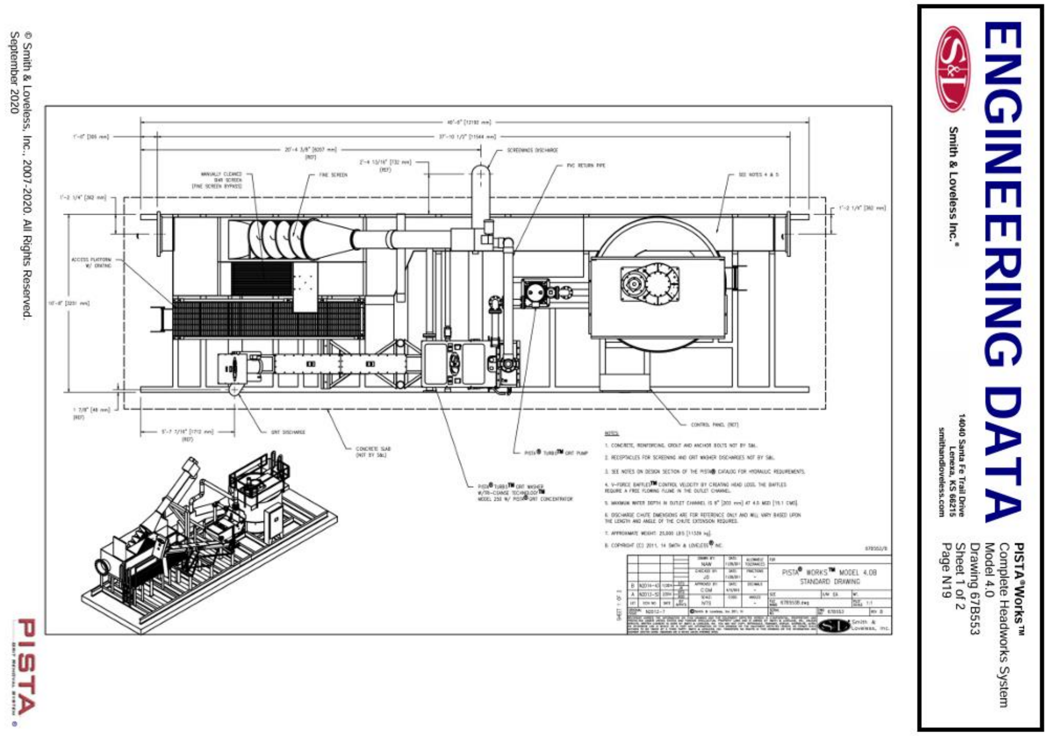
Appendix I-4: Detailed Solids Management Decision Matrix

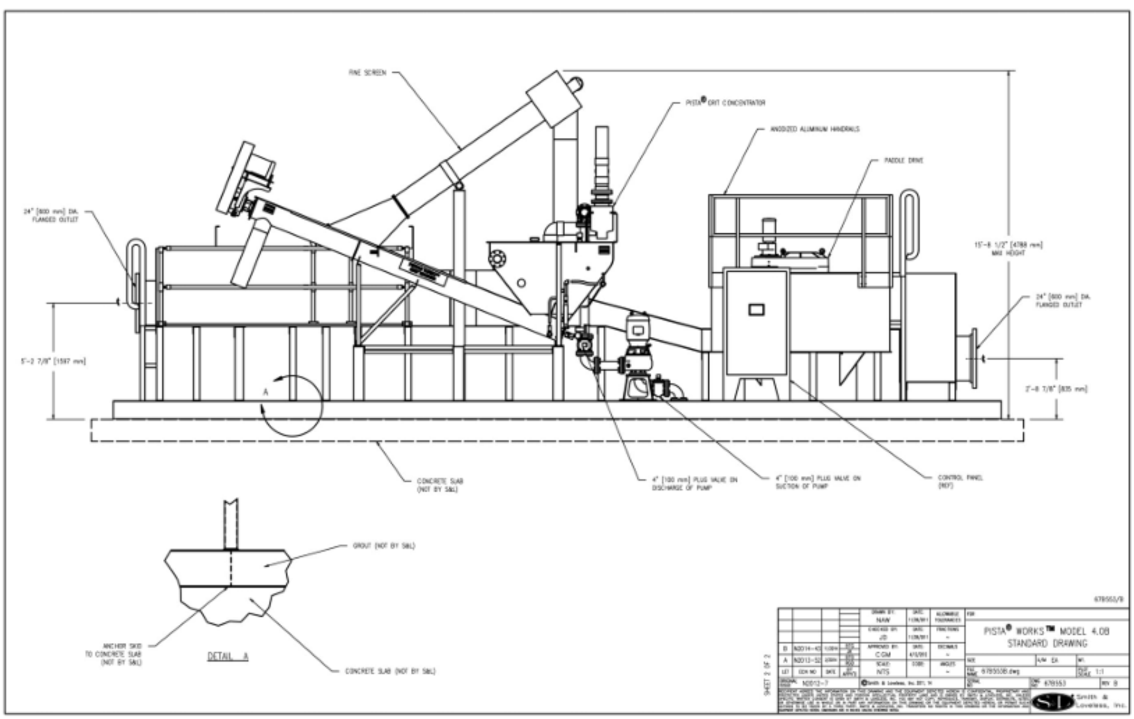
Solids Management				
Criteria	**Weight (%)**	**Centrifuge**	**Drying Beds**	**Filter Press**
Relative Cost	30	2	1	3
		Capital costs are more than a belt press, but operation and maintenance costs can be less expensive. High energy consumption	No energy consumption, only need to build the beds. Relatively low capital cost	Low energy consumption but requires a larger footprint
Environmental/Social Impacts	10	3	1	2
		Fairly noisy, small and unnoticeable	No noise produced but may look concerning to the public, odor and insect activity may be an issue	Less noise produced than centrifuges, odor is sometimes an issue
Drying Time	20	3	1	2
		<20 minutes	Days to weeks	>1-2 hours
Surface Area Requirements	25	3	1	2
		Smallest footprint	Large land area required	Larger than a centrifuge but smaller than drying beds
Maintenance & Operation	15	2	3	1
		Requires minimal operator attention and is easy to clean. Operations can be fully automated but starting the bowl is usually done manually.	Sludge removal is labor intensive and time consuming. Clogging of the sand and gravel bed is common which doesn't allow the liquid to drain	Can be started and stopped quickly compared to centrifuges, require more operator attention. Requires belt washing which is time consuming. Belts may need to be replaced; average belt life is 2700 running hours.
Weighted Average	**100**	**2.55**	**1.3**	**2.15**

Appendix J: Preliminary Treatment Design

Appendix J-1: Spiral Fine Screen and Baffled Vortex Grit Chamber Product Drawings

Appendix J-1: Record Drawings for Spiral Fine Screen and Vortex Grit Chamber (Loveless, 2012)





Appendix K: Equalization Basin Design

Appendix K-1: Equalization Basin Design Calculations

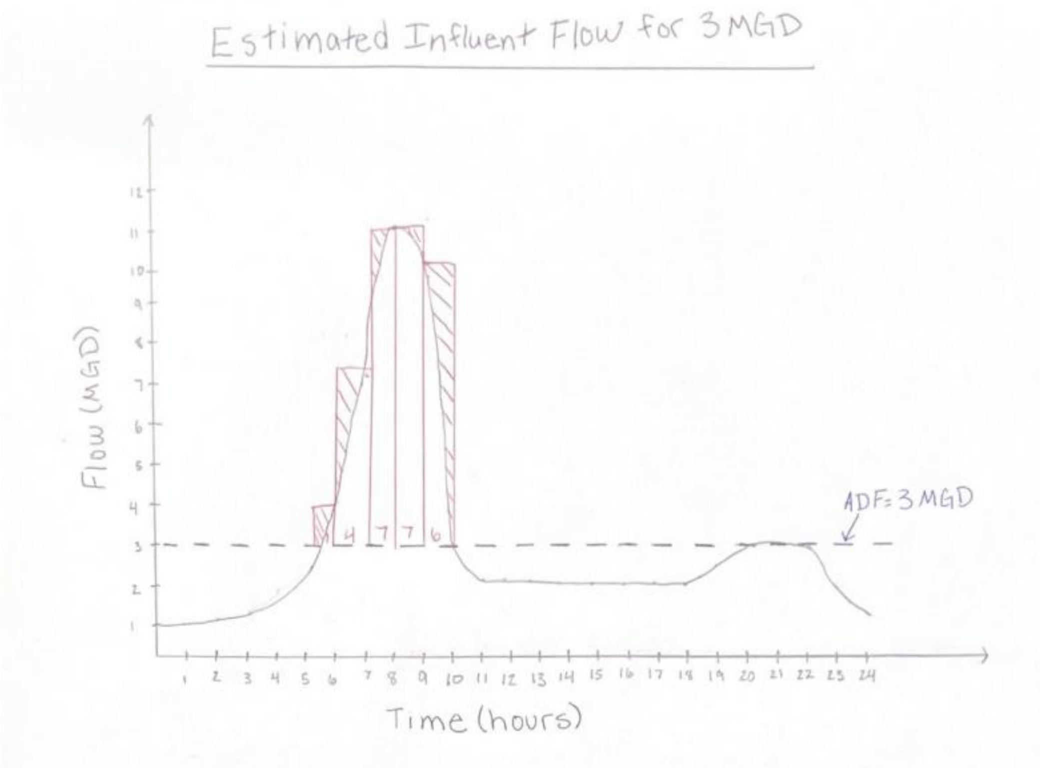
Appendix K-1: Equalization Basin Design Calculations

Peak Hour Flow for 0.75 MGD = 2.66 MGD

$$\frac{2.66\ MGD}{0.75\ MGD} = \frac{x}{3\ MGD}$$

$$x = 10.6\ MGD$$

*Assume 10.64 MGD as the peak hour flow for 3 MGD



<u>Area Under the Curve and Above ADF Estimation:</u>

$$1 + 4 + 7 + 7 + 6 - 2 = 23\ MGD$$

$$\frac{23\ MGD}{24\ hr/day} = 0.958\ MG/hr$$

$$\frac{0.958\ MG/hr}{5\ hr} = 0.192\ MG = 25622.151\ ft^3$$

<u>Dimensions:</u>

$$L \times W \times D$$

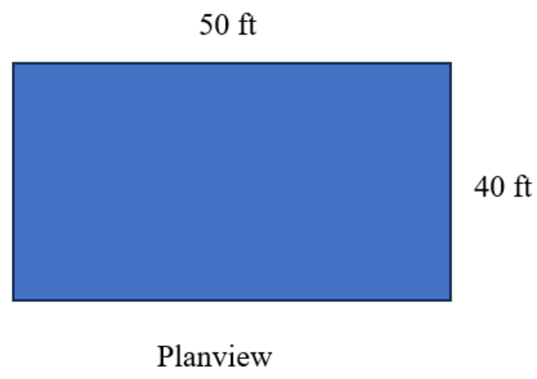
$$50\ ft \times 40\ ft \times 15\ ft$$

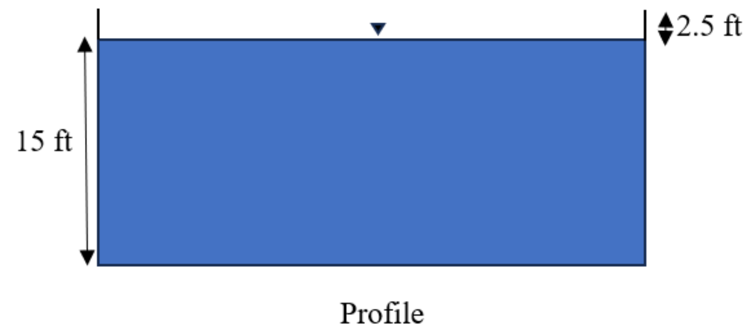
<u>Air Requirements:</u>

$$Air\ needed = 0.02 \times V$$

$$Air\ needed = 0.02 \times 25622.151 ft^3$$

$$Air\ needed = 512.44\frac{ft^3}{min}$$





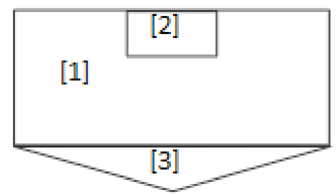
Freeboard Calculations:

$$Volume\ of\ the\ Tank = 50 \times 40 \times 15 = 30{,}000\ ft^3$$
$$Volume\ of\ Inffluent = 25622.151\ ft^3$$
$$Volume\ of\ Empty\ Space = 30000 - 25622.151 = 4377.849 ft^3$$
$$Freeboard = \frac{4377.849 ft^3}{50\ ft \times 40 ft} = 2.5\ ft$$

Appendix L: Primary Treatment Design

Appendix L-1: Primary Clarifier Design Calculations

Appendix L-1: Primary Clarifier Design Calculations



[1] $\text{Cylinder} = \pi r^2 h$
$Where{:}\, r = radias\ (ft), h = height(ft)$
$\pi \times (32.5ft)^2 \times 10.167ft = \mathbf{33737.23\ ft^3}$

[2] $Feedwell = \pi r^2 h$
$Where{:}\, r = radias\ (ft), h = height(ft)$
$\pi \times (8ft)^2 \times 5.5ft = \mathbf{1105.84\ ft^3}$

[3] $Cone = \pi r^2 \frac{h}{3}$
$Where{:}\, r = radias\ (ft), h = height(ft)$
$\pi \times (32.5ft)^2 \times \frac{2.71ft}{3} = \mathbf{2997.54\ ft^3}$

$Total\ clarifier\ volume = 33737.33 + 2997.54 - 1105.84 = \mathbf{35628.7ft^3 = 266521.18\ gallons}$

$$Detention\ Time = \frac{Tank\ volume}{Flow\ rate} = \frac{266521.18\ gallons}{3\ MGD} = 0.089 = \mathbf{2.13\ hours}$$

$Surface\ Area = \pi r^2$
$Where{:}\, r = radias\ (ft)$
$\pi \times (32.5ft)^2 = \mathbf{3318.31\ ft^2}$

$$Surface\ Overflow\ Rate = \frac{Flow\ Rate}{Surface\ Area} = \frac{3\ MGD}{3318.31\ ft^2} = \mathbf{904.07 gpd/ft^2}$$

$$Weir\ Overflow\ Rate = \frac{Flow\ Rate}{Length\ of\ Weir} = \frac{3\ MGD}{204.2\ ft} = \mathbf{14691.48 gpd/ft}$$
$Length\ of\ Weir = \pi \times d = \pi \times 65' = 204.2\ ft$
$Where\ d = diameter\ (ft)$

Energy Consumption

3/4 HP motor requires 0.559 kW per hour.

$$0.559 \frac{kW}{hour} \times 24\ hours = \mathbf{13.42 \frac{kW}{day}}$$

Settling Velocity

Particle size: Diameter= 0.2mm

Specific gravity= 2.65

Average water temperature= 25 °C

Water density $(25\ °C) = 997.049\frac{kg}{m^3} = 1000\ kg/m^3$

Dynamic Viscosity $(25\ °C) = 0.890\ mPa \cdot s = 0.890 \times 10^{-3}\ Pa \cdot s$

Stokes law:

$$V_s = \frac{g(\rho_s - \rho)d^2}{18\mu}$$

Where:

$g = Acceleration\ due\ to\ gravity\ \left(\frac{m}{s^2}\right)$

$\rho_s = Density\ of\ the\ particle\ \left(\frac{kg}{m^3}\right)$

$\rho = Density\ of\ the\ water\ \left(\frac{kg}{m^3}\right)$

$d = diameter\ of\ the\ particle\ (mm)$

$\mu = viscosity\ of\ the\ water\ (Pa \cdot s)$

$$V_s = \frac{\left(9.81\frac{m}{s^2}\right)\left(2650\frac{kg}{m^3} - 1000\frac{kg}{m^3}\right)(2 \times 10^{-4} m)^2}{18(8.90 \times 10^{-4}\ Pa \cdot s)} = 4.04 \times 10^{-2}\frac{m}{s}$$

Check R:

$$R = \frac{d(v_s)}{v}$$

Where:

$v = kinematic\ viscosity\ \left(\frac{m^2}{s}\right)$

$d = diameter\ of\ particle\ (m)$

$v_s = velocity\ of\ the\ particle\ (\frac{m}{s})$

$kinematic\ viscosity\ (25\ °C) = 0.893\ \times 10^{-6}\frac{m^2}{s}$

$$R = \frac{(2.0 \times 10^{-4}\ m)\left(4.04 \times 10^{-2}\frac{m}{s}\right)}{0.893 \times 10^{-6}\frac{m^2}{s}} = 9.05$$

**R is in the transition range so Stokes law is not valid, must use Newtons equation*

Check Cd:

$$C_D = \frac{24}{R} + \frac{3}{R^{\frac{1}{2}}} + 0.34$$

Where:

$C_D = Drag\ coefficient$

$R = Reynolds\ number$

$$C_D = \frac{24}{9.05} + \frac{3}{9.05^{\frac{1}{2}}} + 0.34 = 3.99$$

Newtons equation for settling velocity:

$$v_s = [\frac{4g(\rho_s - \rho)d}{3C_D\rho}]^{1/2}$$

Where:

$g = Acceleration\ due\ to\ gravity\ \left(\frac{m}{s^2}\right)$

$\rho_s = Density\ of\ the\ particle\ \left(\frac{kg}{m^3}\right)$

$\rho = Density\ of\ the\ water\ \left(\frac{kg}{m^3}\right)$

$d = diameter\ of\ the\ particle\ (mm)$

$C_D = Drag\ coefficient$

A solver in excel was used to complete iterations of these calculations. The R value of 9.05 was used for the starting R value the calculate a new settling velocity. The new settling velocity is used to calculate a new R value. The process is continued until the value of R used to calculate the velocity matches the check of the Reynolds number.

*Final Settling Velocity = **0.0286 m/s = 0.0938 ft/s***

How long will it take this particle to settle in the primary clarifier?

Side water depth= 10'2"= 10.167'

Settling time= $\frac{10.167ft}{0.0938\frac{ft}{s}} = 108.39\ seconds = \mathbf{1\ minutes\ 48.39\ seconds}$

Compare overflow rate to settling velocity:
**The settling velocity must be faster than the overflow rate to ensure that the particle have time to settle in the clarifier before the water flows out of the clarifier*

Overflow rate: 904.07 gpd/ft^2
Settling velocity: 0.0938 ft/s

Convert overflow rate to ft/s:

$1\frac{ft}{s}=7.4805\frac{gpd}{ft^2}$

$$\frac{1}{7.4805}=\frac{x}{904.07}$$
x=120.86 ft/d = 0.0014 ft/s

0.0014 ft/s < 0.0938 ft/s ***OK!***

Removal of TSS and BOD primary clarifiers:

Approximate TSS removal in primary treatment: 50-65%
Approximate BOD removal in primary treatment: 25-40%

Influent in primary clarifier:
TSS= 250 mg/l
BOD= 225 mg/l

Effluent from primary clarifier:
TSS=$250\frac{mg}{l}\times 0.50 = \boldsymbol{125\frac{mg}{l}}$
BOD= $225\frac{mg}{l}\times 0.75 = \boldsymbol{168.75\frac{mg}{l}}$

Appendix L-2: Primary Clarifier Settling Velocity Excel Solver

Appendix L-2: Primary Clarifier Settling Velocity Excel Solver

Diameter	2.00E-04	m
Particle density	2650	kg/m^3
Water density	1000	kg/m^3
Temperature	25	C
Dynamic Viscosity	8.90E-04	Pa-s
Kinematic Viscosity	8.93E-07	m^2/s

Stokes' Settling Velocity

v(s) = 0.040416 m/s

Check Reynolds number

R = 9.05E+00

Because R > 1 must use Newtons equation and iterate

Use Solver

Set up the equations below and enter the value of R from B18 as a first guess

R = 6.41E+00

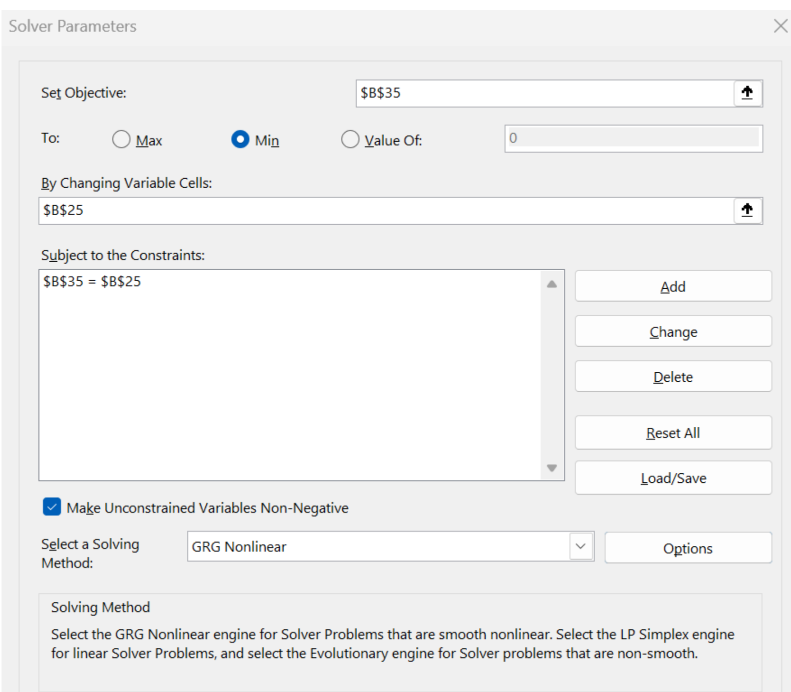
Calculate Newton's drag coefficient for R between 0.5 and 10^4

Cd = 5.27E+00

v(s) = 2.86E-02 m/s

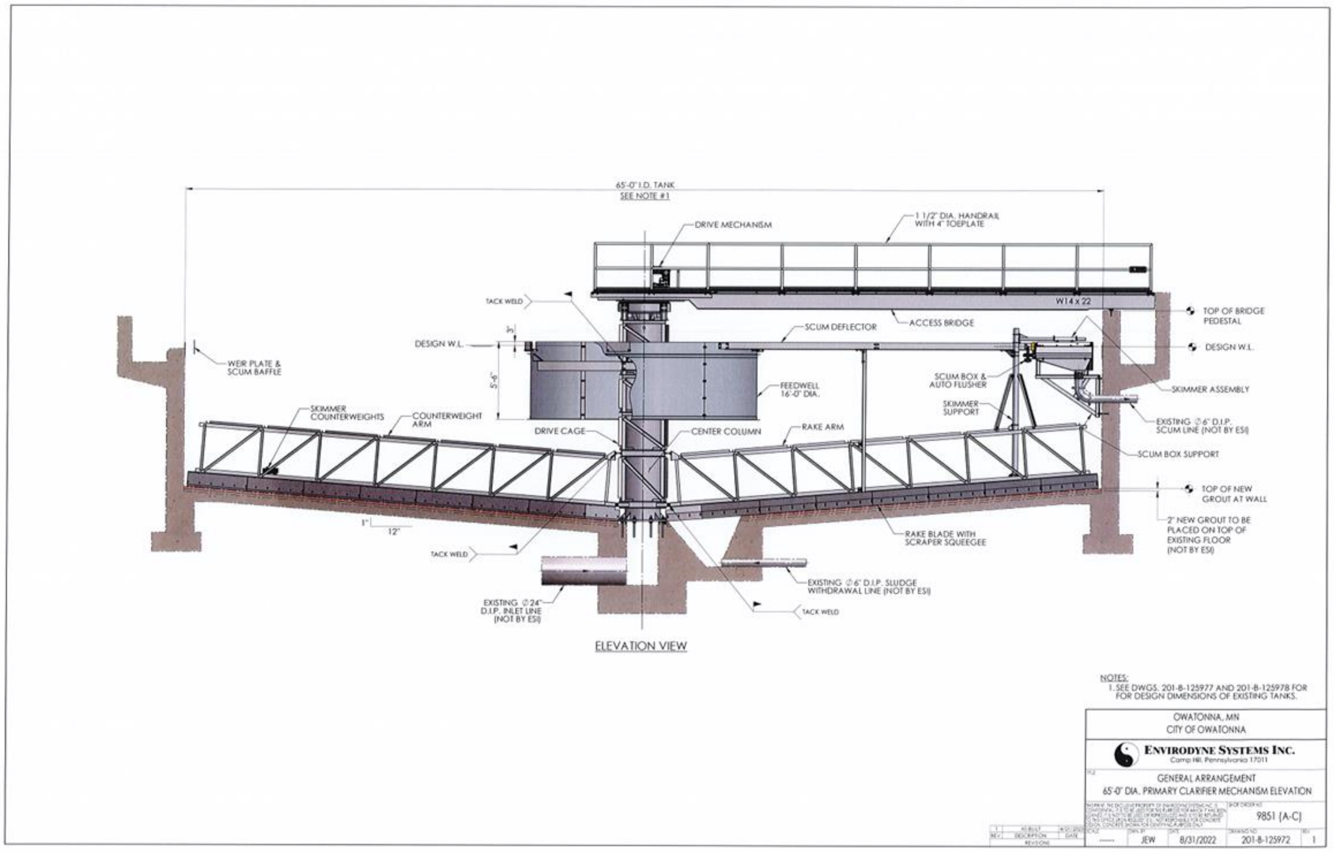
Check the Reynolds number

R = 6.41E+00



Appendix L-3: Primary Clarifier Record Drawings

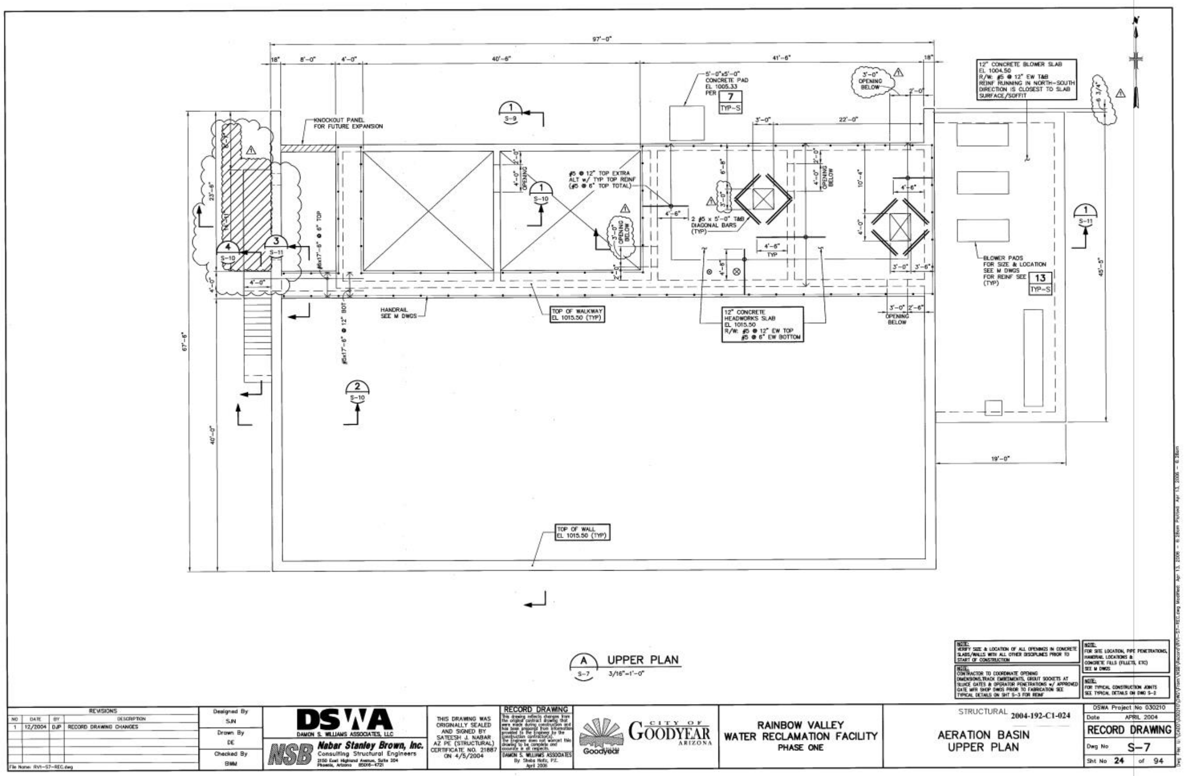
Appendix L-3: Primary Clarifier Record Drawings (Envirodyne, 2022)

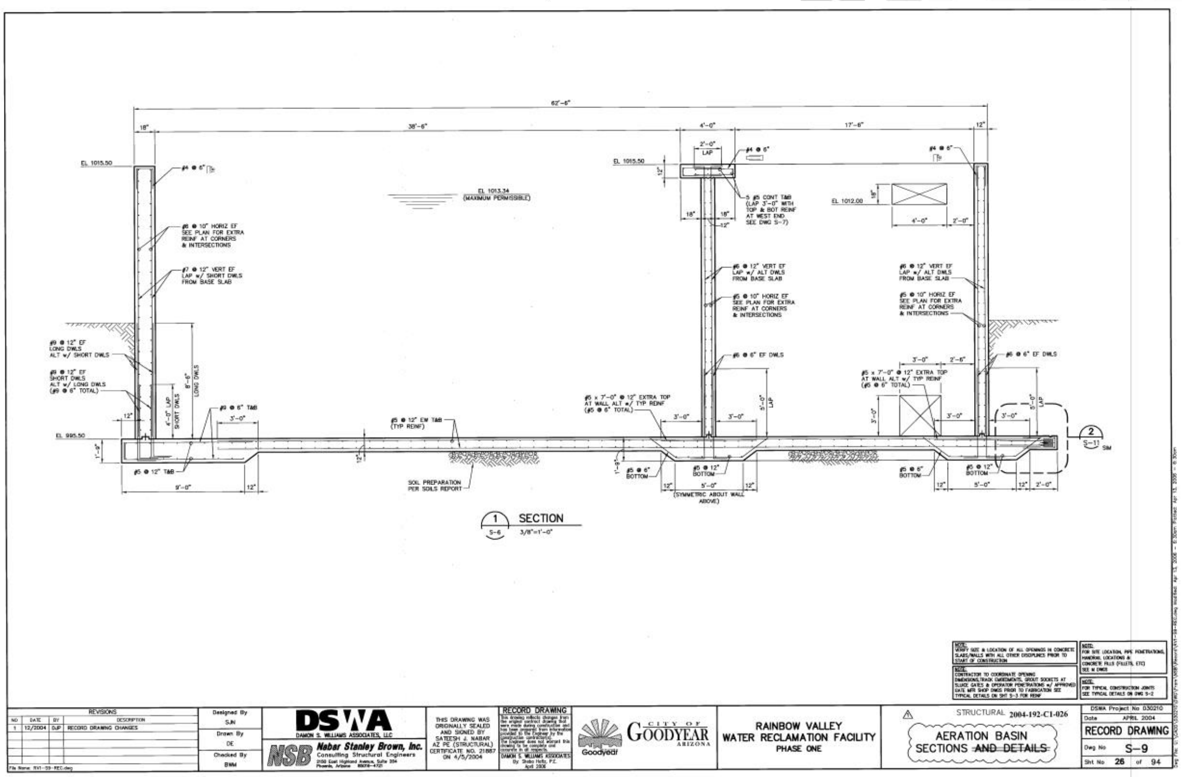


Appendix M: Secondary Treatment Design

Appendix M-1: Rainbow WRD Aeration Basin Record Drawing

Appendix M-1: Aeration Basin Record Drawing (SJN ,2004)





Appendix M-2: Activated Sludge Calculations

Appendix M-2: Activated Sludge Design Calculations

The assumptions for Ks, μm, Kd, Y and MLVSS were taken from Table 23-4 from the Water and Wastewater Engineering Design 2nd Edition by Mackenzie Davis.

Activated Sludge		
Parameter	Value	Units
Q (flow)	3	MGD
So (BOD5)	168.75	mg/L
TSS (Secondary clarifier effluent)	125	mg/L
MLVSS (secondary clarifier effluent)	1,500	mg/L
TSS	10	mg/L
BOD5	10	mg/L
Ks	25	mg/L BOD5
μm	3	d^{-1}
Kd	0.10	d^{-1}
Y	0.60	mg VSS/mg BOD5
RAS	85	%
Was	15	%

Existing primary effluent Q:

$$Q\ (flow) = 3\ MGD * \frac{3785.4118\frac{m^3}{d}}{1\ MGD} = 11{,}356.2354\frac{m^3}{d}$$

Allowable soluble BOD5 in effluent (S):

$$BOD\ of\ TSS = \frac{0.85}{mg\ TSS} * \left(10\frac{mg}{L}\right) = 8.5\frac{mg}{L}BOD$$

$$S = BOD\ in\ effluent - BOD\ of\ TSS = 10\frac{mg}{L} - 8.5\frac{mg}{L} = 1.5\frac{mg}{L}$$

$$Design\ for\ S = 1.5\frac{mg}{L}$$

$$1.5\frac{mg}{L} < 30\frac{mg}{L}, Good$$

Mean cell-residence time (θc):

$$\theta c = \frac{Ks + S}{(S * \mu m) - (S * Kd) - (Ks * Kd)}$$

$$\theta c = \frac{25\frac{mg}{L}BOD + 1.5\frac{mg}{L}BOD}{\left(1.5\frac{mg}{L} * 3\frac{1}{D}\right) - \left(1.5\frac{mg}{L} * 0.10\frac{1}{D}\right) - \left(25\frac{mg}{L} * 0.10\frac{1}{D}\right)} = 14.32\ d = \theta c$$

Check Safety Factor (SF):

$$SF = \frac{\theta c}{\theta cmin} = \theta c(\mu m - Kd)$$

$$SF = 14.32\ days * \left(3\frac{1}{D} - 0.10\frac{1}{D}\right) = 41.54 = SF$$

Conventional loading → implied SF range of 10 > 41.54 < 80, Good

Hydraulic detention time (θ):

$$\theta = \frac{\theta c * Y(So - S)}{x(1 + Kd * \theta c)}$$

$$\theta = \frac{14.32\ d * 0.6 \frac{mg\ VSS}{mg\ BOD}\left(168.75 \frac{mg}{L} - 1.5 \frac{mg}{L}\right)}{1500 \frac{mg}{L} VSS\left(1 + \left(0.10 \frac{1}{D} * 14.32\ d\right)\right)} = 0.39392\ d\ \text{d}$$

$$\theta = 0.39392\ d * \frac{24\ h}{1\ d} = 9.45\ h = \theta$$

Volume of aeration tank (V):

The team will use the same size aeration at the facility.

L= 62'-6"

W= 40'

H= 20'

V= 1,393.1888 m^2

MLVSS fraction of MLSS :

$$Qr = 0.85Q$$

$$Qr = 9{,}652.8 \frac{m^3}{d}$$

$$X' = \frac{x}{0.85} = \frac{1.500 \frac{g}{L}}{0.85} = 1.764 \frac{g}{L} MLSS = X'$$

Return sludge concentration (Xr') of maximum return sludge flow rate (Qr)

$$X'r = \frac{X'\left[Q + Qr - \left(\frac{V}{\theta c}\right)\right]}{Q}$$

$$X'r = \frac{\left(1.764 \frac{g}{L} MLSS\right) * \left[\left(11{,}356.2354 \frac{m^3}{d} + 9{,}652.8 \frac{m^3}{d}\right) - \frac{1{,}393.1888\ m^3}{14.32\ d}\right]}{11{,}356.2354 \frac{m^3}{d}} = 3.248 \frac{g}{L} = X'r$$

$$Qw = \frac{V * X'}{\theta c * X'r} = \frac{1{,}393.1888\ m^3 * 1.764 \frac{g}{L}\ MLSS}{14.32\ d * 3.248 \frac{g}{L}} = 52.84\ \frac{m^3}{d} = Qw$$

$$Mass\ flow\ rate = Qw * Xr' = \left(52.84 \frac{m^3}{d} * 3.248 \frac{g}{L}\right) * \frac{1000\ L}{m^3} * \frac{kg}{1000\ g} = 171.6 \frac{kg}{d}$$

Food to microorganism (F/M)

$$\frac{F}{M} = \frac{Q * So}{V * X} = \frac{11{,}356.2354 \frac{m^3}{d} * 168.75 \frac{mg}{L}}{1{,}393.188 * 1500 \frac{mg}{L}} = 0.917\ d = F/M$$

$$0.917 < 2\ \ , Good$$

Mass of sludge to be wasted each day from new activated plant

$$Yobs = \frac{Y}{1 + (Kd * \theta c)} = \frac{0.6 \frac{Kg\ VSS}{kg\ BOD5}}{1 + \left(0.10 \frac{1}{d} * 14.32\ d\right)} = 0.246 = Yobs$$

Net wasted activated sludge produced each day (VSS)

$$Px = Yobs * Q(So - S) = 0.246 * \left(11{,}356.2354 \frac{m^3}{d}\right) * \left(168.75 \frac{mg}{L} - 1.5 \frac{mg}{L}\right) = 468{,}584.79$$

$$Px = 468{,}584.79 * \left(\frac{kg}{10^3 g}\right) = 468.58 \frac{kg}{d} = Px$$

Total mass produced

$$Px' = Px * \left(\frac{1}{\frac{MLVSS}{MLSS} ratio}\right) = 468.58 \frac{kg}{d} * \left(\frac{1}{0.85}\right) = 398.297 \frac{kg}{d} = Px'$$

Mass of solids lost in effluent

$$(Q - Qw) * Xe' = \left[11{,}356.2354 \frac{m^3}{d} - 52.84 \frac{m^3}{d}\right] * \left(10 \frac{g}{m^3}\right) * \left(\frac{kg}{10^3 g}\right) = 113.03 \frac{kg}{d}$$

Mass to be wasted

$$Mass = Px' - (Q - Qw)Xe' = 398.297 \frac{kg}{d} - 113.03 \frac{kg}{d} = 285.267 \frac{kg}{d}\ (dry\ solids)$$

Mass of oxygen supplied (rbsCOD to bCOD)

$$So = \frac{168.75 \frac{g}{m^3}}{0.85} = 198.53 \frac{g}{m^3}$$

$$S = \frac{1.5 \frac{g}{m^3}}{0.85} = 1.76 \frac{g}{m^3}$$

Mass of O2

$$M_{O2} = Q * (So - S) - 1.42 * (Px)$$

$$M_{O2} = \left(11{,}356.23 \frac{m^3}{d}\right) * \left(198.53 \frac{g}{m^3} - 1.76 \frac{g}{m^3}\right) * \left(\frac{kg}{10^3 g}\right) - 1.42 * \left(468.58 \frac{kg}{d}\right)$$

$$= 1{,}569.136 \frac{kg}{d}\ Oxgen$$

O2 is 23% of air by mass

$$Air = 1{,}569.136 \frac{kg}{d} \left(\frac{1}{0.23}\right) = 6{,}822.33 \frac{kg}{d}\ Air$$

Removal for Activated Sludge

TSS removal: 58-90% (will use 90%)

BOD removal: 85-98% (will use 95%)

Influent Activated Sludge

TSS= 125 mg/L

BOD= 168.75 mg/L

Effluent Activated Sludge

$$TSS = 125 \frac{mg}{L} * 0.10 = 12.5 \frac{mg}{L}$$

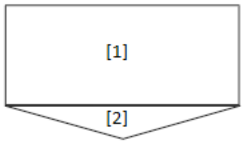
$$BOD = 168.75 \frac{mg}{L} * 0.50 = 8.4 \frac{mg}{L}$$

Appendix M-3: Secondary Clarifier Calculations

Appendix M-3: Secondary Clarifier Design Calculations

Clarifier Volume

[1] Cylinder $= \pi r^2 h$
$Where: r = radias\ (ft), h = height(ft)$
$\pi \times (27.5ft)^2 \times 15ft = \mathbf{35,637.44\ ft^3}$



[2] $Cone = \pi r^2 \frac{h}{3}$
$Where: r = radias\ (ft), h = height(ft)$
$\pi \times (27.5ft)^2 \times \frac{2.43ft}{3} = \mathbf{1,924.42\ ft^3}$

$Total\ clarifier\ volume = 35637.44 + 1924.42 = \mathbf{37,561.86\ ft^3 = 280,982.23\ gallons}$

$$Detention\ Time = \frac{Tank\ volume}{Flow\ rate} = \frac{280982.23\ gallons}{0.75\ MGD} = 0.375\ days = \mathbf{9\ hours}$$

$Surface\ Area = \pi r^2$
$Where: r = radias\ (ft)$
$\pi \times (27.5ft)^2 = \mathbf{2375.83\ ft^2}$

$$Surface\ Overflow\ Rate = \frac{Flow\ Rate}{Surface\ Area} = \frac{0.75\ MGD}{2375.83\ ft^2} = \mathbf{315.7\ gpd/ft^2}$$

$$Weir\ Overflow\ Rate = \frac{Flow\ Rate}{Length\ of\ Weir} = \frac{0.75\ MGD}{172.8\ ft} = \mathbf{4340.28\ gpd/ft}$$

$Length\ of\ Weir = \pi \times d = \pi \times 55' = 172.8\ ft$
$Where\ d = diameter\ (ft)$

Energy Consumption

3/4 HP motor requires 0.559 kW per hour.

$$0.559 \frac{kW}{hour} \times 24\ hours = \mathbf{13.42 \frac{kW}{day}}$$

Settling Velocity

Particle size:
Diameter= 1 mm
Specific gravity= 1.10
Average water temperature= 25 °C
$Water\ density\ (25\ °C) = 997.049 \frac{kg}{m^3} = 1000\ kg/m^3$
$Dynamic\ Viscosity\ (25\ °C) = 0.890\ mPa \cdot s = 0.890 \times 10^{-3}\ Pa \cdot s$

Stokes law:

$$V_s = \frac{g(\rho_s - \rho)d^2}{18\mu}$$

$$V_s = \frac{\left(9.81 \frac{m}{s^2}\right)\left(1100 \frac{kg}{m^3} - 1000 \frac{kg}{m^3}\right)(2 \times 10^{-4} m)^2}{18(8.90 \times 10^{-4}\ Pa \cdot s)} = 2.45 \times 10^{-3} \frac{m}{s}$$

Where:

$g = Acceleration\ due\ to\ gravity\ \left(\frac{m}{s^2}\right)$

$\rho_s = Density\ of\ the\ particle\ \left(\frac{kg}{m^3}\right)$

$\rho = Density\ of\ the\ water\ \left(\frac{kg}{m^3}\right)$

$d = diameter\ of\ the\ particle\ (mm)$
$\mu = viscosity\ of\ the\ water\ (Pa \cdot s)$

Check R:

$$R = \frac{d(v_s)}{v}$$

Where:

$v = kinematic\ viscosity\ \left(\frac{m^2}{s}\right)$

$d = diameter\ of\ particle\ (m)$

$v_s = velocity\ of\ the\ particle\ (\frac{m}{s})$

$kinematic\ viscosity\ (25\ °C) = 0.893\ \times 10^{-6}\frac{m^2}{s}$

$$R = \frac{(0.001\ m)\left(2.45 \times 10^{-3}\frac{m}{s}\right)}{0.893 \times 10^{-6}\frac{m^2}{s}} = 2.74$$

Final Settling Velocity = ***0.00245 m/s = 0.00804 ft/s***

How long will it take this particle to settle in the primary clarifier?

Side water depth= 15'

Settling time= $\frac{15ft}{0.00804\frac{ft}{s}} = 1865.67\ seconds = \mathbf{31\ minutes\ 5.67\ seconds}$

Compare overflow rate to settling velocity:
**The settling velocity must be faster than the overflow rate to ensure that the particle have time to settle in the clarifier before the water flows out of the clarifier*

Overflow rate: 315.7 gpd/ft^2
Settling velocity: 0.00804 ft/s

Convert overflow rate to ft/s:

$1\frac{ft}{s}=7.4805\frac{gpd}{ft^2}$

$$\frac{1}{7.4805} = \frac{x}{315.7}$$

x=42.2 ft/d = 0.00049 ft/s

0.00049 ft/s < 0.00804 ft/s ***OK!***

Appendix N: Advanced Treatment Design

Appendix N-1: Disc Filter Schematic

Appendix N-1: Image of Disc Filter (Technologies, 2021)

3.3.2 HSF2200 type 2, filter without tank

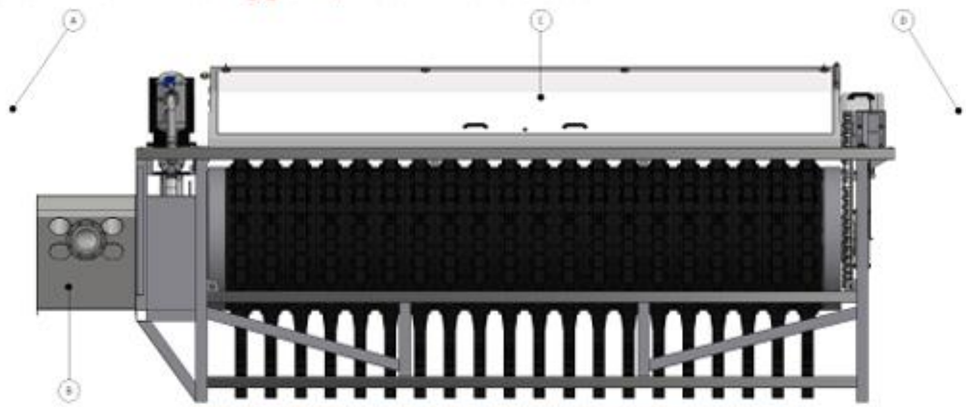


Figure 3.4 Hydrotech Disc filter in the HSF2200 series type 2 (side view).
A. Inlet side
B. Inlet passage
C. Filter cover
D. Outlet side

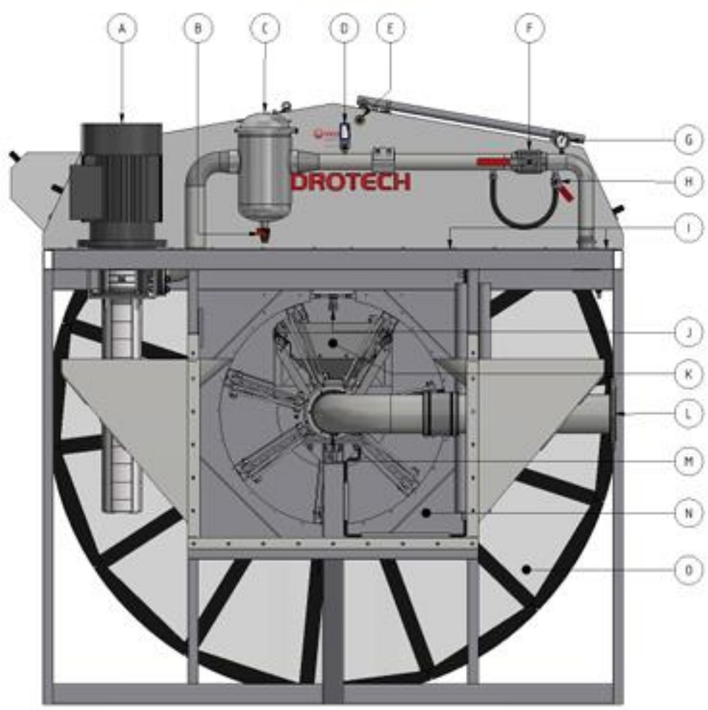


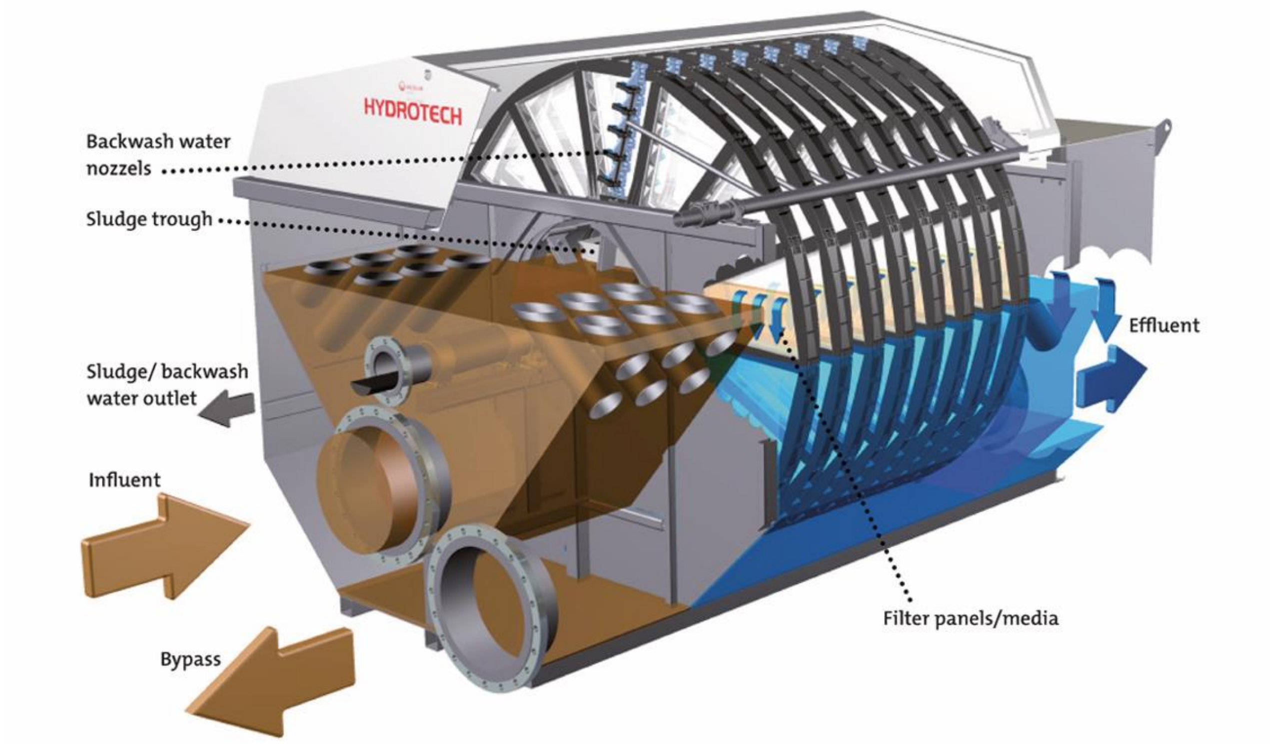
Figure 3.5 Hydrotech Disc filter in the HSF2200 series type 2 (inlet side).

A. Backwash pump (optional)
B. Drain valve
C. Wash water filter
D. Pressure switch (protects pump from running dry) (optinal)
E. Connection, chemical cleaning
F. Shut off valve for wash pipe
G. Manometer
H. By-pass valve for nozzle check

I. Lubrication point
J. Drum lifter
K. Sludge trough
L. Sludge outlet
M. Drum bearing, inlet side
N. Inlet
O. Filter panel

Appendix N-2: Disc Filter Function

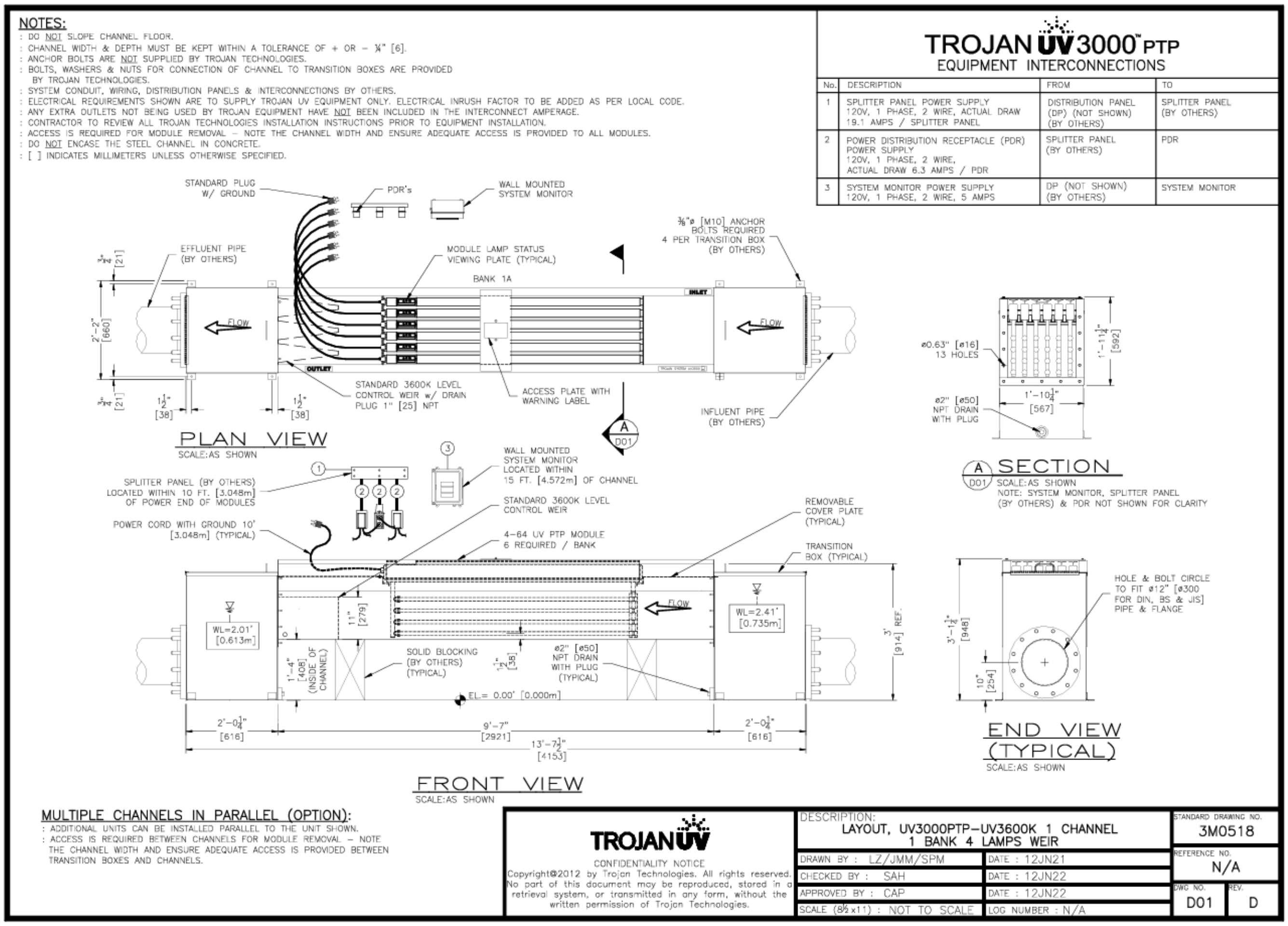
Appendix N-2: Disc Filter Function (Technologies, 2021)



Appendix O: Disinfection Design

Appendix O-1: TROJANUV3000 PTP Record Drawing

Appendix O-1: TROJANUV3000 Record Drawing



Appendix P: Solids Management Design

Appendix P-1: Andritz Decanter Centrifuge D

Appendix P-1: Andritz Decanter Centrifuge D (Separation, ANDRITZ , 2024)

Appendix P-2: Andritz Decanter Centrifuge D Brochure

Appendix P-2: Andritz Decanter Centrifuge D Brochure (Separation, ANDRITZ , 2024)

Getting to know your ANDRITZ decanter centrifuge D

Design optimized to the very smallest detail to provide best results, while ensuring ease of maintenance and providing modularity for optimum fit to your needs.

SCROLL

The scroll of the ANDRITZ decanter centrifuge D is the most flexible scroll available on the market. Its specific open flight design reduces the torque created by the sludge and maximizes the clarification rate. The special cone design leads to high sludge compaction.

- Reduction of sludge conveying torque by 30%, which impacts the gear box lifetime and the scroll drive size positively.
- High cake dryness due to better sludge compaction.
- Excellent centrate quality due to minimized internal turbulences and maximized settling volume.

BOWL

The bowl design is carefully selected to balance the various needs for integrity, stability, smooth operation, minimized windage, high durability, low wear, and easy maintenance, while ensuring the principle process functions. The design is modular to allow an easy fit to different basic process conditions by adjustment of diameter, length, and cone angle. The overall design is optimized to minimize the power consumption and provide the best possible stiffness. ANDRITZ decanter centrifuges are not only factory-tested before delivery to a customer's site, but also extensively type-tested according to international standards to meet all product safety requirements.

COVER

Covers protect you against spillage and touching rotating parts, meet the noise radiation and thus are vital safety features. The shape is optimized for easy cleaning and handling. Different options are available to fit in with your needs, be it highest corrosion resistance, lowest noise radiation, or similar.

Appendix Q: Hydraulic Analysis

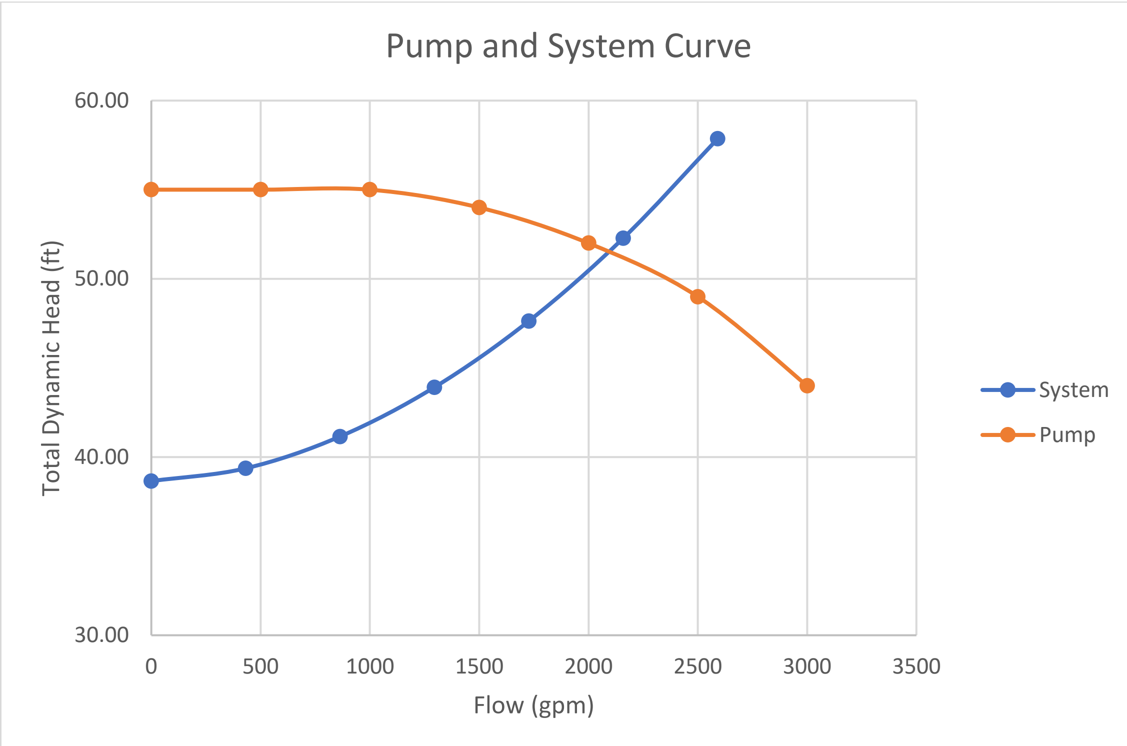
Appendix Q-1: System Analysis

Appendix Q-1: System Analysis Calculations

V (ft/s)	e/d	Nr	f	hf (ft)	hme (ft)	hmb (ft)		THD (ft)	Q (cfs)	Q (gpm)
0	4.29E-05	0.00	0.000	0.00	0.000000	9.98E-12		38.65	0	0
0.1	4.29E-05	20710.06	0.027	0.72	0.000078	9.98E-12		39.37	0.962113	431.7962
0.2	4.29E-05	41420.12	0.023	2.50	0.000311	9.98E-12		41.15	1.924226	863.5924
0.3	4.29E-05	62130.18	0.022	5.27	0.000699	9.98E-12		43.92	2.886338	1295.389
0.4	4.29E-05	82840.24	0.021	8.98	0.001242	9.98E-12		47.63	3.848451	1727.185
0.5	4.29E-05	103550.30	0.020	13.63	0.001941	9.98E-12		52.28	4.810564	2158.981
0.6	4.29E-05	124260.36	0.020	19.21	0.002795	9.98E-12		57.86	5.772677	2590.777
0.7	4.29E-05	144970.41	0.020	25.71	0.003804	9.98E-12		64.36	6.734789	3022.573
0.8	4.29E-05	165680.47	0.019	33.12	0.004969	9.98E-12		71.78	7.696902	3454.37
0.9	4.29E-05	186390.53	0.019	41.46	0.006289	9.98E-12		80.12	8.659015	3886.166
1	4.29E-05	207100.59	0.019	50.71	0.007764	9.98E-12		89.37	9.621128	4317.962
1.1	4.29E-05	227810.65	0.019	60.87	0.009394	9.98E-12		99.53	10.58324	4749.758
1.2	4.29E-05	248520.71	0.019	71.94	0.011180	9.98E-12		110.60	11.54535	5181.554
1.3	4.29E-05	269230.77	0.019	83.93	0.013121	9.98E-12		122.59	12.50747	5613.351
1.4	4.29E-05	289940.83	0.019	96.82	0.015217	9.98E-12		135.48	13.46958	6045.147
1.5	4.29E-05	310650.89	0.018	110.62	0.017469	9.98E-12		149.29	14.43169	6476.943
1.6	4.29E-05	331360.95	0.018	125.33	0.019876	9.98E-12		164.00	15.3938	6908.739
1.7	4.29E-05	352071.01	0.018	140.94	0.022438	9.98E-12		179.62	16.35592	7340.535
1.8	4.29E-05	372781.07	0.018	157.47	0.025155	9.98E-12		196.14	17.31803	7772.332
1.9	4.29E-05	393491.12	0.018	174.90	0.028028	9.98E-12		213.58	18.28014	8204.128
2	4.29E-05	414201.18	0.018	193.24	0.031056	9.98E-12		231.92	19.24226	8635.924

Appendix Q-2: Pump and System Curve

Appendix Q-2: Pump and System Curve



Appendix Q-3: CI40009D Taco Curve Data

Appendix Q-3: CI Series Pump Data Sheet (Taco Comfort Solutions, 2020)

CI Series Pump | Submittal Data

Submittal No: 301-2421D | **Model:** 4009D | **RPM:** 1760 - 60 Hz | **Effective:** January 27, 2020 | **Supersedes:** July 12, 2018

JOB: ______ **REPRESENTATIVE:** ______

ENGINEER: ______ **CONTRACTOR:** ______

PRODUCT DATA

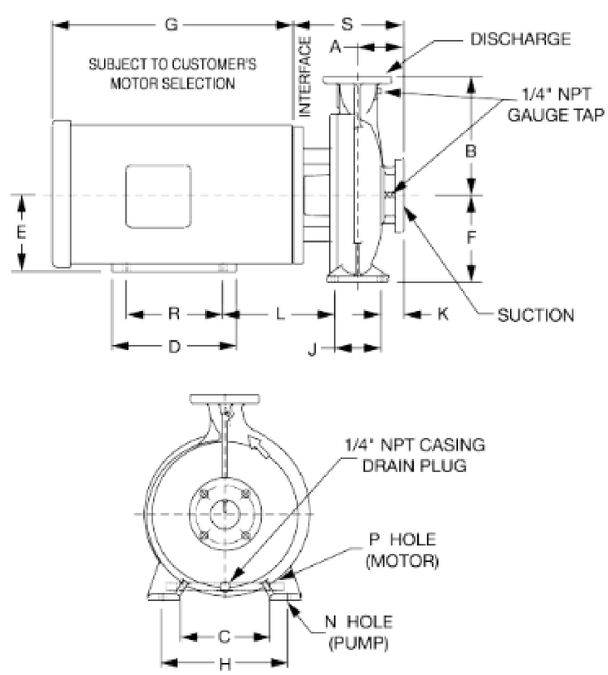
ITEM NO.		**MODEL NO.**	4009D
IMPELLER DIAMETER		**HORSEPOWER**	
GPM		**VOLTAGE**	
HEAD/FT		**RPM**	1760
WEIGHT		**PUMP/MOTOR**	

Configuration	DOE Basic Model Number	PEI Value		Energy Rating
Bare Pump	CI4009D-4P-BP	PEI$_{CL}$	0.92	8
Pump + Motor	CI4009D-4P-PM	PEI$_{CL}$	0.92	8

OPERATING SPECIFICATIONS

FLANGE	PRESSURE	TEMPERATURE
ANSI Class 125	175 PSIG* (1210 KPA)	250°F (120°C)
ANSI Class 250	300 PSIG** (2070 KPA)	250°F (120°C)

Motors: All NEMA Standard (JM Frame)
* In accordance with ANSI Standard B16.1 Class 125
** In accordance with ANSI Standard B16.1 Class 250



DIMENSIONS

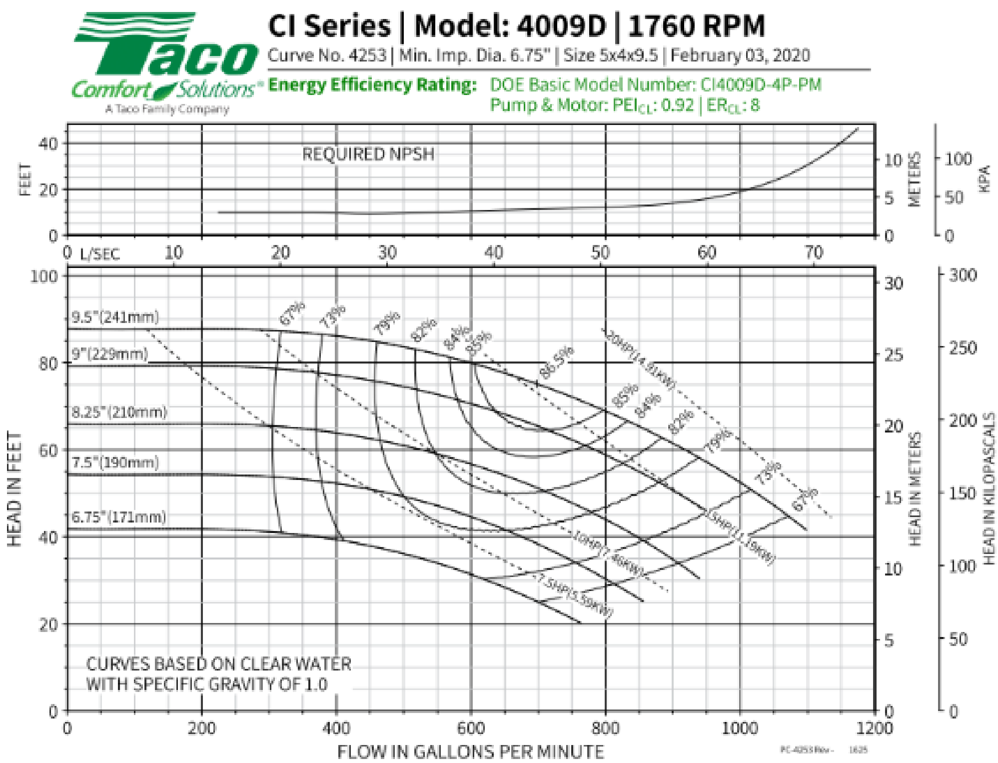
Model No. | 4009D
Flange Size (Suction x Discharge) | 5 x 4 (127 x 102)

	HORSEPOWER	7.5	10	15	20
ODP	MOTOR FRAME	213JM	215JM	254JM	256JM
ODP	G MAX	15.47 (393)	16.60 (422)	19.44 (494)	
ODP	MAXIMUM ASSEMBLY WEIGHT LBS. (KG)	361 (164)		458 (208)	
TEFC	MOTOR FRAME	213JM	215JM	254JM	256JM
TEFC	G MAX	16.64 (423)	18.11 (460)	20.05 (509)	
TEFC	MAXIMUM ASSEMBLY WEIGHT LBS. (KG)	361 (164)		458 (208)	
	A	ANSI Class 125: 5.31 (135)			
		ANSI Class 250: 5.75 (146)			
	B	ANSI Class 125: 12.5 (318)			
		ANSI Class 250: 12.81 (325)			
	C (MOTOR)	8.5 (216)		10.0 (254)	
	D	8.00 (203)		9.5 (241)	11.75 (298)
	E	5.25 (133)		6.25 (159)	
	F	9.84 (250)			
	H (PUMP)	12.4 (315)			
	J	4.72 (120)			
	K	ANSI Class 125: 2.95 (75)			
		ANSI Class 250: 3.39 (86)			
	L	6.92 (176)		8.09 (205)	
	N (PUMP)	0.75 (19)			
	P	0.41 (10)		0.53 (13)	
	R	5.5 (140)	7.0 (178)	8.25 (210)	10.0 (254)
	S	ANSI Class 125: 9.83 (250)		ANSI Class 125: 10.75 (273)	
		ANSI Class 250: 10.27 (261)		ANSI Class 250: 11.19 (284)	

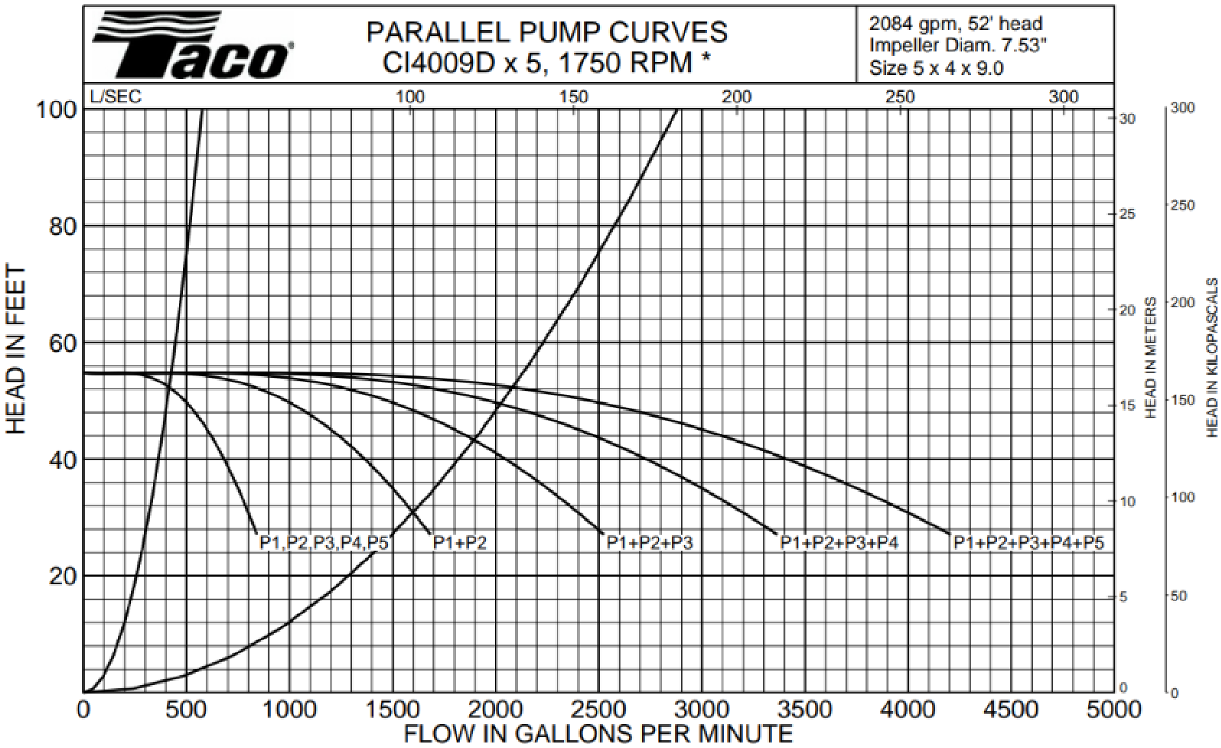
English dimensions are in inches. Metric dimensions are in millimeters.
Metric data is presented in ( ). Do not use for construction purposes unless certified.

MATERIALS OF CONSTRUCTION		CASING	COVER	IMPELLER	WEAR RING	SHAFT	SHAFT SLEEVE	MECHANICAL SEAL	SEAL FLUSH LINE ASSEMBLY
STANDARD CONSTRUCTION	125# FLANGE	Cast Iron ASTM A48/A48M-03 Class 30A	Cast Iron ASTM A48/A48M-03 Class 30A	Bronze ASTM B584 ALLOY C83600 or C84400	N/A	Carbon Steel	Bronze ASTM B584-98A C92200	Ceramic/EPT	N/A
	250# FLANGE	Ductile Iron ASTM A536-84 Grade 65-45-12	Cast Iron ASTM A48/A48M-03 Class 30A	Bronze ASTM B584 ALLOY C83600 or C84400	N/A	Carbon Steel	Bronze ASTM B584-98A C92200	Ceramic/EPT	N/A
OPTIONAL	125# OR 250#	N/A	N/A	Stainless Steel ASTM A351/A 351M-08	Bronze ASTM B584-98A C92200	N/A	Stainless Steel TYPE 303 ASTM A276	Tyngsten Carbide /EPT or Silicon-Carbide/EPT	Copper & Brass C3600

N/A - Not Available



COMMENTS



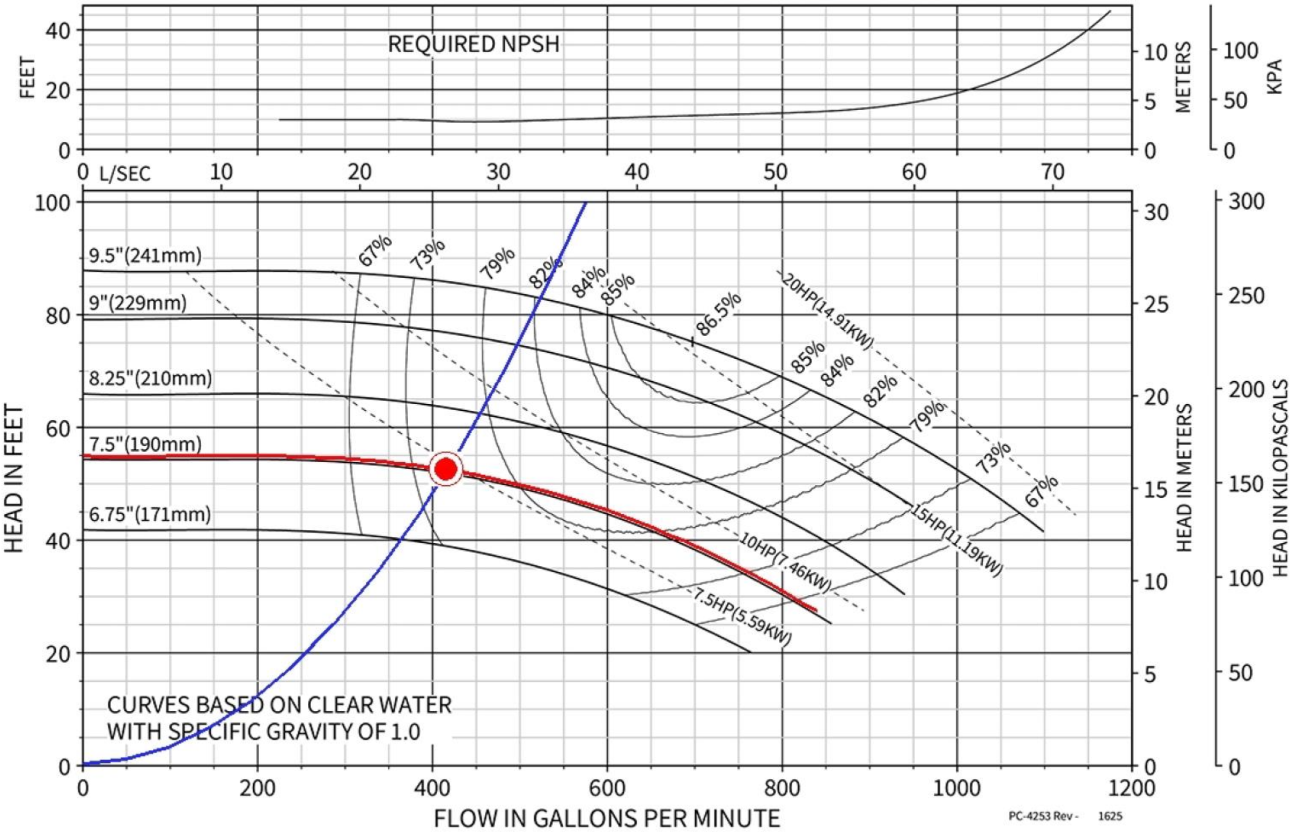
	Flow (gpm)	Head (ft)	BHP
One Pump	---	---	---
Two Pumps	1598	31	14.57
Three Pumps	1894	43	21.85
Four Pumps	2023	49	29.13
Five Pumps	2084	52	36.42

* This model is not suitable for single pump operation.

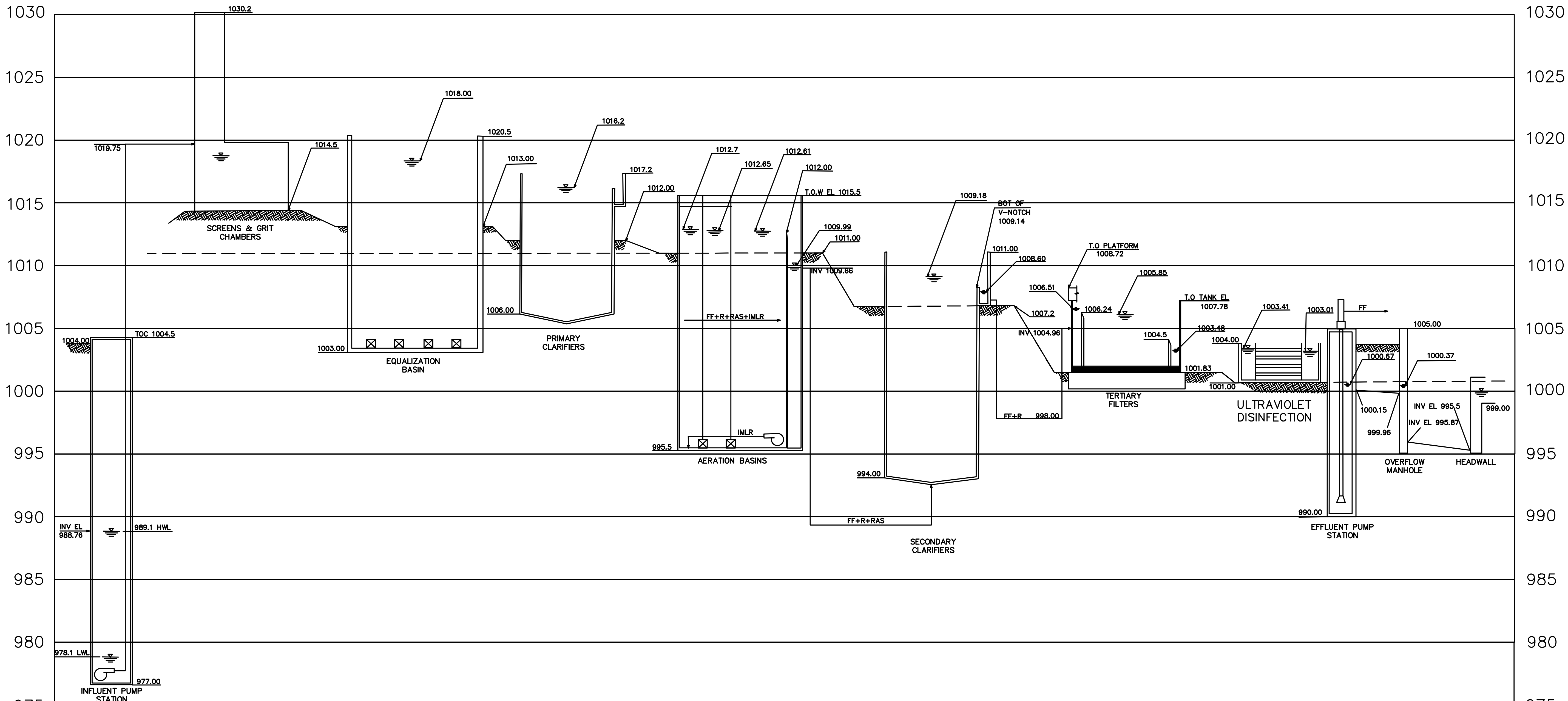
CI Series | Model: 4009D | 1760 RPM

Curve No. 4253 | Min. Imp. Dia. 6.75" | Size 5x4x9.5 | February 03, 2020

Energy Efficiency Rating: DOE Basic Model Number: CI4009D-4P-PM
Pump & Motor: PEI_{CL}: 0.92 | ER_{CL}: 8



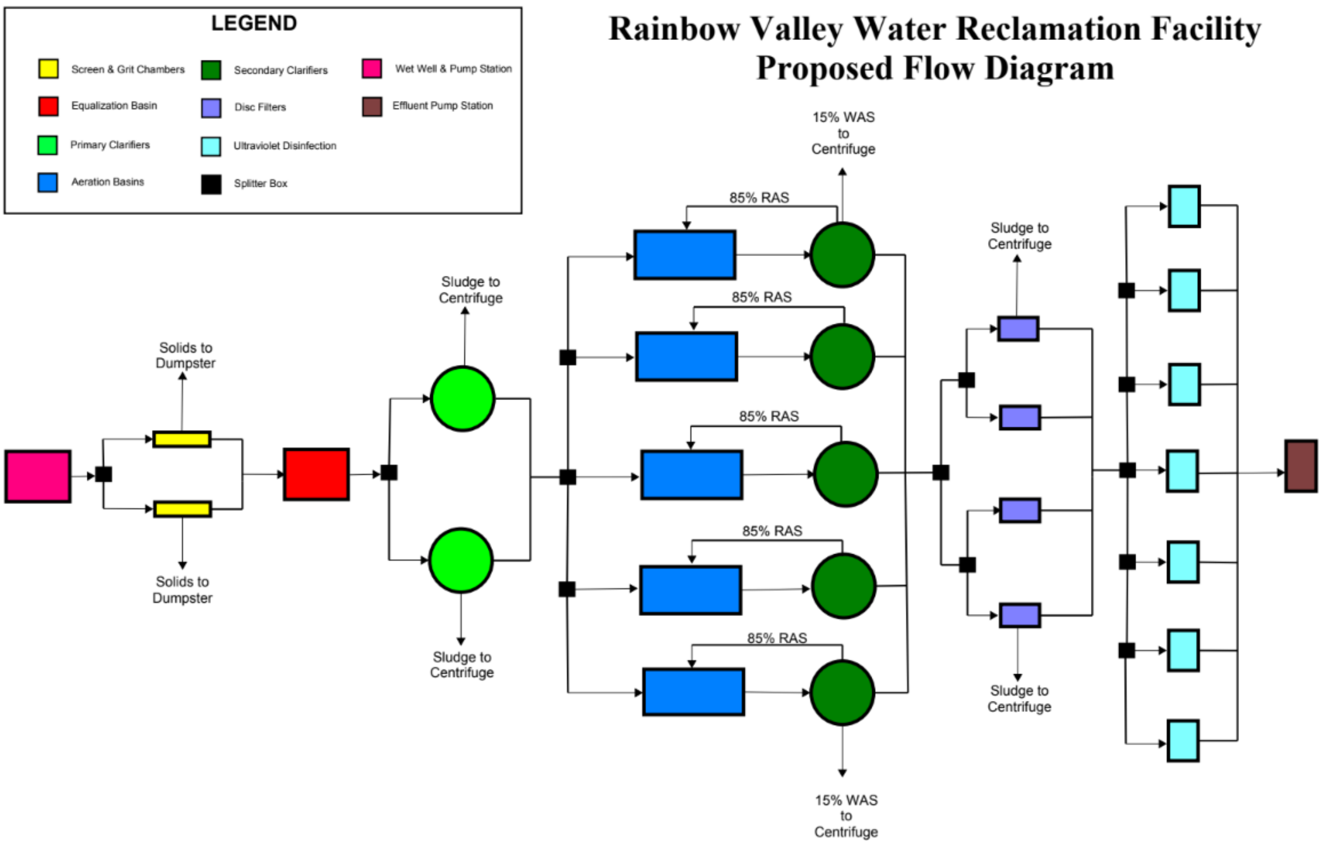
Appendix Q-4: Hydraulic Profile


1030
1025
1020
1015
1010
1005
1000
995
990
985
980
975
1030.2
1019.75
1014.5
SCREENS & GRIT CHAMBERS
1018.00
1020.5
1003.00
EQUALIZATION BASIN
1016.2
1013.00
1017.2
1012.00
1006.00
PRIMARY CLARIFIERS
1012.7
1012.65
1012.61
1012.00
T.O.W EL 1015.5
FF+R+RAS+IMLR
IMLR
995.5
AERATION BASINS
1009.99
1011.00
INV 1009.66
1009.18
BOT OF V-NOTCH 1009.14
1011.00
1008.60
994.00
FF+R+RAS
SECONDARY CLARIFIERS
FF+R
998.00
1007.2
INV 1004.96
T.O PLATFORM 1008.72
1006.51
1006.24
1005.85
T.O TANK EL 1007.78
1004.5
1003.48
1001.83
TERTIARY FILTERS
1003.41
1004.00
1003.01
1001.00
ULTRAVIOLET DISINFECTION
FF
1005.00
1000.67
1000.37
1000.15
999.96
INV EL 995.5
INV EL 995.87
999.00
990.00
EFFLUENT PUMP STATION
OVERFLOW MANHOLE
HEADWALL
1004.00
TOC 1004.5
INV EL 988.76
989.1 HWL
978.1 LWL
977.00
INFLUENT PUMP STATION

Appendix R: Process Flow Diagram

Appendix R-1: Process Flow Diagram

Appendix R-1: Process Flow Diagram



Appendix S: Economic Analysis

Appendix S-1: Opinion of Probable Construction Cost

Appendix S-1: Opinion of Probable Construction Cost

Engineers Opinion of Probable Cost					
Item #	**Description**	**Quantity**	**Unit**	**$/Unit**	**Total Cost**
	EARTHWORK	1	LS	$65,000	$65,000
	CONCRETE EXCAVATION	520	CY	$19	$9,968
	PISTAWORKS MODEL 7.0B	2	EA	$798,750	$1,597,500
	CONCRETE SLAB FOR SCREEN/GRIT CHAMBER BUILDING	135	CY	$1,013	$136,755
	CINDERBLOCKS FOR SCREEN/GRIT CHAMBER BUILDING	7518	EA	$3	$18,870
	ACTIVATED CARBON DRUMS	4	EA	$2,772	$11,088
	BLOWERS FOR EQUALIZATION BASIN	4	EA	$1,065	$4,260
	CONCRETE FOR EQUALIZATION BASIN	95	CY	$1,013	$96,235
	ENVIRODYNE PRIMARY CLARFIER EQUIPEMENT	2	EA	$244,950	$489,900
	CONCRETE FOR PRIMARY CLARIFIER TANK	842	CY	$1,013	$852,946
	ACTIVATED SLUDGE CONSTRUCTION (AERATION BASINS AND SECONDARY CLARIFIERS)	1	LS	$22,000,000	$22,000,000
	VEOLIA HYDROTECH DISC FILTER	2	EA	$383,400	$766,800
	TROJAN UV 3000 PTP	7	EA	$186,375	$1,304,625
	ANDRITZ D4L DECANTER CENTRIFUGE	3	EA	$441,975	$1,325,925
	20' JDV EQUIPMENT CONVEYOR BELT	3	EA	$18,105	$54,315
	60 HZ PUMP W/ CAPACITY OF 400 GPM	5	EA	$9,407	$47,036
	42" COMMERCIAL STEEL PIPE	580	LF	$818	$474,394
	21" COMMERCIAL STEEL PIPE	1470	LF	$515	$757,726
	SPLITTER BOX	13	EA	$7,988	$103,838
	VALVES AND FITTINGS	1	LS	$1,500,000	$1,500,000
Total					**$31,617,181**

*All prices in the analysis include labor, installation, and construction costs

Appendix S-2: Operation and Maintenance Cost

Appendix S-2: Operation and Maintenance Cost

Operation & Maintenance Costs					
	Item	Quantity	Unit	$/Unit	Total Cost
Influent Pumps					
Operation Cost	Energy Consumption	47584	kW-hr/year	$0.13	$6,186
Maintenance Cost	Inspect Pumps for Solids Blockage	24	per year	-	$0
				Total for 5 Pumps	**$30,930**
Screen/Grit Chamber					
Operation Cost	Energy Consumption	18370	kW-hr/year	$0.13	$2,388
Maintenance Cost	Screen Gearbox, Chamber Gear, & Grit Washer Gearbox Oil Change	2	EA/year	$790	$1,580
	Fill Grease Bearing on Classifier	12	EA/year	$20	$240
	Replace Screen Brushes	1	EA/year	$1,500	$1,500
	Grease Pump Motor	2	EA/year	$45	$90
				Total for 2 Systems	**$11,596**
Equalization Basin					
Operation Cost	Energy Consumption	19587	kW-hr/year	$0.13	$2,546
Maintenance Cost	Check for Obstructions in Blowers	12	EA/year	-	$0
				Total for 1 System	**$2,546**
Primary Clarifier					
Operation Cost	Energy Consumption	4898	kW-hr/year	$0.13	$637
Maintenance Cost	Grease Winsmith Reducer	12	EA/year	$45	$540
	Grease Cone Reducer	1	EA/year	$45	$45
	Primary Gear Reducer Winsmith, Seocndary Gear Reducer Cone, &	1	EA/year	$260	$260
				Total for 2 Clarifiers	**$2,963**
Activated Sludge					
Operation & Maintenance Costs		1	LS	$4,414,776	**$4,414,776**
Disc Filter					
Operation Cost	Energy Consumption	9855	kW-hr/year	$0.13	$1,281
Maintenance Cost	Grease Pump Bearings	2	EA/year	$45	$90
	Inspect Drum Bearings	2	EA/year	-	$0
	Inspect Disc and Drum seals	2	EA/year	-	$0
	Grease Drum Bearings	26	EA/year	$45	$1,170
				Total for 4 Disc Filters	**$10,165**
Ultraviolet Disinfection					
Operation Cost	Energy Consumption	5406	kW-hr/year	$0.13	$703
Maintenance Cost	Replace Bulbs	48	EA/year	$127	$6,096
	Clean Glass Sleeves	48	EA/year	$70	$3,360
				Total for 7 Ultraviolet Disinfection Systems	**$71,111**
Centrifuge					
Operation Cost	Energy Consumption	7683	kW-hr/year	$0.13	$999
Maintenance Cost	Remove Any Accumulated Solids	12	EA/year	-	$0
	Replace Filter & Filter System if Necessary	2	EA/year	$1,020	$2,040
	Change Oil in Hydraulic Pump	2	EA/year	$260	$520
	Clean the Hydraulic Drive Oil Tank	2	EA/year	-	$0
	Clean the Hydraulic Drive Suction Strainer	2	EA/year	-	$0
				Total for 4 Centrifuges	**$14,235**
Labor					
Operation Cost	Grade 1 Operator	1	LS	$53,435	$53,435
	Grade 2 or 3 Operator	1	LS	$59,613	$59,613
	Grade 4 Operator	1	LS	$72,176	$72,176
				Total Labor Cost	**$185,224**
Total Operation & Maintenance Costs Per Year					**$4,731,951**

Appendix T: Project Hours

Appendix T-1: Preliminary Hours

Task	SENG	ENG	EIT	INT	Total Task Hours
Task 1: Preliminary Assessment					
Task 1.1: WEF Application	1	1	0	0	2
Task 1.2: Additional Treatments Research	0	0	20	25	45
Task 1.3: Research Regulations	0	0	20	10	30
Task 2: Site Assessment					
Task 2.1: Site Visit	3	3	3	3	12
Task 2.2: Data Analysis	0	5	5	0	10
Task 2.3: Determine Topography	0	5	3	2	10
Task 3: Treatment Design					
Task 3.1: Determine Plant Requirements	0	1	4	0	5
Task 3.2: Preliminary Treatment					
Task 3.2.1: Determine Criteria	0	3	5	2	10
Task 3.2.2: Develop Preliminary Treatment Alternatives	0	8	17	5	30
Task 3.2.3: Select Best Alternative	2	2	0	0	4
Task 3.3: Primary Treatment					
Task 3.3.1: Determine Criteria	0	4	7	4	15
Task 3.3.2: Develop Primary Treatment Alternatives	0	10	35	10	55
Task 3.3.3: Select Best Alternative	5	5	0	0	10
Task 3.4: Secondary Treatment					
Task 3.4.1: Determine Criteria	0	4	8	4	16
Task 3.4.2: Develop Secondary Treatment Alternatives	0	10	35	10	55
Task 3.4.3: Select Best Alternative	5	5	0	0	10
Task 3.5: Advanced Treatment					
Task 3.5.1: Determine Criteria	0	4	8	4	16
Task 3.5.2: Develop Advanced Treatment Alternatives	0	10	35	10	55
Task 3.5.3: Select Best Alternative	5	5	0	0	10
Task 3.6: Disinfection					
Task 3.6.1: Determine Criteria	0	3	5	4	12
Task 3.6.2: Develop Disinfection Alternatives	0	10	25	5	40
Task 3.6.3: Select Best Alternative	3	3	0	0	6

Task 3.7: Solids Management					
Task 3.7.1: Determine Criteria	0	4	5	4	13
Task 3.7.2: Develop Solids Management Alternatives	0	10	25	5	40
Task 3.7.3: Select Best Alternative	4	4	0	0	8
Task 4: Final Design					
Task 4.1: Site Layout	4	6	10	0	20
Task 4.2: Hydraulic Analysis					
Task 4.2.1: System Analysis	5	10	25	5	45
Task 4.2.2: Pump Selection	3	5	15	2	25
Task 4.3: Construction Phasing	2	8	10	0	20
Task 4.4: Economic Analysis					
Task 4.4.1: Construction Cost	0	10	10	5	25
Task 4.4.2: Maintenance and Operation Costs	0	3	10	2	15
Task 4.4.3: Life Cycle Cost Analysis	0	3	5	2	10
Task 5: Project Impacts Analysis					
Task 5: Project Impacts Analysis	1	4	0	0	5
Task 6: Project Deliverables					
Task 6.1: 30% Deliverable	2	5	5	5	17
Task 6.2: 60% Deliverable	2	5	5	5	17
Task 6.3: 90% Deliverable	4	10	10	6	30
Task 6.4: 100% Deliverable	4	5	5	5	19
Task 6.5: Competition Final Report	4	5	5	5	19
Task 6.6: Competition Final Presentation	4	5	5	5	19
Task 7: Project Management					
Task 7.1: Meetings	15	20	20	20	75
Task 7.2: Schedule Management	5	5	0	0	10
Task 7.3: Resource Management	5	5	0	0	10
Subtotal	**88**	**233**	**405**	**174**	
Total Person Hours					**900**

Appendix T-2: Updated Hours

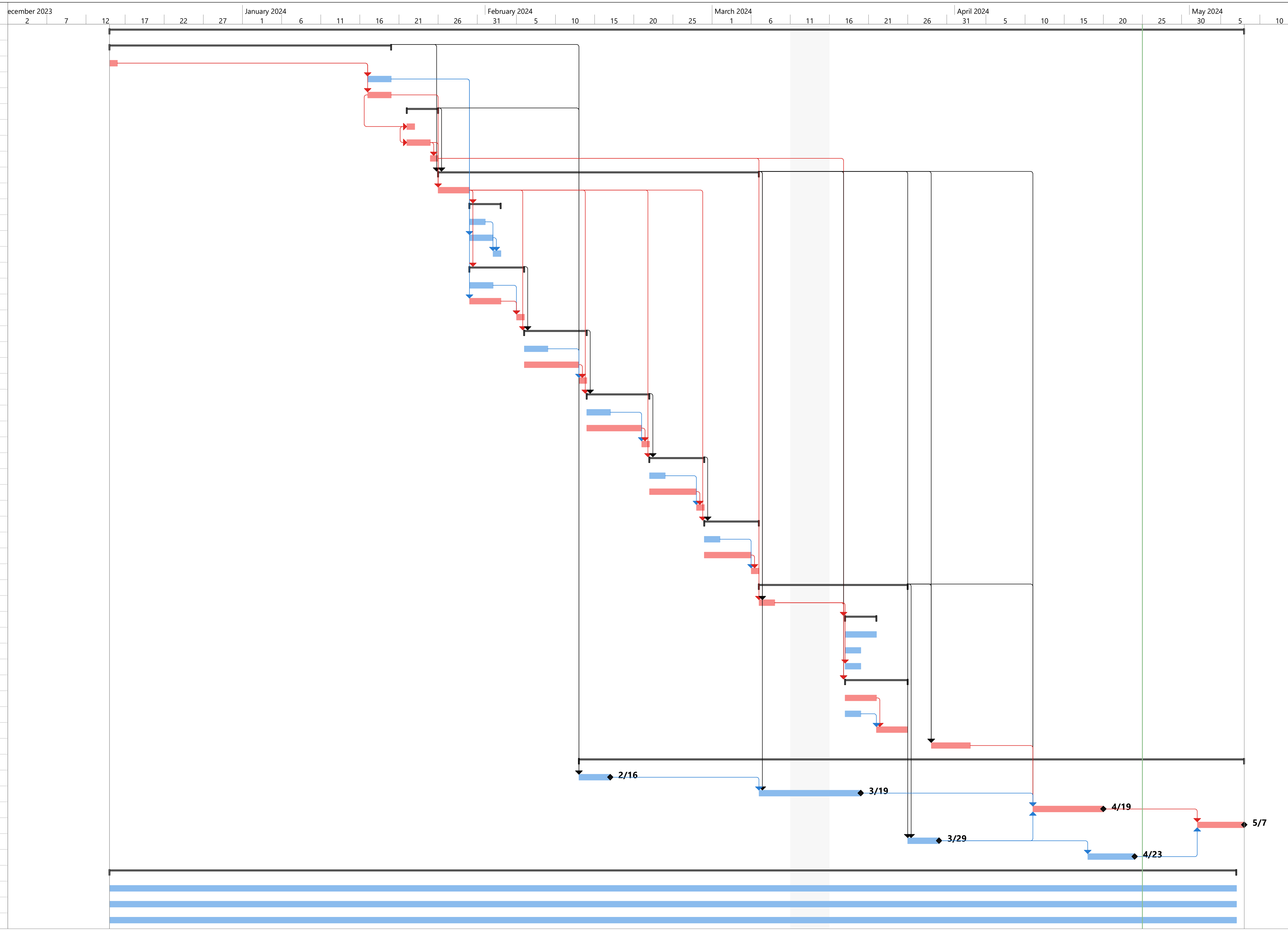
Task	SENG	ENG	EIT	INT	Total Task Hours
Task 1: Preliminary Assessment					
Task 1.1: WEF Application	1	0	0	0	1
Task 1.2: Additional Treatments Research	0	0	19	21	40
Task 1.3: Research Regulations	0	0	12	14	26
Task 2: Site Assessment					
Task 2.1: Site Visit	3	16	10	6	35
Task 2.2: Data Analysis	0	7	5	2	14
Task 2.3: Determine Topography	0	2	3	4	9
Task 3: Treatment Design					
Task 3.1: Determine Plant Requirements	0	3	4	1	8
Task 3.2: Preliminary Treatment					
Task 3.2.1: Determine Criteria	0	7	3	0	10
Task 3.2.2: Develop Preliminary Treatment Alternatives	0	12	24	18	54
Task 3.2.3: Select Best Alternative	1	0	2	5	8
Task 3.3: Primary Treatment					
Task 3.3.1: Determine Criteria	0	5	4	0	9
Task 3.3.2: Develop Primary Treatment Alternatives	1	14	24	24	63
Task 3.3.3: Select Best Alternative	1	0	1	4	6
Task 3.4: Secondary Treatment					
Task 3.4.1: Determine Criteria	1	1	2	1	5
Task 3.4.2: Develop Secondary Treatment Alternatives	0	10	21	11	42
Task 3.4.3: Select Best Alternative	1	1	1	2	5
Task 3.5: Advanced Treatment					
Task 3.5.1: Determine Criteria	0	1	4	2	7
Task 3.5.2: Develop Advanced Treatment Alternatives	0	3	8	3	14
Task 3.5.3: Select Best Alternative	2	0.5	0	0.5	3
Task 3.6: Disinfection					
Task 3.6.1: Determine Criteria	0	0.5	0.5	2	3
Task 3.6.2: Develop Disinfection Alternatives	0	3	4	4	11
Task 3.6.3: Select Best Alternative	1	0.5	0	1	2.5
Task 3.7: Solids Management					

Task 3.7.1: Determine Criteria	0	0	1	2	3
Task 3.7.2: Develop Solids Management Alternatives	0	1	4	0	5
Task 3.7.3: Select Best Alternative	1	0	0.5	1	2.5
Task 4: Final Design					
Task 4.1: Site Layout	0	0	5	4	9
Task 4.2: Hydraulic Analysis					
Task 4.2.1: System Analysis	1	5	5	5	16
Task 4.2.2: Pump Selection	0	4	6	2	12
Task 4.3: Construction Phasing	0	2	3	4	9
Task 4.4: Economic Analysis					
Task 4.4.1: Construction Cost	0	2	5	2	9
Task 4.4.2: Maintenance and Operation Costs	0	2	4	5	11
Task 4.4.3: Life Cycle Cost Analysis	0	0	0	1	1
Task 5: Project Impacts Analysis					
Task 5: Project Impacts Analysis	0	0	0	2	2
Task 6: Project Deliverables					
Task 6.1: 30% Deliverable	3	12	22	15	52
Task 6.2: 60% Deliverable	5	20	15	13	53
Task 6.3: 90% Deliverable	1	3	10	11	25
Task 6.4: 100% Deliverable	0	0	1	0	1
Task 6.5: Competition Final Report	4	16	14	9	43
Task 6.6: Competition Final Presentation	3	4	6	6	19
Task 7: Project Management					
Task 7.1: Meetings	24	37	44	32	137
Task 7.2: Schedule Management	2	1	5	1	9
Task 7.3: Resource Management	0	1	1	1	3
Subtotal	**56**	**196.5**	**303**	**241.5**	
Total Person Hours					**797**

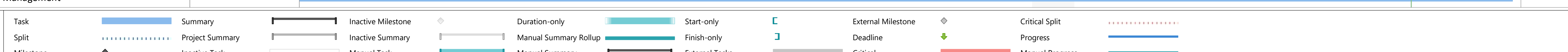
Appendix U: Project Schedule

Appendix U-1: Preliminary Schedule

ID	Task Name
1	**WEF Capstone**
2	**Task 1: Preliminary Assessment**
3	Task 1.1: WEF Application
4	Task 1.2: Additional Treatments Research
5	Task 1.3: Research Regulations
6	**Task 2: Site Assessment**
7	Task 2.1: Site Visit
8	Task 2.2: Data Analysis
9	Task 2.3: Determine Topography
10	**Task 3: Treatment Design**
11	Task 3.1: Determine Plant Requirements
12	**Task 3.2: Preliminary Treatment**
13	Task 3.2.1: Determine Criteria
14	Task 3.2.2: Develop Preliminary Treatment Alternatives
15	Task 3.2.3: Select Best Alternative
16	**Task 3.3: Primary Treatment**
17	Task 3.3.1: Determine Criteria
18	Task 3.3.2: Develop Primary Treatment Alternatives
19	Task 3.3.3: Select Best Alternative
20	**Task 3.4: Secondary Treatment**
21	Task 3.4.1: Determine Criteria
22	Task 3.4.2: Develop Secondary Treatment Alternatives
23	Task 3.4.3: Select Best Alternative
24	**Task 3.5: Advanced Treatment**
25	Task 3.5.1: Determine Criteria
26	Task 3.5.2: Develop Advanced Treatment Alternatives
27	Task 3.5.3: Select Best Alternative
28	**Task 3.6: Disinfection**
29	Task 3.6.1: Determine Criteria
30	Task 3.6.2: Develop Disinfection Alternatives
31	Task 3.6.3: Select Best Alternative
32	**Task 3.7: Solids Management**
33	Task 3.7.1: Determine Criteria
34	Task 3.7.2: Develop Solids Management Alternatives
35	Task 3.7.3: Select Best Alternative
36	**Task 4: Final Design**
37	Task 4.1: Site Layout
38	**Task 4.2: Hydraulic Analysis**
39	Task 4.2.1: System Analysis
40	Task 4.2.2: Pump Selection
41	Task 4.3: Construction Phasing
42	**Task 4.4: Economic Analysis**
43	Task 4.4.1: Construction Costs
44	Task 4.4.2: Maintenance and Operation Costs
45	Task 4.4.3: Life Cycle Cost Analysis
46	**Task 5: Project Impacts Analysis**
47	**Task 6: Project Deliverables**
48	Task 6.1: 30% Deliverable
49	Task 6.2: 60% Deliverable
50	Task 6.3: 90% Deliverable
51	Task 6.4: 100% Deliverable
52	Task 6.5: Competition Final Report
53	Task 6.6: Competition Final Presentation
54	**Task 7: Project Management**
55	Task 7.1: Meetings
56	Task 7.2: Schedule Management
57	Task 7.3: Resource Management

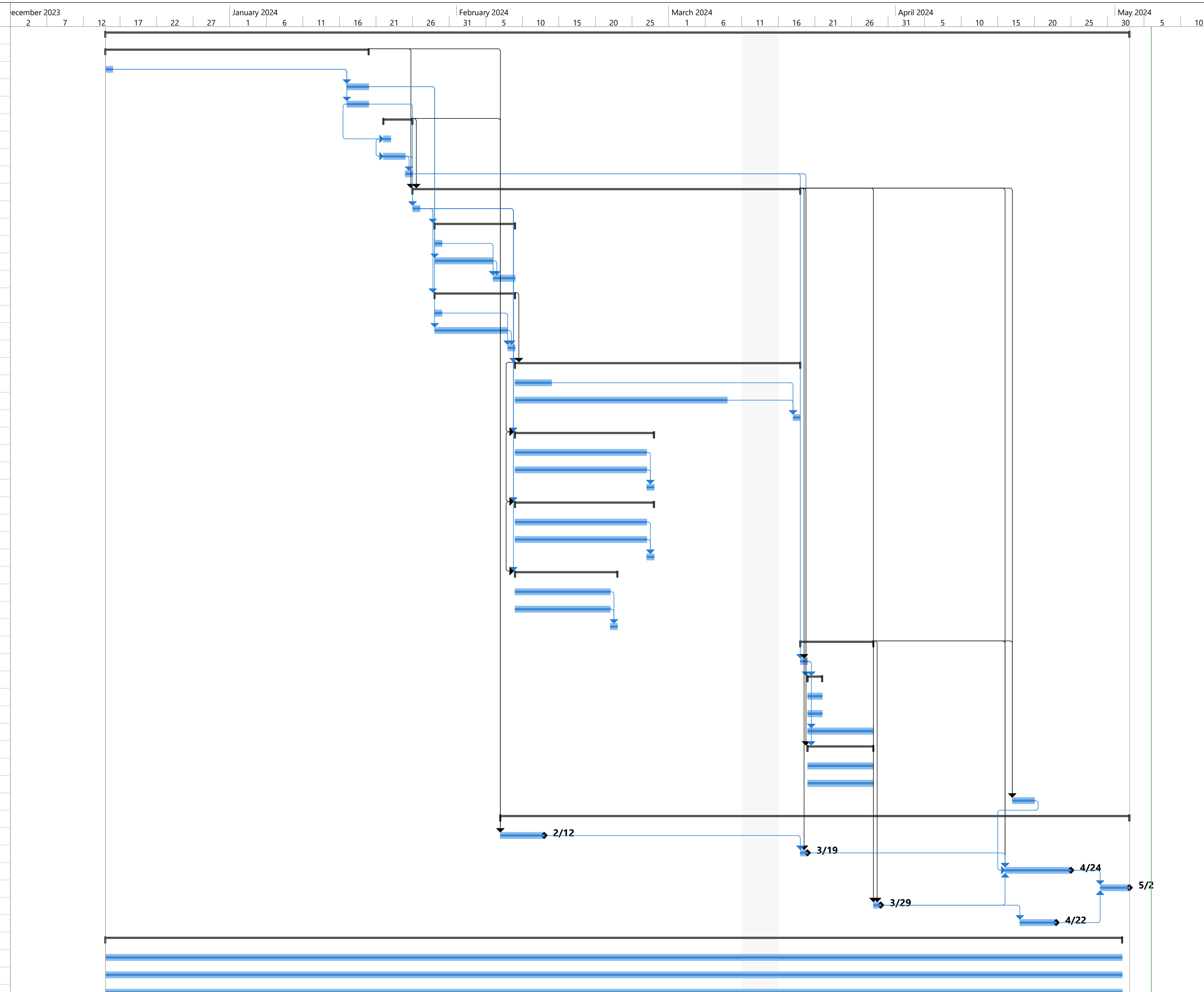


Project: WEF Capstone Schedul
Date: Thu 4/25/24



Appendix U-2: Updated Schedule

ID	Task Name	Duration	Actual Finish
1	**WEF Capstone**	**95 days**	**Thu 5/2/24**
2	**Task 1: Preliminary Assessment**	**26 days**	**Fri 1/19/24**
3	Task 1.1: WEF Application	1 day	Fri 12/15/23
4	Task 1.2: Additional Treatments Research	3 days	Fri 1/19/24
5	Task 1.3: Research Regulations	3 days	Fri 1/19/24
6	**Task 2: Site Assessment**	**4 days**	**Thu 1/25/24**
7	Task 2.1: Site Visit	1 day	Mon 1/22/24
8	Task 2.2: Data Analysis	3 days	Wed 1/24/24
9	Task 2.3: Determine Topography	1 day	Thu 1/25/24
10	**Task 3: Treatment Design**	**32 days**	**Mon 3/18/24**
11	Task 3.1: Determine Plant Requirements	1 day	Fri 1/26/24
12	**Task 3.2: Preliminary Treatment**	**9 days**	**Thu 2/8/24**
13	Task 3.2.1: Determine Criteria	1 day	Mon 1/29/24
14	Task 3.2.2: Develop Preliminary Treatment Alternatives	6 days	Mon 2/5/24
15	Task 3.2.3: Select Best Alternative	3 days	Thu 2/8/24
16	**Task 3.3: Primary Treatment**	**9 days**	**Thu 2/8/24**
17	Task 3.3.1: Determine Criteria	1 day	Mon 1/29/24
18	Task 3.3.2: Develop Primary Treatment Alternatives	8 days	Wed 2/7/24
19	Task 3.3.3: Select Best Alternative	1 day	Thu 2/8/24
20	**Task 3.4: Secondary Treatment**	**22 days**	**Mon 3/18/24**
21	Task 3.4.1: Determine Criteria	3 days	Tue 2/13/24
22	Task 3.4.2: Develop Secondary Treatment Alternatives	21 days	Fri 3/8/24
23	Task 3.4.3: Select Best Alternative	1 day	Mon 3/18/24
24	**Task 3.5: Advanced Treatment**	**13 days**	**Tue 2/27/24**
25	Task 3.5.1: Determine Criteria	12 days	Mon 2/26/24
26	Task 3.5.2: Develop Advanced Treatment Alternatives	12 days	Mon 2/26/24
27	Task 3.5.3: Select Best Alternative	1 day	Tue 2/27/24
28	**Task 3.6: Disinfection**	**13 days**	**Tue 2/27/24**
29	Task 3.6.1: Determine Criteria	12 days	Mon 2/26/24
30	Task 3.6.2: Develop Disinfection Alternatives	12 days	Mon 2/26/24
31	Task 3.6.3: Select Best Alternative	1 day	Tue 2/27/24
32	**Task 3.7: Solids Management**	**10 days**	**Thu 2/22/24**
33	Task 3.7.1: Determine Criteria	9 days	Wed 2/21/24
34	Task 3.7.2: Develop Solids Management Alternatives	9 days	Wed 2/21/24
35	Task 3.7.3: Select Best Alternative	1 day	Thu 2/22/24
36	**Task 4: Final Design**	**8 days**	**Thu 3/28/24**
37	Task 4.1: Site Layout	1 day	Tue 3/19/24
38	**Task 4.2: Hydraulic Analysis**	**2 days**	**Thu 3/21/24**
39	Task 4.2.1: System Analysis	2 days	Thu 3/21/24
40	Task 4.2.2: Pump Selection	2 days	Thu 3/21/24
41	Task 4.3: Construction Phasing	7 days	Thu 3/28/24
42	**Task 4.4: Economic Analysis**	**7 days**	**Thu 3/28/24**
43	Task 4.4.1: Construction Costs	7 days	Thu 3/28/24
44	Task 4.4.2: Maintenance and Operation Costs	7 days	Thu 3/28/24
45	**Task 5: Project Impacts Analysis**	3 days	Fri 4/19/24
46	**Task 6: Project Deliverables**	**57 days**	**Thu 5/2/24**
47	Task 6.1: 30% Deliverable	4 days	Mon 2/12/24
48	Task 6.2: 60% Deliverable	1 day	Tue 3/19/24
49	Task 6.3: 90% Deliverable	7 days	Wed 4/24/24
50	Task 6.4: 100% Deliverable	4 days	Thu 5/2/24
51	Task 6.5: Competition Final Report	1 day	Fri 3/29/24
52	Task 6.6: Competition Final Presentation	3 days	Mon 4/22/24
53	**Task 7: Project Management**	**94 days**	**Wed 5/1/24**
54	Task 7.1: Meetings	94 days	Wed 5/1/24
55	Task 7.2: Schedule Management	94 days	Wed 5/1/24
56	Task 7.3: Resource Management	94 days	Wed 5/1/24



Project: WEF Capstone Schedul
Date: Mon 5/6/24

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 V: References

Appendix V-1: References

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